



Thetis Engineering

32 Valley Road, Chandlers Ford
Structural Calculations

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1.0 Introduction

Thetis Engineering Ltd has been engaged by Mr Mike Ragi to provide structural calculations to justify the proposed structural works at 32 Valley Road, Chandlers Ford.

The site contained a detached bungalow with loft conversion and a detached garage, which were demolished earlier and are proposed to be replaced by a two-storey, single occupancy dwelling with an attached garage. The proposed structure comprises traditional timber floors/roof, supported on timber stud walls on first floor and load bearing masonry walls below, with a beam and block ground floor. The proposed new building is designed to occupy a similar footprint to the one being replaced.

Disproportionate Collapse:

The building is a single occupancy house not exceeding 4 storeys. As such the building is classed as Class 1 in accordance with Section A3 of Approved Document A – Structure of the Building Regulations 2000. Therefore, no additional robustness requirements are obligatory if designed to BS EN 1993 due to low consequences of failure.

Site location and geology

Site location is in Chandlers Ford, nearby borehole logs indicate expected water level approximately 1m below ground, very soft clay or silt layer followed by firmer clay or sand.

Referring to Figure 1 (below) and assuming ground conditions as soft clay/silt down to at least 2m below ground, the founding level to be on a suitable bearing strata of at least medium soft clay, medium dense sand, gravel or silt with minimum ground bearing pressure taken as 50kN/m² to the satisfaction of building control officer.

Therefore for further calculations allowable groundbearing pressure is taken as 50kN/m² at minimum 2m below ground level.

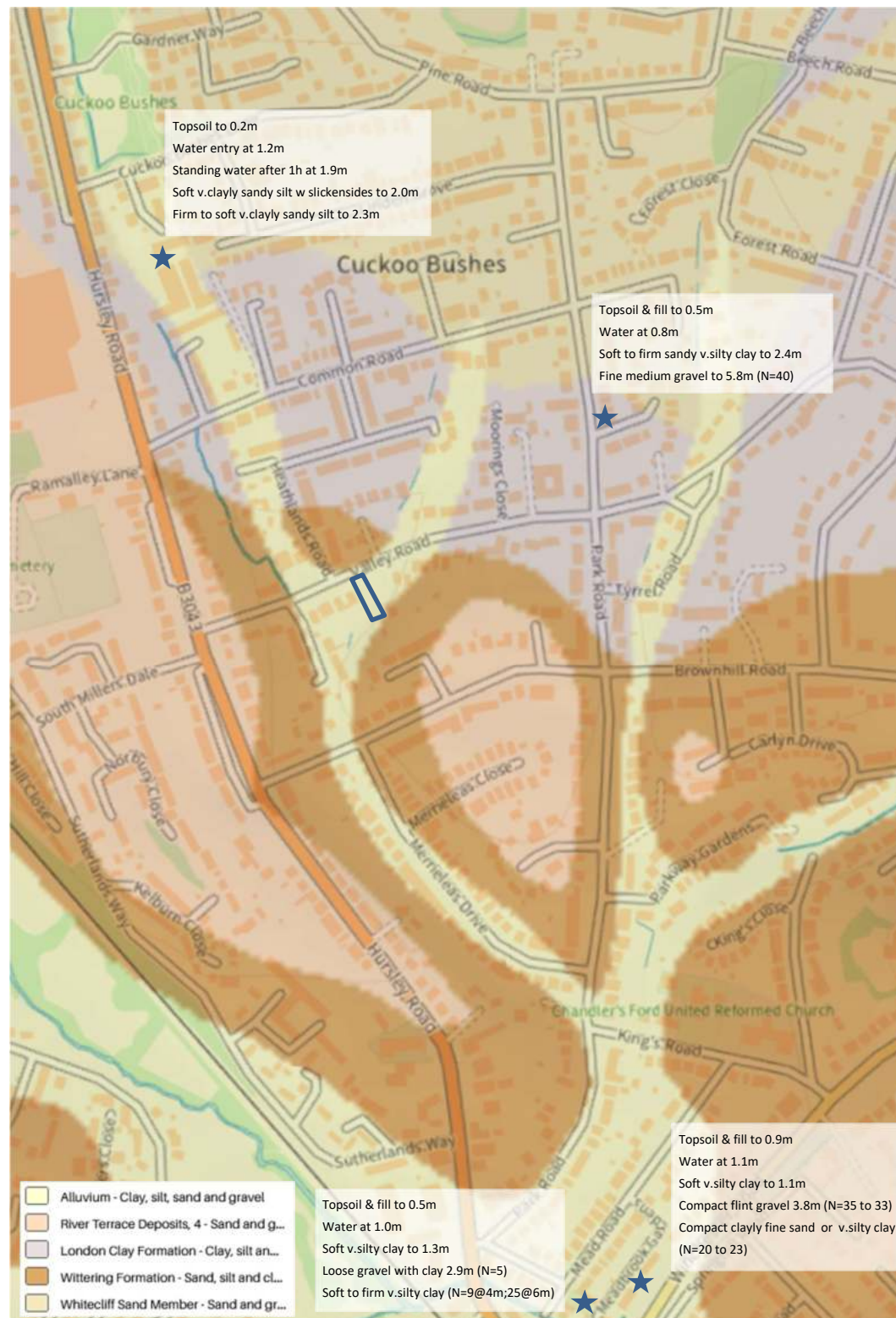


Figure 01: Extract from BGS Geology Viewer for the locality showing borehole info from similar sites

2.0 Design Aids

Design Codes

Eurocode 1: BS EN 1991-1-1:2002

Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings

Eurocode 2: BS EN 1992-1-1:2004

Design of concrete structures - Part 1-1: General rules and rules for buildings Eurocode 3: BS EN 1993-1-1:2005

Design of steel structures - Part 1-1: General rules and rules for buildings

Eurocode 5: BS EN 1995-1-1:2004 +A2:2014

Design of timber structures - Part 1-1: General – Common rules and rules for buildings

Eurocode 6: BS EN 1996-1-1:2005 +A1:2012

Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures

Literature

BGS Report : Determination of the shrinking and swelling properties of the London Clay Formation: Laboratory Report IR/06/058

3.0 Structural Loadings Schedule**Beam and Block Floor:**

		kN/m²
Dead:	Beam and block	2.15(1.96)
	Insulation/finishes	0.10
	Screed	1.80
	Partitions	0.50
	Total:	4.55(4.36)
Imposed:	Domestic	1.50
	Garage	2.50

Timber Floor:

		kN/m²
Dead:	Timber floor joists	0.15
	Insulation/finishes	0.10
	Floor boards/plywood	0.15
	Partitions	0.50
	Total:	0.90
Imposed:	Domestic	1.50

Cavity Wall - Plynth:

		kN/m²
Dead:	102mm brickwork	2.00
	Insulation	0.05
	100mm dense block	2.00
	Finishes	0.25
	Total:	4.45

Cavity Wall - Typical:

		kN/m²
Dead:	100mm lightweight block	0.90
	Insulation	0.05
	100mm lightweight block	0.90
	Finishes	0.25
	Total:	2.10

Internal Solid Masonry Wall:

		kN/m²
Dead:	140mm lightweight block	1.25
	Finishes	0.30
	Total:	1.55

Stability stud wall:

		kN/m²
Dead:	Timber studs	0.15
	Insulation	0.05
	Plywood sheathing	0.10
	Finishes	0.15
	Total:	0.45

Structural Calculations

New Pitched Roof:

		kN/m²
Dead:	Concrete roof tiles	0.55
	Membrane and battens	0.05
	Timber rafters and noggins	0.15
	Insulation	0.05
	12.5mm plasterboard and skim coat	0.15
	Vaulted roof services	0.15
	Total permanent =	1.10

Slope correction factor (40° pitch) = 1.31

Total dead on plan **1.44**

Imposed:	Domestic roof	0.60
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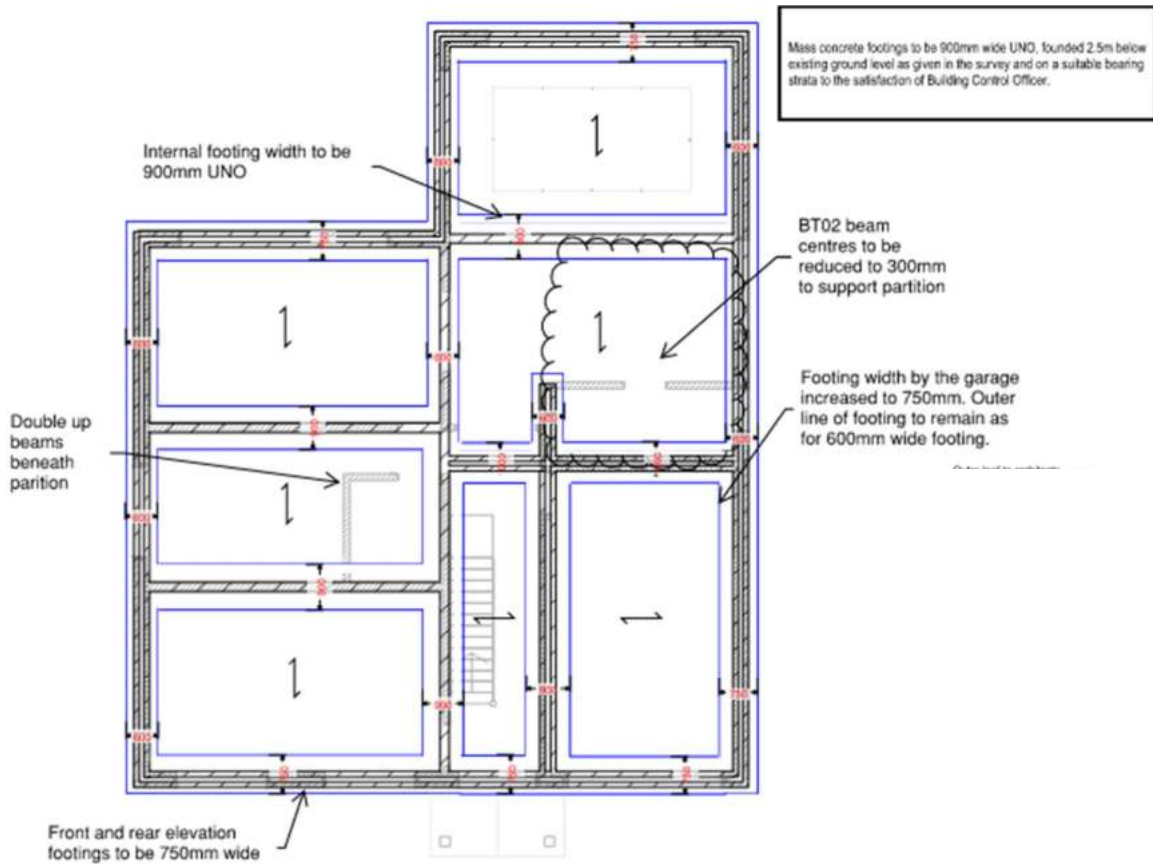
Flat Roof Construction

		kN/m²
Dead:	Roofing felt	0.15
	Plywood boarding	0.15
	Timber firrings	0.05
	Insulation	0.05
	Timber joists & noggins	0.15
	12.5mm plasterboard & skim coat	0.15
	Total permanent =	0.70

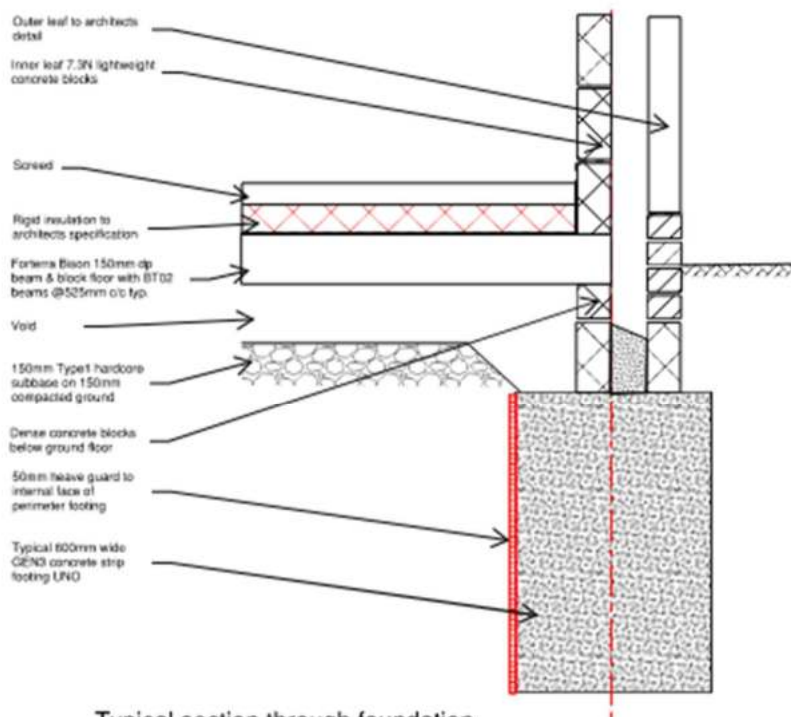
Imposed load	0.75
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4.0 Structural Arrangement

Refer to Thetis drawings 2230109-THE-ZZ-00-SK-S-0001 , 2230109-THE-ZZ-00-SK-S-0002 , 2230109-THE-ZZ-00-SK-S-0003 for details.

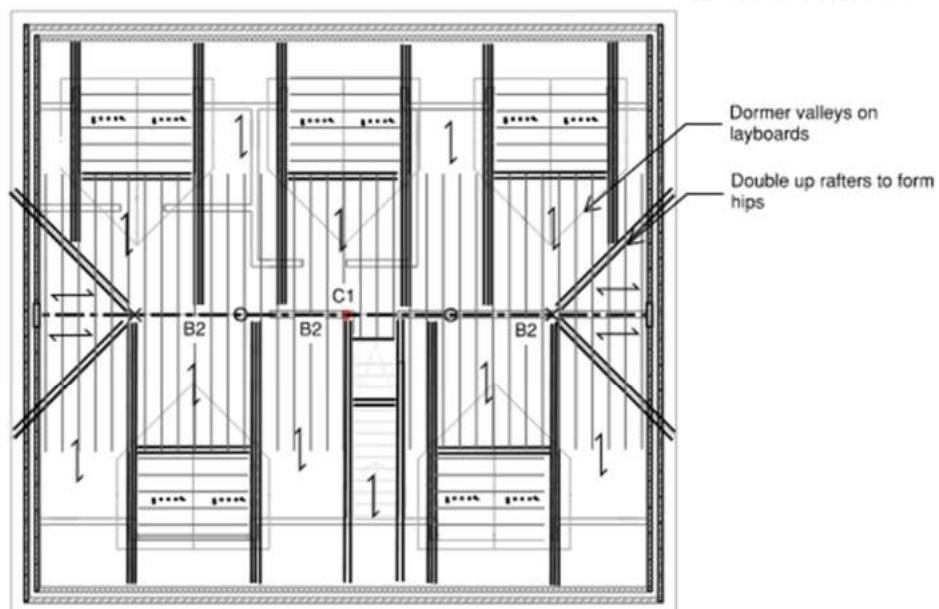
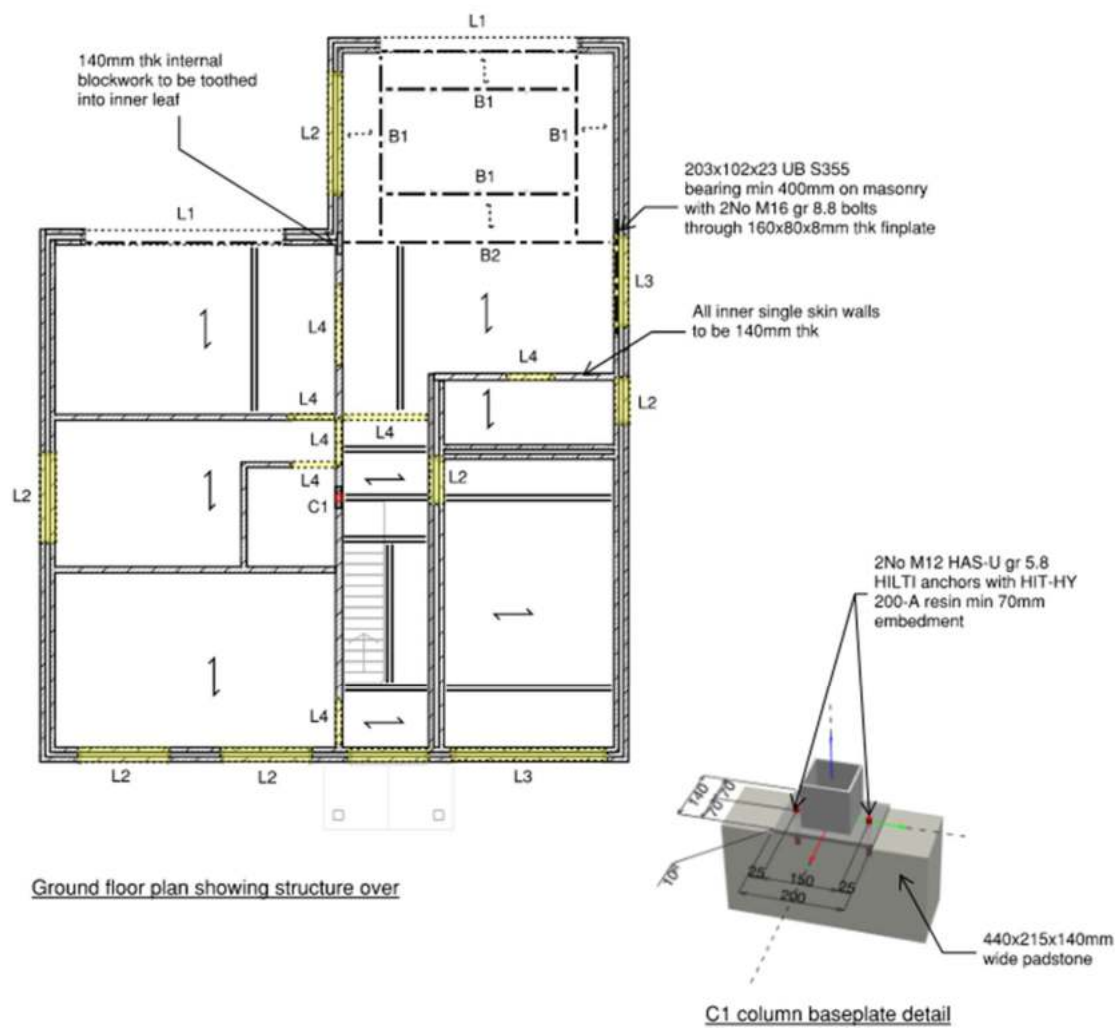


Foundation and ground floor structure general arrangement plan



Typical section through foundation

Structural Calculations



First floor plan showing roof structure over

5.0 Structural Calculations

Check Timber Rafters to Pitched Roof:

Timber rafters overall length is 6800mm. They are propped by the ceiling joints thus creating two spans $L_1=4100\text{mm}$ and $L_2=2700\text{mm}$; Spacing = 400mm

Load acting on rafters on plan:

$$\begin{aligned} G_k &= 0.4 \times 1.43 = 0.57 \text{ kN/m} \\ Q_k &= 0.4 \times 0.6 = 0.24 \text{ kN/m} \end{aligned}$$

TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedd's calculation version 1.0.06

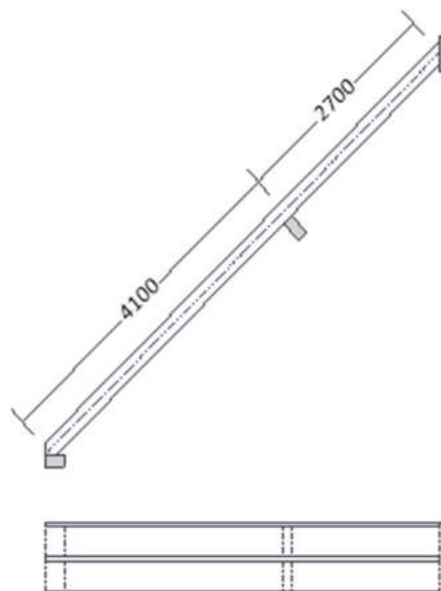
Rafter details

Rafter spacing

$s_{\text{Rafter}} = 400 \text{ mm}$

Rafter inclination

$\theta_{\text{Rafter}} = 43 \text{ deg}$



Forces input on Rafter

Permanent load on slope

$F_{G_Rafter} = 1.25 \text{ kN/m}^2$

Imposed load on plan

$F_{Q_Rafter} = 0.60 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope

$p_G = 0.50 \text{ kN/m}$

Imposed load on slope

$p_Q = 0.18 \text{ kN/m}$

Member span 1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	14.2	0.5	0.032	PASS
Bending stress	N/mm ²	16.2	3.5	0.213	PASS
Shear stress	N/mm ²	2.7	0.4	0.146	PASS
Bending and axial force				0.214	PASS
Column stability check				0.234	PASS
Deflection	mm	16.4	7.6	0.466	PASS

ANALYSIS

Tedd's calculation version 1.0.37

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + ψ_0 1.50Q (Strength)	1.35	1.05	0.00	0.00
ξ 1.35G + 1.50Q (Strength)	1.25	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.5 kN/m at 0 m to 6.8 m
Member	Imposed	UDL	GlobalZ	0.18 kN/m at 0 m to 6.8 m

Tedd's calculation version 2.2.17

Member - Span 1

Partial factor for material properties and resistances

Partial factor $\gamma_M = 1.300$

Member details

Load duration Medium-term Service class 2

Timber section details

Number of timber sections $N = 1$
 Breadth of sections $b = 47$ mm Depth of sections $h = 200$ mm
 Timber strength class C24



47x200 timber section
 Cross-sectional area, A , 9400 mm²
 Section modulus, W_y , 313333.3 mm³
 Section modulus, W_z , 73633 mm³
 Second moment of area, I_y , 31333333 mm⁴
 Second moment of area, I_z , 1730383 mm⁴
 Radius of gyration, i_y , 57.7 mm
 Radius of gyration, i_z , 13.6 mm
Timber strength class C24
 Characteristic bending strength, $f_{t,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0,5\%}$, 7400 N/mm²
 Shear modulus of elasticity, $G_{0,mean}$, 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 2 - ξ 1.35G + 1.50Q (Strength)

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d} = 0.458$ N/mm²

Design compressive strength $f_{c,0,d} = 14.215$ N/mm²

Utilisation = 0.032

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at end of span

Check shear force - Section 6.1.7

Design shear stress	$\tau_{y,d} = 0.394 \text{ N/mm}^2$	Design shear strength	$f_{v,y,d} = 2.708 \text{ N/mm}^2$
		Utilisation	= 0.146

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending stress	$\sigma_{m,y,d} = 3.468 \text{ N/mm}^2$	Design bending strength	$f_{m,y,d} = 16.246 \text{ N/mm}^2$
		Utilisation	= 0.213

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = 0.214

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis	$L_{e,y} = 3690 \text{ mm}$	Relative slenderness ratio	$\lambda_{rel,y} = 1.084$
Effective length for z-axis	$L_{e,z} = 0 \text{ mm}$	Relative slenderness ratio	$\lambda_{rel,z} = 0$
			$\lambda_{rel,y} > 0.3$ column stability check is required
		Utilisation	= 0.234

PASS - Column stability is acceptable

Check design 1854 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep	$\delta_{y,Final} = 7.6 \text{ mm}$	Allowable deflection	$\delta_{y,Allowable} = 16.4 \text{ mm}$
		Utilisation	= 0.466

PASS - Allowable deflection exceeds final deflection

ADOPT 200x47mm C24 TIMBER RAFTERS @ 400mm C/C

Doubled Up Timber Trimmer in Pitched Roof

Either side of the roof light 2 No 200x47 C24 timbers are proposed.
Loaded width per doubled up rafter $b = (900+400)/2 = 650\text{mm}$

Loaded width per rafter section $650/2 = 325\text{mm} < 400\text{mm}$ THEREFORE OK
By inspection doubled up rafters are sufficient.

ADOPT 2No 200x47mm C24 TIMBER RAFTERS EITHER SIDE OF ROOF LIGHT

BOLTED TOGETHER WITH M12 BOLTS @600mm C/C

Check Timber Ceiling Joists

Maximum span $L = 4.25\text{m}$
Loading on Ceiling: permanent 0.35 kN/m^2 imposed 0.25kN/m^2
Loading is much less than loading on roof, while the span of joist is comparable, therefore by inspection the same timber section is sufficient.

ADOPT 200x47mm C24 TIMBER CEILING JOISTS @ 400mm C/C

Check Timber Rafters to Dormer Windows**TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)**

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the recommended values

Tedds calculation version 1.0.06

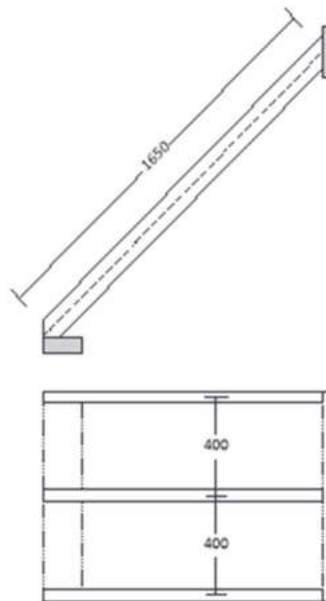
Rafter details

Rafter spacing

$s_{\text{Rafter}} = 400 \text{ mm}$

Rafter inclination

$\theta_{\text{Rafter}} = 43 \text{ deg}$

**Forces input on Rafter**

Permanent load on slope

$F_{G_Rafter} = 1.25 \text{ kN/m}^2$

Imposed load on plan

$F_{Q_Rafter} = 0.60 \text{ kN/m}^2$

Rafter loading details**Distributed loads**

Permanent load on slope

$p_G = 0.50 \text{ kN/m}$

Imposed load on slope

$p_Q = 0.18 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	14.2	0.2	0.015	PASS
Bending stress	N/mm ²	17.6	2.9	0.164	PASS
Shear stress	N/mm ²	2.7	0.3	0.096	PASS
Bending and axial force				0.164	PASS
Column stability check				0.174	PASS
Deflection	mm	6.6	2.8	0.428	PASS

Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	0.5 (max abs)	0.5	0 (min)	
	0.825	0		0.2 (max)	0.2
	1.65	-0.5	-0.5	0 (min)	

Tedd's calculation version 2.2.17

Member - Span 1

Partial factor for material properties and resistances

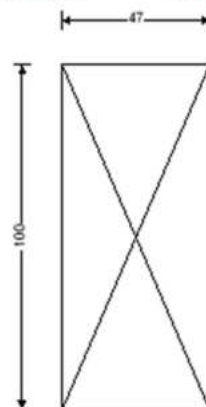
Partial factor $\gamma_M = 1.300$

Member details

Load duration Medium-term Service class 2

Timber section details

Number of timber sections $N = 1$
 Breadth of sections $b = 47 \text{ mm}$ Depth of sections $h = 100 \text{ mm}$
 Timber strength class **C24**



47x100 timber section

Cross-sectional area, A , 4700 mm²
 Section modulus, W_y , 78333.3 mm³
 Section modulus, W_x , 36817 mm³
 Second moment of area, I_y , 3916667 mm⁴
 Second moment of area, I_x , 866192 mm⁴
 Radius of gyration, i_y , 28.9 mm
 Radius of gyration, i_x , 13.6 mm
Timber strength class C24
 Characteristic bending strength, $f_{t,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0,5\%}$, 7400 N/mm²
 Shear modulus of elasticity, $G_{0,mean}$, 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, $\rho_{0,mean}$, 420 kg/m³

Span details

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

Consider Combination 2 - $\xi 1.35G + 1.50Q$ (Strength)

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d} = 0.217 \text{ N/mm}^2$ Design compressive strength $f_{c,0,d} = 14.215 \text{ N/mm}^2$
 Utilisation = 0.015

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d} = 0.261 \text{ N/mm}^2$ Design shear strength $f_{v,y,d} = 2.708 \text{ N/mm}^2$
 Utilisation = 0.096

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis $L_{e,y} = 1485$ mmRelative slenderness ratio $\lambda_{rel,y} = 0.872$ Effective length for z-axis $L_{e,z} = 0$ mmRelative slenderness ratio $\lambda_{rel,z} = 0$ $\lambda_{rel,y} > 0.3$ column stability check is required

Utilisation = 0.02

PASS - Column stability is acceptable**Check design 825 mm along span**

Check bending moment - Section 6.1.6

Design bending stress $\sigma_{m,y,d} = 2.884$ N/mm²Design bending strength $f_{m,y,d} = 17.618$ N/mm²

Utilisation = 0.164

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = 0.164

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis $L_{e,y} = 1485$ mmRelative slenderness ratio $\lambda_{rel,y} = 0.872$ Effective length for z-axis $L_{e,z} = 0$ mmRelative slenderness ratio $\lambda_{rel,z} = 0$ $\lambda_{rel,y} > 0.3$ column stability check is required

Utilisation = 0.174

PASS - Column stability is acceptable**Check design 825 mm along span**

Check y-y axis deflection - Section 7.2

Final deflection with creep $\delta_{y,Final} = 2.8$ mmAllowable deflection $\delta_{y,Allowable} = 6.6$ mm

Utilisation = 0.428

PASS - Allowable deflection exceeds final deflection**ADOPT 100x47mm C24 TIMBER RAFTERS @ 400mm C/C**

Check Timber Ceiling Joists to Dormer

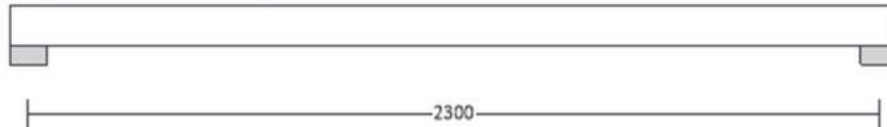
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the recommended values

Tedd's calculation version 1.0.08

Joist details

Joist spacing $s_{joist} = 400 \text{ mm}$



Forces input on Joist

Vertical permanent load on joist $F_{G_joist} = 0.30 \text{ kN/m}^2$

Vertical imposed load on joist $F_{Q_joist} = 0.25 \text{ kN/m}^2$

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = 0.12 \text{ kN/m}$

Vertical imposed load on joist $p_Q = 0.10 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.1	0.048	PASS
Bending stress	N/mm ²	17.6	2.8	0.160	PASS
Shear stress	N/mm ²	2.7	0.2	0.068	PASS
Deflection	mm	9.2	3.7	0.402	PASS

ANALYSIS

Tedd's calculation version 1.0.37

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.12 kN/m at 0 m to 2.3 m
Member	Imposed	UDL	GlobalZ	0.1 kN/m at 0 m to 2.3 m

Tedd's calculation version 2.2.17

Member - Span 1

Partial factor for material properties and resistances

Partial factor $\gamma_M = 1.300$

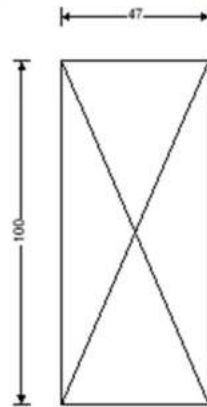
Structural Calculations

Member details

Load duration Medium-term Service class 2

Timber section details

Number of timber sections $N = 1$
 Breadth of sections $b = 47 \text{ mm}$ Depth of sections $h = 100 \text{ mm}$
 Timber strength class C24

**47x100 timber section**

Cross-sectional area, A , 4700 mm²
 Section modulus, W_x , 78333.3 mm³
 Section modulus, W_y , 36817 mm³
 Second moment of area, I_x , 3916657 mm⁴
 Second moment of area, I_y , 865192 mm⁴
 Radius of gyration, i_x , 28.9 mm
 Radius of gyration, i_y , 13.6 mm
Timber strength class C24
 Characteristic bending strength, $f_{t,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²
 Shear modulus of elasticity, $G_{0.05}$, 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, $\rho_{0.05}$, 420 kg/m³

Span details

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

Consider Combination 1 - 1.35G + 1.50Q (Strength)**Check design at start of span****Check compression perpendicular to the grain - cl.6.1.5**

Des.perp.comp.stress $\sigma_{c,y,90,d} = 0.082 \text{ N/mm}^2$ Des.perp.comp.strength $f_{c,y,90,d} = 1.692 \text{ N/mm}^2$
 Utilisation = 0.048

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d} = 0.183 \text{ N/mm}^2$ Design shear strength $f_{v,y,d} = 2.708 \text{ N/mm}^2$
 Utilisation = 0.068

PASS - Design shear strength exceeds design shear stress

Check design 1150 mm along span**Check bending moment - Section 6.1.6**

Design bending stress $\sigma_{m,y,d} = 2.818 \text{ N/mm}^2$ Design bending strength $f_{m,y,d} = 17.618 \text{ N/mm}^2$
 Utilisation = 0.16

PASS - Design bending strength exceeds design bending stress

Consider Combination 2 - 1.00G + 1.00Q (Service)**Check design 1150 mm along span****Check y-y axis deflection - Section 7.2**

Final deflection with creep $\delta_{y,Final} = 3.7 \text{ mm}$ Allowable deflection $\delta_{y,Allowable} = 9.2 \text{ mm}$
 Utilisation = 0.402

PASS - Allowable deflection exceeds final deflection

ADOPT 100x47mm C24 TIMBER DORMER CEILING JOISTS @ 400mm C/C

Structural Calculations

Check Timber Joists to Flat Roof:

Span = 1.7m

Spacing = 600mm

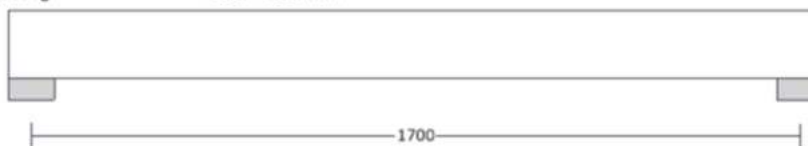
Load acting on rafters:

$$\begin{aligned}
 G_k &= 0.4 \times 0.70 = 0.28 \text{ kN/m} \\
 Q_k &= 0.4 \times 0.75 = 0.3 \text{ kN/m}
 \end{aligned}$$

TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.06

Joist detailsJoist spacing $S_{\text{joist}} = 600 \text{ mm}$ **Forces input on Joist**Vertical permanent load on joist $F_{G_{\text{joist}}} = 0.75 \text{ kN/m}^2$ Vertical imposed load on joist $F_{Q_{\text{joist}}} = 0.75 \text{ kN/m}^2$ **Joist loading details****Distributed loads**Vertical permanent load on joist $p_G = 0.45 \text{ kN/m}$ Vertical imposed load on joist $p_Q = 0.45 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.2	0.141	PASS
Bending stress	N/mm ²	16.2	2.7	0.166	PASS
Shear stress	N/mm ²	2.7	0.4	0.131	PASS
Beam stability check				0.166	PASS
Deflection	mm	6.8	1.4	0.205	PASS

Tedds calculation version 2.2.17

Member - Span 1**Partial factor for material properties and resistances**Partial factor $\gamma_M = 1.300$ **Member details**

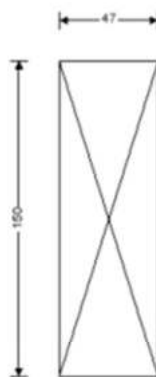
Load duration Medium-term Service class 2

Timber section details

Number of timber sections $N = 1$

Breadth of sections $b = 47 \text{ mm}$ Depth of sections $h = 150 \text{ mm}$

Timber strength class **C24**



47x150 timber section
 Cross-sectional area, A , 7050 mm²
 Section modulus, W_y , 176250 mm³
 Section modulus, W_x , 55225 mm³
 Second moment of area, I_y , 13218750 mm⁴
 Second moment of area, I_x , 1297787 mm⁴
 Radius of gyration, i_y , 43.3 mm
 Radius of gyration, i_x , 13.6 mm
Timber strength class C24
 Characteristic bending strength, $f_{m,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{t,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, E_{mean} , 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²
 Shear modulus of elasticity, G_{mean} , 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des. perp. comp. stress $\sigma_{c,y,90,d} = 0.238$ N/mm² Des. perp. comp. strength $f_{c,y,90,d} = 1.692$ N/mm²
 Utilisation = 0.141

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d} = 0.355$ N/mm² Design shear strength $f_{v,y,d} = 2.708$ N/mm²
 Utilisation = 0.131

PASS - Design shear strength exceeds design shear stress

Check design 850 mm along span

Check bending moment - Section 6.1.6

Design bending stress $\sigma_{m,y,d} = 2.696$ N/mm² Design bending strength $f_{m,y,d} = 16.246$ N/mm²
 Utilisation = 0.166

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = 0.166

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 850 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep $\delta_{y,Final} = 1.4$ mm Allowable deflection $\delta_{y,Allowable} = 6.8$ mm
 Utilisation = 0.205

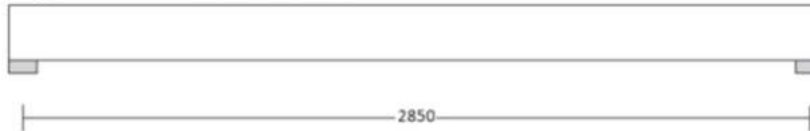
PASS - Allowable deflection exceeds final deflection

ADOPT 150x47mm C24 TIMBER JOISTS @ 600mm C/C

Timber Floor Joists**TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)**

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.06

Joist detailsJoist spacing $S_{Joist} = 400 \text{ mm}$ **Forces input on Joist**

Vertical permanent load on joist

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

Vertical imposed load on joist

 $F_{O_Joist} = 1.50 \text{ kN/m}^2$ **Joist loading details****Distributed loads**

Vertical permanent load on joist

 $p_O = 0.30 \text{ kN/m}$

Vertical imposed load on joist

 $p_O = 0.60 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.4	0.242	PASS
Bending stress	N/mm ²	16.2	4.4	0.269	PASS
Shear stress	N/mm ²	2.7	0.5	0.169	PASS
Beam stability check				0.269	PASS
Deflection	mm	11.4	4.5	0.395	PASS

Tedds calculation version 2.2.17

Member - Span 1**Partial factor for material properties and resistances**Partial factor $\gamma_M = 1.300$ **Member details**

Load duration Medium-term Service class 2

Timber section details

Number of timber sections $N = 1$
 Breadth of sections $b = 47 \text{ mm}$ Depth of sections $h = 200 \text{ mm}$
 Timber strength class **C24**

**47x200 timber section**Cross-sectional area, $A, 9400 \text{ mm}^2$ Section modulus, $W_y, 313333.3 \text{ mm}^3$ Section modulus, $W_x, 73633 \text{ mm}^3$ Second moment of area, $I_y, 31333333 \text{ mm}^4$ Second moment of area, $I_x, 1730383 \text{ mm}^4$ Radius of gyration, $i_y, 57.7 \text{ mm}$ Radius of gyration, $i_x, 13.6 \text{ mm}$ **Timber strength class C24**Characteristic bending strength, $f_{t,k}, 24 \text{ N/mm}^2$ Characteristic shear strength, $f_{v,k}, 4 \text{ N/mm}^2$ Characteristic compression strength parallel to grain, $f_{c,k}, 21 \text{ N/mm}^2$ Characteristic compression strength perpendicular to grain, $f_{c90,k}, 2.5 \text{ N/mm}^2$ Characteristic tension strength parallel to grain, $f_{t,k}, 14.5 \text{ N/mm}^2$ Mean modulus of elasticity, $E_{0,mean}, 11000 \text{ N/mm}^2$ Fifth percentile modulus of elasticity, $E_{0.05}, 7400 \text{ N/mm}^2$ Shear modulus of elasticity, $G_{mean}, 690 \text{ N/mm}^2$ Characteristic density, $\rho_k, 350 \text{ kg/m}^3$ Mean density, $\rho_{mean}, 420 \text{ kg/m}^3$ **Span details**Bearing length $L_b = 100 \text{ mm}$

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress	$\sigma_{c,y,90,d} = 0.409 \text{ N/mm}^2$	Des.perp.comp.strength	$f_{c,y,90,d} = 1.692 \text{ N/mm}^2$
		Utilisation =	0.242

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress	$\tau_{y,d} = 0.458 \text{ N/mm}^2$	Design shear strength	$f_{v,y,d} = 2.708 \text{ N/mm}^2$
		Utilisation =	0.169

PASS - Design shear strength exceeds design shear stress

Check design 1425 mm along span

Check bending moment - Section 6.1.6

Design bending stress	$\sigma_{m,y,d} = 4.37 \text{ N/mm}^2$	Design bending strength	$f_{m,y,d} = 16.246 \text{ N/mm}^2$
		Utilisation =	0.269

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = 0.269

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1425 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep	$\delta_{y,Final} = 4.5 \text{ mm}$	Allowable deflection	$\delta_{y,Allowable} = 11.4 \text{ mm}$
		Utilisation =	0.395

PASS - Allowable deflection exceeds final deflection

ADOPT 200x47mm C24 TIMBER FLOOR JOISTS @ 400mm C/C

DOUBLE UP FLOOR JOISTS UNDER PARTITION 2No 200x47mm C24 BOLTED TOGETHER WITH M12 BOLTS @600mm C/C

Steelwork design**Steel cranked ridge Beam supported on SHS column**

Span = 7.4 m

Load acting on beam:

Width of roof supported = $4.7/2 = 2.35$ m

$$G_k = 2.35 \times 1.58 = 3.71 \text{ kN/m}$$

$$Q_k = 2.35 \times 0.6 = 1.41 \text{ kN/m}$$

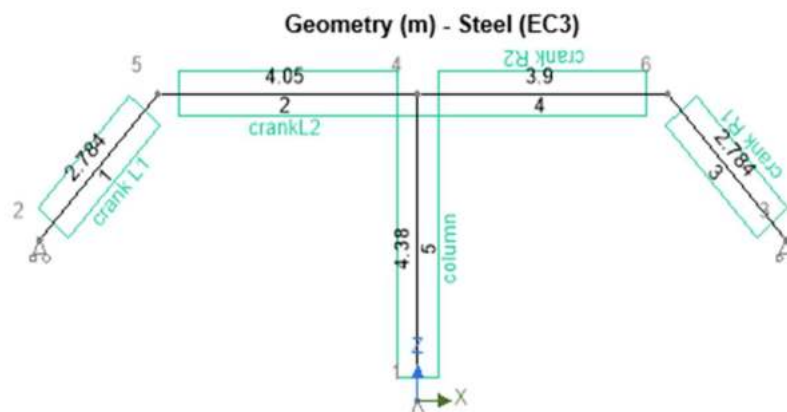
STEEL MEMBER ANALYSIS & DESIGN (EN1993-1-1:2005)

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

Tedds calculation version 4.4.10

ANALYSIS

Tedds calculation version 1.0.37

Geometry**Materials**

Name	Density (kg/m ³)	Youngs Modulus kN/mm ²	Shear Modulus kN/mm ²	Thermal Coefficient °C ⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm ²)	Moment of inertia		Shear area parallel to	
		Major (cm ⁴)	Minor (cm ⁴)	Minor (cm ²)	Major (cm ²)
UB 254x146x37	47.2	5536.8	570.6	16.1	28.7
SHS 100x100x5.0	18.7	279.4	279.4	9.4	9.4

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. kNm/°
1	0	0	Fixed	Fixed	Free		0	0	0	0
2	-5.9	2.3	Free	Fixed	Free		0	0	0	0
3	5.75	2.3	Free	Fixed	Free		0	0	0	0
4	0	4.38	Free	Free	Free		0	0	0	0
5	-4.05	4.38	Free	Free	Free		0	0	0	0
6	3.9	4.38	Free	Free	Free		0	0	0	0

Elements

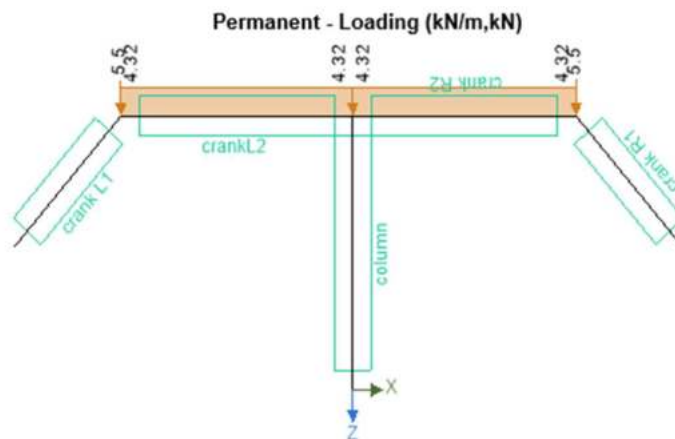
Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	2.784	2	5	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
2	4.05	5	4	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
3	2.784	3	6	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
4	3.9	6	4	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
5	4.38	1	4	SHS 100x100x5.0	Steel (EC3)	Fixed	Fixed	Fixed	

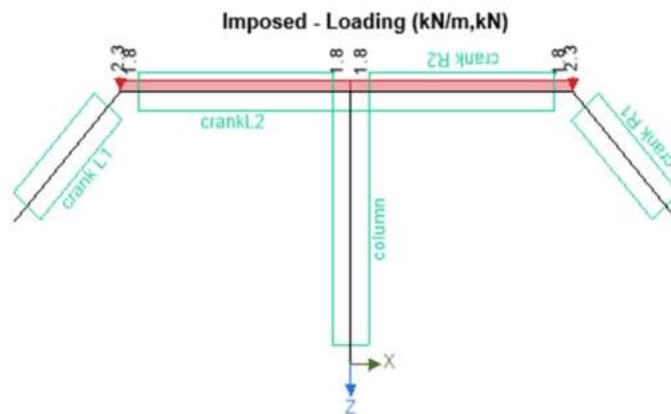
Members

Name	Elements	
	Start	End
crank L1	1	1
crank R1	3	3
column	5	5
crankL2	2	2
crank R2	4	4

Loading

Self weight included

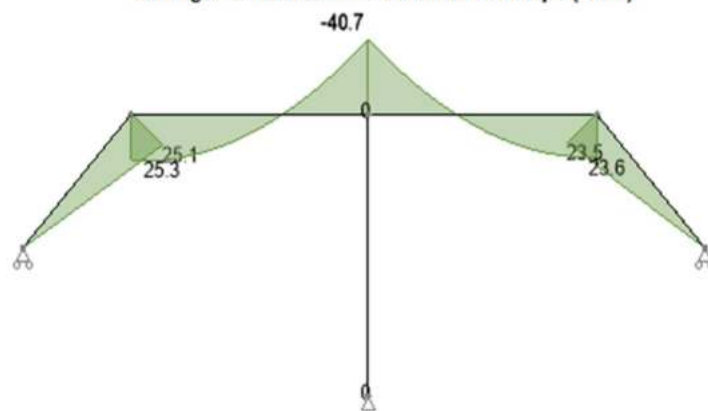
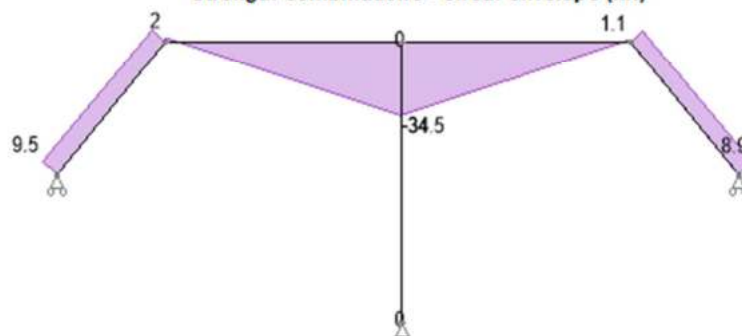
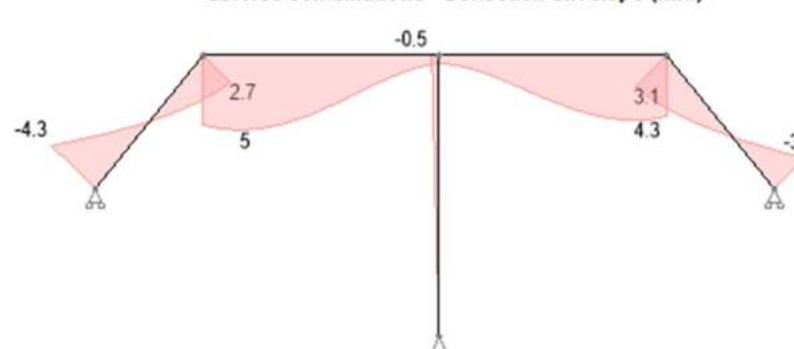


**Load combination factors**

Load combination	Self Weight	Permanent	Imposed
combination 1 (Strength)	1.35	1.35	1.50
SLS (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
crankL2	Permanent	UDL	GlobalZ	4.32 kN/m
crank R2	Permanent	UDL	GlobalZ	4.32 kN/m
crank L1	Permanent	Point load	GlobalZ	5.5 kN at 2.784 m
crank R1	Permanent	Point load	GlobalZ	5.5 kN at 2.784 m
crankL2	Imposed	UDL	GlobalZ	1.8 kN/m
crank R2	Imposed	UDL	GlobalZ	1.8 kN/m
crank L1	Imposed	Point load	GlobalZ	2.3 kN at 2.784 m
crank R1	Imposed	Point load	GlobalZ	2.3 kN at 2.784 m

Results**Forces****Strength combinations - Moment envelope (kNm)****Strength combinations - Shear envelope (kN)****Service combinations - Deflection envelope (mm)**Partial factors - Section 6.1 $\gamma_{M0} = 1$ $\gamma_{M1} = 1$ $\gamma_{M2} = 1.1$

crank L1 design**Section details**

Steel grade

Nominal yield strength

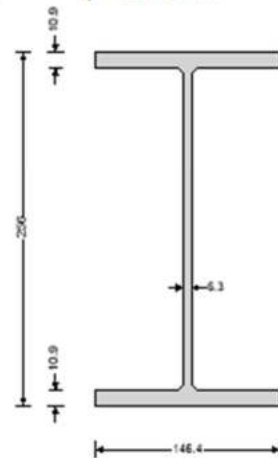
UB 254x146x37 (BS4-1)

S355

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

Modulus of elasticity

Nominal ult.tensile strength

 $E = 210000 \text{ N/mm}^2$ $f_u = 470 \text{ N/mm}^2$ 

UB 254x146x37 (BS4-1)
 Section depth, h , 256 mm
 Section breadth, b , 146.4 mm
 Mass of section, M , 37 kg/m
 Flange thickness, t_f , 10.9 mm
 Web thickness, t_w , 6.3 mm
 Root radius, r , 7.6 mm
 Area of section, A , 4717 mm²
 Radius of gyration about y-axis, i_y , 108.347 mm
 Radius of gyration about z-axis, i_z , 34.783 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 432562 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 77957 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 463231 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 119374 mm³
 Second moment of area about y-axis, I_y , 55367902 mm⁴
 Second moment of area about z-axis, I_z , 5706456 mm⁴

crank L1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Shear resistance (y-y)	kN	360.6	9.5	0.026	PASS
Bending resistance (y-y)	kNm	171.3	25.1	0.147	PASS
Compression resistance	kN	949.8	10.7	0.011	PASS
Comb. bending and axial force				0.175	PASS
Deflection (y-y)	mm	7.7	4.3	0.558	PASS

crank R1 design**Section details**

Steel grade

Nominal yield strength

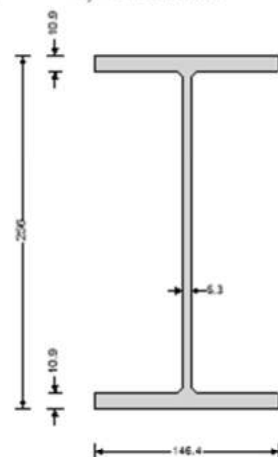
UB 254x146x37 (BS4-1)

S355

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

Modulus of elasticity

Nominal ult.tensile strength

 $E = 210000 \text{ N/mm}^2$ $f_u = 470 \text{ N/mm}^2$ 

UB 254x146x37 (BS4-1)
 Section depth, h , 256 mm
 Section breadth, b , 146.4 mm
 Mass of section, M , 37 kg/m
 Flange thickness, t_f , 10.9 mm
 Web thickness, t_w , 6.3 mm
 Root radius, r , 7.6 mm
 Area of section, A , 4717 mm²
 Radius of gyration about y-axis, i_y , 108.347 mm
 Radius of gyration about z-axis, i_z , 34.783 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 432562 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 77957 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 463231 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 119374 mm³
 Second moment of area about y-axis, I_y , 55367902 mm⁴
 Second moment of area about z-axis, I_z , 5706456 mm⁴

crank R1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Shear resistance (y-y)	kN	360.6	8.9	0.025	PASS
Bending resistance (y-y)	kNm	171.3	23.5	0.137	PASS
Compression resistance	kN	949.8	10.0	0.011	PASS
Comb. bending and axial force				0.163	PASS
Deflection (y-y)	mm	7.7	3.1	0.407	PASS

column design**Section details**

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

Steel grade

S355H

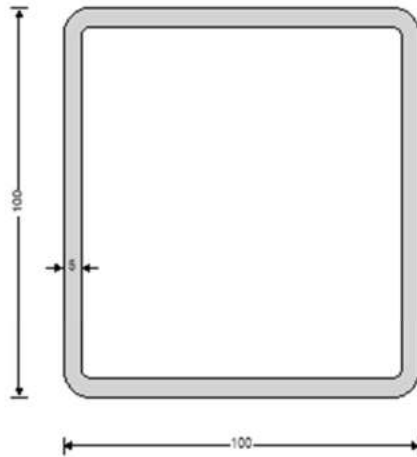
Modulus of elasticity

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

Nominal yield strength

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

Nominal ult.tensile strength

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

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

Section depth, h , 100 mmSection breadth, b , 100 mmMass of section, M , 14.7 kg/mSection thickness, t , 5 mmArea of section, A , 1873 mm²Radius of gyration about y-axis, i_y , 38.623 mmRadius of gyration about z-axis, i_z , 38.623 mmElastic section modulus about y-axis, $W_{el,y}$, 55886 mm³Elastic section modulus about z-axis, $W_{el,z}$, 55886 mm³Plastic section modulus about y-axis, $W_{pl,y}$, 66358 mm³Plastic section modulus about z-axis, $W_{pl,z}$, 66358 mm³Second moment of area about y-axis, I_y , 2794323 mm⁴Second moment of area about z-axis, I_z , 2794323 mm⁴

column results summary	Unit	Capacity	Maximum	Utilisation	Result
Compression resistance	kN	252.2	69.4	0.275	PASS
Deflection (y-y)	mm	12.2	0.5	0.045	PASS

crankL2 design**Section details**

UB 254x146x37 (BS4-1)

Steel grade

S355

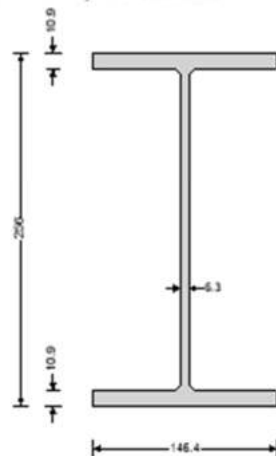
Modulus of elasticity

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

Nominal yield strength

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

Nominal ult.tensile strength

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

UB 254x146x37 (BS4-1)

Section depth, h , 256 mmSection breadth, b , 146.4 mmMass of section, M , 37 kg/mFlange thickness, t_f , 10.9 mmWeb thickness, t_w , 6.3 mmRoot radius, r , 7.6 mmArea of section, A , 4717 mm²Radius of gyration about y-axis, i_y , 108.347 mmRadius of gyration about z-axis, i_z , 34.783 mmElastic section modulus about y-axis, $W_{el,y}$, 432562 mm³Elastic section modulus about z-axis, $W_{el,z}$, 77957 mm³Plastic section modulus about y-axis, $W_{pl,y}$, 483231 mm³Plastic section modulus about z-axis, $W_{pl,z}$, 119374 mm³Second moment of area about y-axis, I_y , 55367902 mm⁴Second moment of area about z-axis, I_z , 5706456 mm⁴

crankL2 results summary	Unit	Capacity	Maximum	Utilisation	Result
Shear resistance (y-y)	kN	360.6	34.5	0.096	PASS
Bending resistance (y-y)	kNm	171.5	40.7	0.237	PASS
Deflection (y-y)	mm	11.3	5.0	0.445	PASS

crank R2 design**Section details**

Steel grade

Nominal yield strength

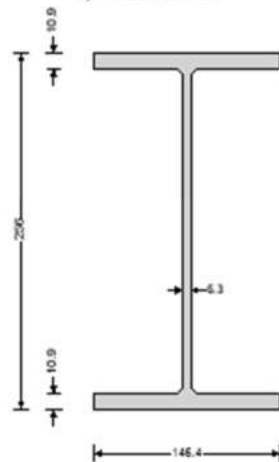
UB 254x146x37 (BS4-1)

S355

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

Modulus of elasticity

Nominal ult.tensile strength

 $E = 210000 \text{ N/mm}^2$ $f_u = 470 \text{ N/mm}^2$ 

UB 254x146x37 (BS4-1)
 Section depth, h , 256 mm
 Section breadth, b , 146.4 mm
 Mass of section, Mass, 37 kg/m
 Flange thickness, t_f , 10.9 mm
 Web thickness, t_w , 6.3 mm
 Root radius, r , 7.6 mm
 Area of section, A , 4717 mm²
 Radius of gyration about y-axis, i_y , 108.347 mm
 Radius of gyration about z-axis, i_z , 34.783 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 432562 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 77957 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 483231 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 119374 mm³
 Second moment of area about y-axis, I_y , 55367902 mm⁴
 Second moment of area about z-axis, I_z , 5705456 mm⁴

crank R2 results summary	Unit	Capacity	Maximum	Utilisation	Result
Shear resistance (y-y)	kN	360.6	34.0	0.094	PASS
Bending resistance (y-y)	kNm	171.5	40.7	0.237	PASS
Deflection (y-y)	mm	10.8	4.3	0.400	PASS

ADOPT 254x146x37 UB S355 STEEL CRANKED RIDGE BEAM AND 100x100x5.0 SHS COLUMN

CRANKED RIDGE BEAM TO BE SPLICED 2m EITHER SIDE OF THE COLUMN

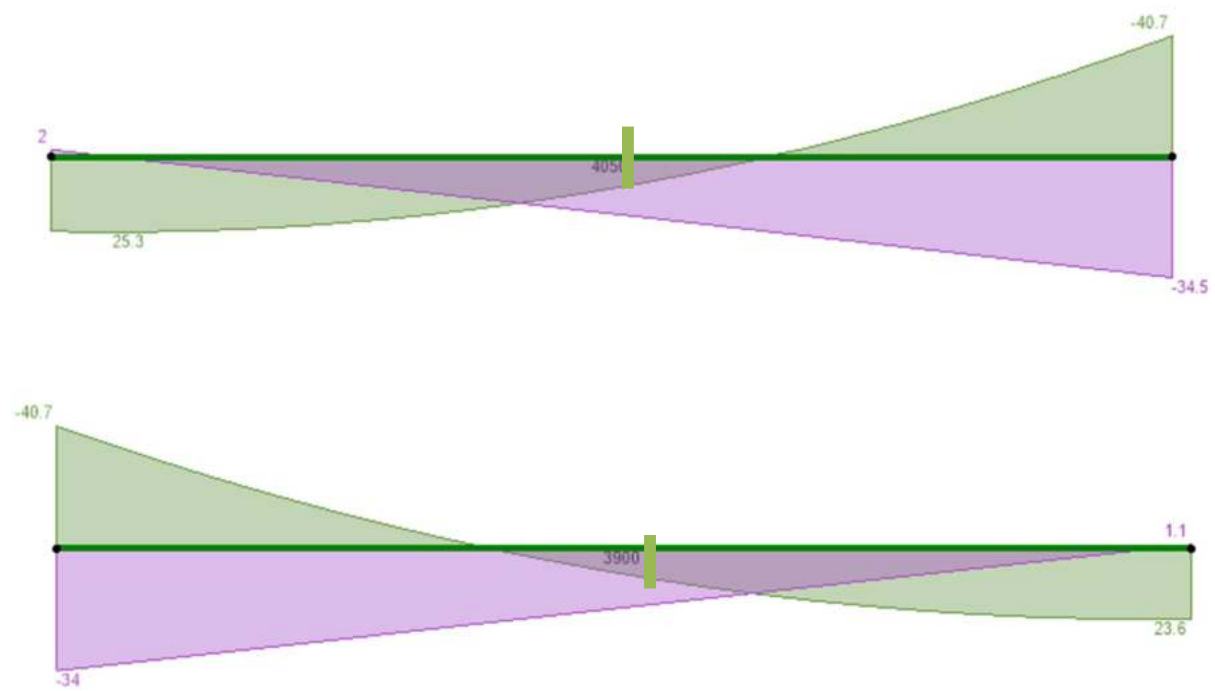
Splice design

The splice is designed 2m away from the column location to avoid maximum bending moment.

ULS forces in the location of the splice are taken as follows:

$$M=11.5 \text{ kNm}$$

$$V=21.5 \text{ kN}$$



STEEL BEAM SPLICE DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

Tedd's calculation version 1.0.00

Design summaryOverall design status **PASS**Overall design utilisation **0.521**

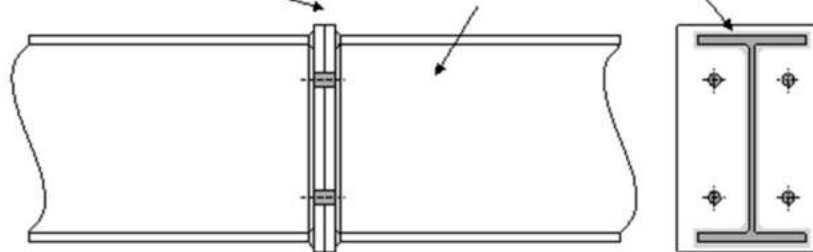
Description	Unit	Design	Resistance	Utilisation	Result
Moment resistance joint	kNm	12.0	34.4	0.349	PASS
Shear resist. bolt group	kN	22.0	154.3	0.143	PASS
Bearing resist. bolt group	kN	22.0	356.9	0.062	PASS
Tension flange weld	kN	175.8	337.1	0.521	PASS
Web weld tension zone	kN/mm	0.72	2.19	0.330	PASS
Web weld shear zone	kN/mm	0.22	1.79	0.122	PASS
Comp. flange weld	mm	6.0	6.0	1.000	PASS

2No. M16 bolts in tension zone

2No. M16 bolts in compression zone

End plate
200 x 280

Beam sections:
UB 254x146x37

**End plate Splice****Design forces**Design moment $M_{Ed} = 12.0$ kNmDesign axial force $N_{Ed} = 0.0$ kN

Design shear force

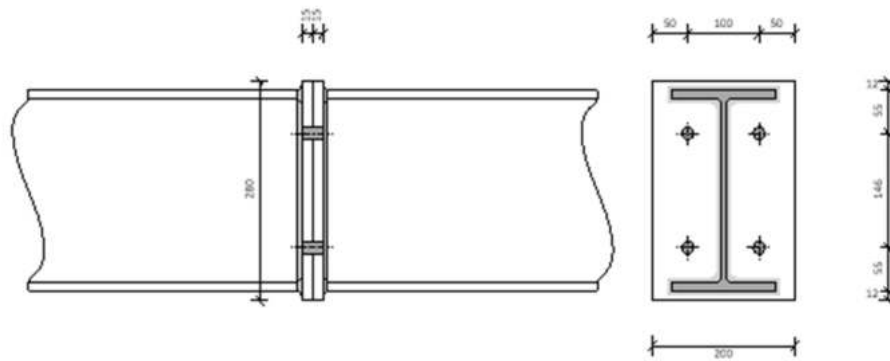
 $V_{Ed} = 22.0$ kN**Partial factors**Resistance of cross section $\gamma_{M0} = 1.00$ Sections in tension to fracture $\gamma_{M2} = 1.10$ Slip resistance $\gamma_{M3} = 1.25$ Structural integrity $\gamma_{M,u} = 1.10$ Resist. members to instability $\gamma_{M1} = 1.00$ Resistance of bolts $\gamma_{M2,b} = 1.25$ Slip resistance, servability $\gamma_{M3,ser} = 1.10$ **Beam details**

Section name UB 254x146x37

Yield strength $f_{y,bm} = 355$ N/mm²Section height $h = 256.0$ mmFlange thickness $t_f = 10.9$ mm

Steel grade S355

Ultimate strength $f_{u,bm} = 470$ N/mm²Section width $b = 146.4$ mmWeb thickness $t_w = 6.3$ mm

**End plate bolt details**

Number of bolt columns	$n_{1,ep} = 2$	Number of lower bolt rows	$n_{2,ep} = 1$
Number of upper bolt rows	$n_{2,ep} = 1$	Bolt pitch	$p_{2,ep} = 100 \text{ mm}$
Bolt gauge	$p_{1,ep} = 100 \text{ mm}$	Bolt grade	8.8
Bolt size	M16	Ultimate strength	$f_{ub,ep} = 800 \text{ N/mm}^2$
Yield strength	$f_{yb,ep} = 640 \text{ N/mm}^2$		
Edge distance in plate	$e_{1,ep} = 50 \text{ mm}$		
Dist. int. bolt to face of flange	$x_{2l,ep} = 55 \text{ mm}$		
Design tension resist. of bolt	$F_{t,Rd} = 90.4 \text{ kN}$		

End plate details

Plate width	$b_{ep} = 200 \text{ mm}$	Plate height	$h_{ep} = 280 \text{ mm}$
Plate thickness	$t_{ep} = 15 \text{ mm}$	Plate grade	S275
Yield strength	$f_{y,ep} = 275 \text{ N/mm}^2$	Ultimate strength	$f_{u,ep} = 410 \text{ N/mm}^2$

End plate bending resistance**Variables used for effective length (Figure 6.10)**

$m_x = 35.2 \text{ mm}$	$m_z = 39.3 \text{ mm}$
$w = 100 \text{ mm}$	$p = 100 \text{ mm}$
$e = 50 \text{ mm}$	$e_x = 50 \text{ mm}$
Quarter of bolt washer dia.	$e_w = 7 \text{ mm}$

Bolt row 1 - other end tension flange of beam

Design resist. T-stub - mode 1	$F_{T,1,Rd,r1} = 382.9 \text{ kN}$	Design resist. T-stub - mode 2	$F_{T,2,Rd,r1} = 175.8 \text{ kN}$
Design resist. T-stub - mode 3	$F_{T,3,Rd,r1} = 180.9 \text{ kN}$	Resist of end plate to bending	$F_{L,ep,Rd,r1} = 175.8 \text{ kN}$
Resist of beam web in tension	$F_{L,wb,Rd,r1} = 516.0 \text{ kN}$		

Resistance of bolt row 1

Resistance of bolt row	$F_{t,Rd,r1} = 175.8 \text{ kN}$
------------------------	----------------------------------

Minimum resistance of bolt rows

Effective resist. of bolt row 1	$F_{L,Rd,eff,r1} = 175.8 \text{ kN}$
---------------------------------	--------------------------------------

Design resistance of beam flange and adjacent compression zone of web

Design bending resistance $m_{nt} M_{c,y,Rd}$	$= 171.5 \text{ kNm}$	Comp zone resist of flng & wb	$F_{c,fb,Rd} = 699.9 \text{ kN}$
---	-----------------------	-------------------------------	----------------------------------

Lever arm of bolts from mid thickness of compression flange**Plastic distribution limit**

Design tension resistance limit	$F_{L,Rd,lim} = 171.8 \text{ kN}$
---------------------------------	-----------------------------------

As only one internal tension bolt is present no reduction to the bolt row tension resistance is required

Total tension resistance of bolt rowsTotal resistance of bolt rows $\Sigma F_{LRd} = 175.8 \text{ kN}$ **Equilibrium of forces**For horizontal equilibrium $\Sigma F_{LRd} + N_{Ed}$ cannot exceed $F_{c,b,Rd}$ $\Sigma F_{LRd} + N_{Ed} \leq F_{c,b,Rd}$ - no reduction in any row resistances is required**Moment resistance of joint**Moment resistance of joint $M_{j,Rd} = 34.4 \text{ kNm}$
 $M_{Ed} / M_{j,Rd} = 0.349$ *PASS - Moment resistance of joint greater than applied moment***Shear resistance of joint**Shear resistance of joint $F_{v,sp,Rd} = 154.3 \text{ kN}$
 $V_{Ed} / F_{v,sp,Rd} = 0.143$ *PASS - Shear resistance of joint greater than applied shear***Bearing resistance of bolts**Bearing resistance of bolts $F_{b,sp,Rd} = 356.9 \text{ kN}$
 $V_{Ed} / F_{b,sp,Rd} = 0.062$ *PASS - Bearing resistance of bolts greater than applied shear***Tension flange weld**Design force - local bolt forces $F_{fl,w,Ed} = 175.8 \text{ kN}$ Design resistance of weld $F_{fl,w,Rd} = 337.1 \text{ kN}$
 $F_{fl,w,Ed} / F_{fl,w,Rd} = 0.521$ *PASS - Design resistance of weld greater or equal than design force due to local bolt forces***Web weld in tension zone**Tension force per unit length $F_{T,ww,Ed} = 0.722 \text{ kN/mm}$ Design resistance of weld $F_{T,ww,Rd} = 2.187 \text{ kN/mm}$
 $F_{T,ww,Ed} / F_{T,ww,Rd} = 0.330$ *PASS - Design resistance of weld greater or equal than tension force in weld***Web weld in shear zone**Tension force per unit length $F_{L,ww,Ed} = 0.217 \text{ kN/mm}$ Design resistance of weld $F_{L,ww,Rd} = 1.785 \text{ kN/mm}$
 $F_{L,ww,Ed} / F_{L,ww,Rd} = 0.122$ *PASS - Design resistance of weld greater or equal than tension force in weld***Compression flange weld - beam end is sawn with a bearing fit so design for nominal welds**Leg length of weld $s_{l,jap} = 6.00 \text{ mm}$ Leg length of weld required $s_{l,jap,nom} = 6.00 \text{ mm}$ *PASS - Weld greater or equal to nominal weld size*

Baseplate

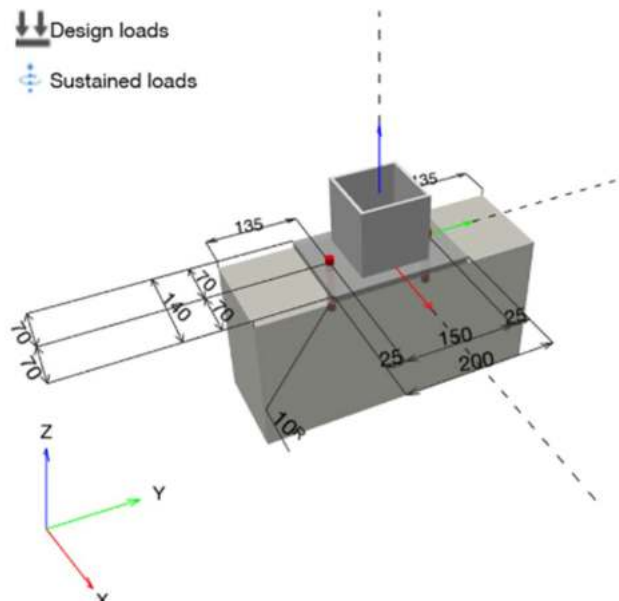
1 Input data

Anchor type and size:	HIT-HY 200-A + HAS-U 5.8 100 Years M12
Return period (service life in years):	100
Item number:	2223821 HAS-U 5.8 M12x110 (insert) / 2022696 HIT-HY 200-A (mortar)
Effective embedment depth:	$h_{ef, opt} = 70.0 \text{ mm}$ ($h_{ef, limit} = 185.0 \text{ mm}$)
Material:	5.8
Approval No.:	ETA 11/0493
Issued Valid:	10/12/2021 -
Proof:	Design Method EN 1992-4, Chemical
Stand-off installation:	$e_b = 0.0 \text{ mm}$ (no stand-off); $t = 10.0 \text{ mm}$
Baseplate ^R :	$l_x \times l_y \times t = 140.0 \text{ mm} \times 200.0 \text{ mm} \times 10.0 \text{ mm}$; (Recommended plate thickness: not calculated)
Profile:	Square hollow, $100 \times 100 \times 5.0$; ($L \times W \times T$) = $100.0 \text{ mm} \times 100.0 \text{ mm} \times 5.0 \text{ mm}$
Base material:	cracked concrete, C45/55, $f_{c, cyl} = 45.00 \text{ N/mm}^2$; $h = 215.0 \text{ mm}$, Temp. short/long: $0/0 \text{ } ^\circ\text{C}$, User-defined partial material safety factor $\gamma_c = 1.500$
Installation:	hammer drilled hole, Installation condition: Dry
Reinforcement:	No reinforcement or Reinforcement spacing $\geq 150 \text{ mm}$ (any $\emptyset$) or $\geq 100 \text{ mm}$ ($\emptyset \leq 10 \text{ mm}$) no longitudinal edge reinforcement



^R - The anchor calculation is based on a rigid baseplate assumption.

Geometry [mm] & Loading [kN, kNm]



1.1 Load combination

Case	Description	Forces [kN] / Moments [kNm]	Seismic	Fire	Max. Util. Anchor [%]
1	Combination 1	$N = -69.400; V_x = 9.500; V_y = 0.000;$ $M_x = 0.000; M_y = 0.000; M_z = 0.000;$ $N_{sUS} = 0.000; M_{x,sUS} = 0.000; M_{y,sUS} = 0.000;$	no	no	73

2 Load case/Resulting anchor forces

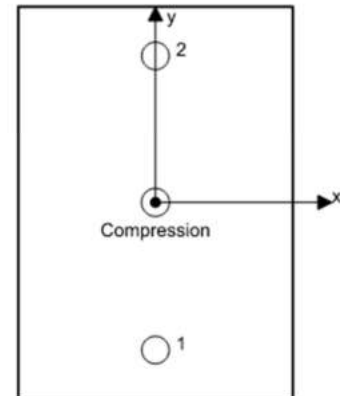
Anchor reactions [kN]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	4.750	4.750	0.000
2	0.000	4.750	4.750	0.000

max. concrete compressive strain: 0.08 [‰]
max. concrete compressive stress: 2.48 [N/mm²]
resulting tension force in (x/y)=(0.0/0.0): 0.000 [kN]
resulting compression force in (x/y)=(0.0/0.0): 69.400 [kN]

Anchor forces are calculated based on the assumption of a rigid baseplate.



3 Tension load (EN 1992-4, Section 7.2.1)

	Load [kN]	Capacity [kN]	Utilization β_N [%]	Status
Steel failure*	N/A	N/A	N/A	N/A
Concrete Breakout failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

4 Shear load (EN 1992-4, Section 7.2.2)

	Load [kN]	Capacity [kN]	Utilization β_V [%]	Status
Steel failure (without lever arm)*	4.750	20.256	24	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout failure**	9.500	32.087	30	OK
Concrete edge failure in direction x+**	9.500	13.191	73	OK

* highest loaded anchor **anchor group (relevant anchors)

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk}	=	0.000 [kN]	δ_N	=	0.0000 [mm]
V_{Sk}	=	3.519 [kN]	δ_V	=	0.1759 [mm]
			δ_{NV}	=	0.1759 [mm]

Long term loading:

N_{Sk}	=	0.000 [kN]	δ_N	=	0.0000 [mm]
V_{Sk}	=	3.519 [kN]	δ_V	=	0.2815 [mm]
			δ_{NV}	=	0.2815 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the baseplate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid baseplates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the baseplate are not considered - the baseplate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required baseplate thickness with CBFEM to limit the stress of the baseplate based on the assumptions explained above. The proof if the rigid baseplate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with EN 1992-4, Annex A!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 6.1 of EN 1992-4! For larger diameters of the clearance hole see section 6.2.2 of EN 1992-4!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- For the determination of the $\psi_{re,v}$ (concrete edge failure) the minimum concrete cover defined in the design settings is used as the concrete cover of the edge reinforcement.
- Drilled hole cleaning must be performed according to instructions for use (blow twice with oil-free compressed air (min. 6 bar), brush twice, blow twice with oil-free compressed air (min. 6 bar)).
- Characteristic bond resistances depend on short- and long-term temperatures.
- Edge reinforcement is not required to avoid splitting failure
- The characteristic bond resistances depend on the return period (service life in years): 100

Fastening meets the design criteria!

Structural Calculations

7 Installation data

Baseplate, steel: S 275; $E = 210,000.00 \text{ N/mm}^2$; $f_{yk} = 275.00 \text{ N/mm}^2$

Profile: Square hollow, $100 \times 100 \times 5.0$; $(L \times W \times T) = 100.0 \text{ mm} \times 100.0 \text{ mm} \times 5.0 \text{ mm}$

Hole diameter in the fixture: $d_f = 14.0 \text{ mm}$

Plate thickness (input): 10.0 mm

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and size: HIT-HY 200-A + HAS-U 5.8 100 Years M12

Item number: 2223821 HAS-U 5.8 M12x110 (insert) / 2022696 HIT-HY 200-A (mortar)

Maximum installation torque: 40 Nm

Hole diameter in the base material: 14.0 mm

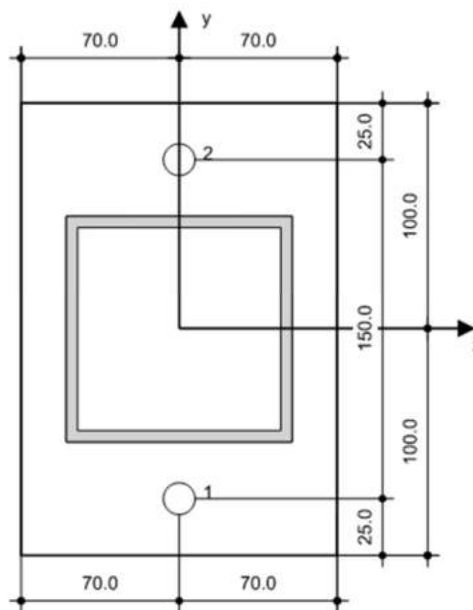
Hole depth in the base material: 70.0 mm

Minimum thickness of the base material: 100.0 mm

Hilti HAS-U threaded rod with HIT-HY 200 injection mortar with 70 mm embedment h_{ef} , M12, Steel galvanized, Hammer drilling installation per ETA 11/0493

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [mm]

Anchor	x	y	$c_{,x}$	$c_{,y}$	$c_{,x}$	$c_{,y}$
1	0.0	-75.0	70.0	70.0	135.0	285.0
2	0.0	75.0	70.0	70.0	285.0	135.0

Structural Calculations

Rooflight Trimmer Steel Beam

Span = 4.0 m

Load acting on beam:

Width of roof supported = $5.0/2 = 2.5$ m

$$G_k = 2.5 \times 0.7 = 1.75 \text{ kN/m}$$

$$Q_k = 2.5 \times 0.75 = 1.88 \text{ kN/m}$$

Point load from trimmer spanning the other way @ 0.8m and 3.0m

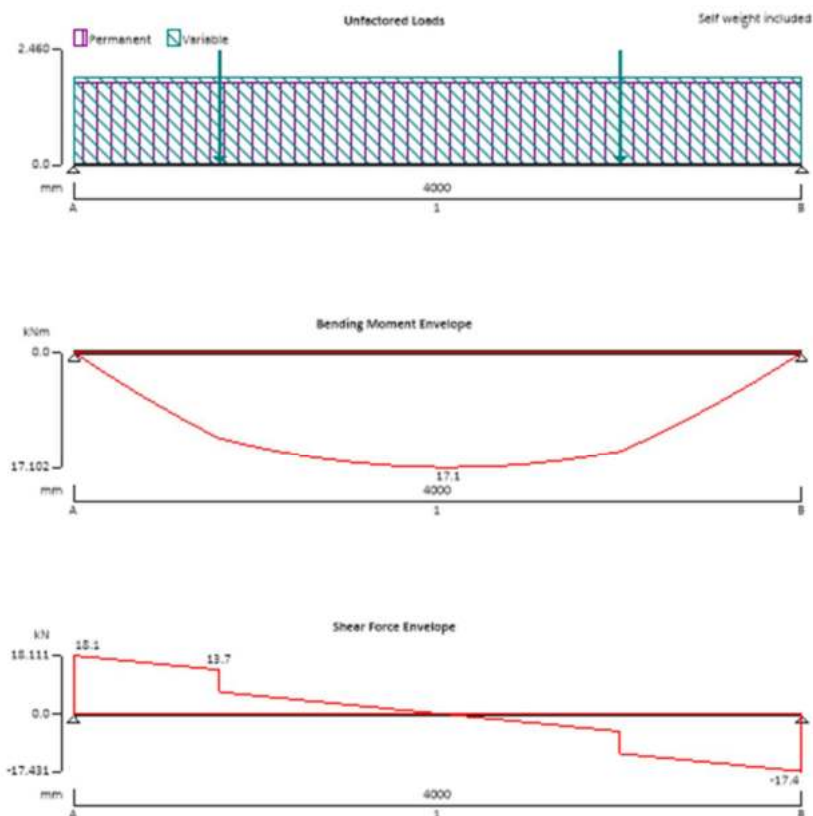
$$G_k = 0.7 \times 3.2/2 \times 4.1/2 = 2.30 \text{ kN}$$

$$Q_k = 0.75 \times 3.2/2 \times 4.1/2 = 2.46 \text{ kN}$$

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

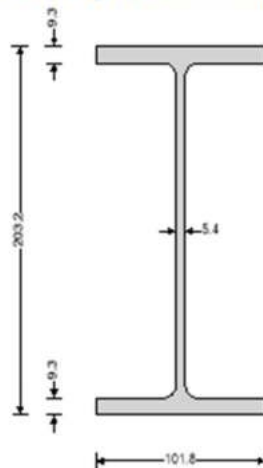
Permanent self weight of beam $\times 1$
Permanent full UDL 1.75 kN/m
Variable full UDL 1.88 kN/m
Permanent point load 2.3 kN at 800 mm
Variable point load 2.46 kN at 800 mm
Permanent point load 2.3 kN at 3000 mm
Variable point load 2.46 kN at 3000 mm

Analysis results

Maximum moment	$M_{max} = 17.1$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 18.1$ kN	$V_{min} = -17.4$ kN
Deflection	$\delta_{max} = 4.7$ mm	$\delta_{min} = 0$ mm
Maximum reaction at support A	$R_{A_{max}} = 18.1$ kN	$R_{A_{min}} = 18.1$ kN
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 6.4$ kN	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 6.3$ kN	
Maximum reaction at support B	$R_{B_{max}} = 17.4$ kN	$R_{B_{min}} = 17.4$ kN
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 6.1$ kN	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 6.1$ kN	

Section details

Section type UKB 203x102x23 (Tata Steel Advance) Steel grade S355



Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force $V_{Ed} = 18$ kN
Design shear resistance $V_{c,Rd} = 253.7$ kN
PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{Ed} = 17.1$ kNm
Des. bending resist. moment $M_{c,Rd} = 83.1$ kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT} = 1.442$
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 $M_{b,Rd} = 37.9$ 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 $\delta_{lim} = 8$ mm
Maximum deflection $\delta = 4.711$ mm
PASS - Maximum deflection does not exceed deflection limit

Structural Calculations

Steel Beam over Kitchen/Diner

Span = 5.65 m

Load acting on beam:

Width of floor supported = $4.25/2 = 2.13$ m

$$\begin{aligned} G_k &= 2.13 \times 0.9 = 1.92 \text{ kN/m} \\ Q_k &= 2.13 \times 1.5 = 3.20 \text{ kN/m} \end{aligned}$$

Point load from timber partition 2.4m high @1.16m

$$G_k = 2.4 \times 0.5 \times 2.13 = 2.56 \text{ kN}$$

Width of pitched roof supported = $5.4/2 = 2.6$ m

$$\begin{aligned} G_k &= 2.6 \times 1.44 = 3.74 \text{ kN/m} \\ Q_k &= 2.6 \times 0.6 = 1.56 \text{ kN/m} \end{aligned}$$

Width of flat roof supported = $1.0/2 = 0.5$ m

$$\begin{aligned} G_k &= 0.5 \times 0.7 = 0.35 \text{ kN/m} \\ Q_k &= 0.5 \times 0.75 = 0.38 \text{ kN/m} \end{aligned}$$

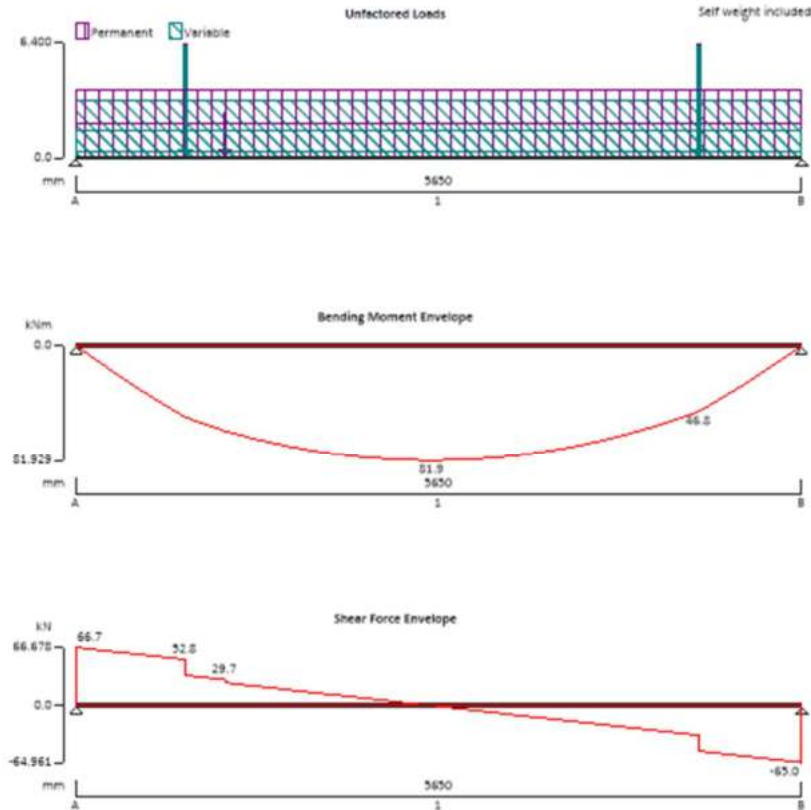
Point load from rooflight trimmers (from above) @0.85m and @4.85m

$$\begin{aligned} G_k &= 6.4 \text{ kN} \\ Q_k &= 6.3 \text{ kN} \end{aligned}$$

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 first floor - Permanent full UDL 1.92 kN/m first floor - Variable full UDL 3.2 kN/m Permanent point load 6.4 kN at 850 mm Variable point load 6.3 kN at 850 mm Permanent point load 6.4 kN at 4850 mm Variable point load 6.3 kN at 4850 mm partition - Permanent point load 2.56 kN at 1160 mm pitched roof - Permanent full UDL 3.74 kN/m
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Maximum reaction at support B	$R_{B_max} = 33.3$ kN	$R_{B_min} = 33.3$ kN
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 11.9$ kN	
Unfactored variable load reaction at support B	$R_{B_Variable} = 11.5$ kN	

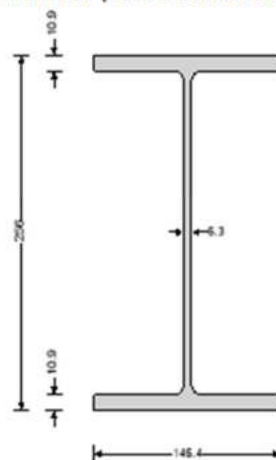
pitched roof - Variable full UDL 1.54 kN/m
flat roof - Permanent full UDL 0.35 kN/m
flat roof - Variable full UDL 0.38 kN/m

Analysis results

Maximum moment	$M_{max} = 81.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 66.7 \text{ kN}$	$V_{min} = -65 \text{ kN}$
Deflection	$\delta_{max} = 7.6 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 66.7 \text{ kN}$	$R_{A_{min}} = 66.7 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 26.4 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 20.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 65 \text{ kN}$	$R_{B_{min}} = 65 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 25 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 20.8 \text{ kN}$	

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 $V_{Ed} = 67 \text{ kN}$ 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 $M_{Ed} = 81.9 \text{ kNm}$ Des. bending resist. moment $M_{c,Rd} = 171.5 \text{ kNm}$
PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads
Limiting deflection $\delta_{lim} = 15.7 \text{ mm}$ Maximum deflection $\delta = 7.576 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit

ADOPT 254x145x37 S355 STEEL BEAM

Steel lintel supporting beam over kitchen/diner

Span = 1.8 m

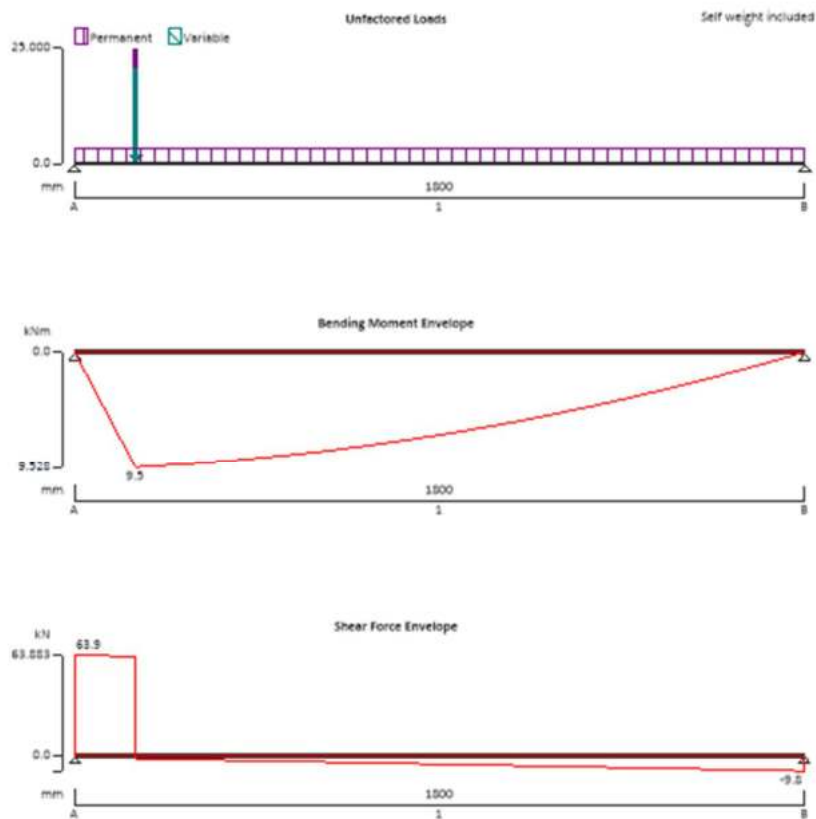
Loadings from steel beam @150mm from support: Permanent 25kN Imposed 20.8kN

Loading from masonry over 3.35kN/m

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
	Permanent point load 25 kN at 150 mm
	Variable point load 20.8 kN at 150 mm
	wall over - Permanent full UDL 3.35 kN/m

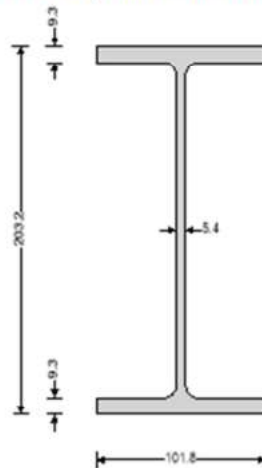
Analysis results

Maximum moment	$M_{max} = 9.5$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 63.9$ kN	$V_{min} = -9.8$ kN
Deflection	$\delta_{max} = 0.4$ mm	$\delta_{min} = 0$ mm

Maximum reaction at support A	$R_{A_max} = 63.9 \text{ kN}$	$R_{A_min} = 63.9 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 26.1 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 19.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 9.8 \text{ kN}$	$R_{B_min} = 9.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 5.3 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 1.7 \text{ kN}$	

Section details

Section type UKB 203x102x23 (Tata Steel Advance) Steel grade S355



Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force $V_{Ed} = 64 \text{ kN}$ Design shear resistance $V_{c,Rd} = 253.7 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{Ed} = 9.5 \text{ kNm}$ Des.bending resist.moment $M_{c,Rd} = 83.1 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT} = 0.978$ 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 $M_{b,Rd} = 60.9 \text{ kNm}$
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 $\delta_{lim} = 5 \text{ mm}$ Maximum deflection $\delta = 0.428 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit

Structural Calculations

Steel beam over large span opening

Span = 4.1 m

Load acting on beam:

Width of floor supported = $3.55/2 = 1.78$ m

$$\begin{aligned} G_k &= 1.78 \times 0.9 = 1.60 \text{ kN/m} \\ Q_k &= 1.78 \times 1.5 = 2.66 \text{ kN/m} \end{aligned}$$

Point load from timber partition 2.4m high @3.50m

$$G_k = 2.4 \times 0.5 \times 1.78 = 2.14 \text{ kN}$$

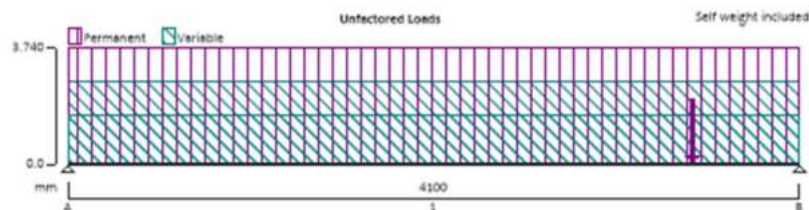
Width of pitched roof supported = $5.4/2 = 2.6$ m

$$\begin{aligned} G_k &= 2.6 \times 1.44 = 3.74 \text{ kN/m} \\ Q_k &= 2.6 \times 0.6 = 1.56 \text{ kN/m} \end{aligned}$$

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

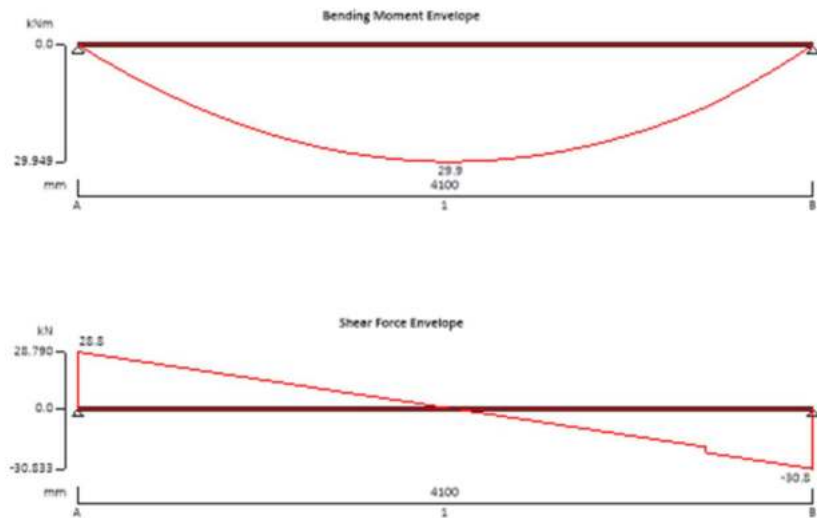
TEDDS calculation version 3.0.14

**Support conditions**

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

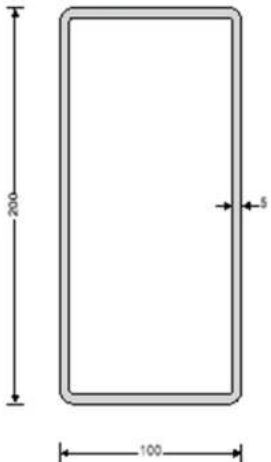
Analysis results

Maximum moment	$M_{max} = 29.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 28.8 \text{ kN}$	$V_{min} = -30.8 \text{ kN}$
Deflection	$\delta_{max} = 4.9 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 28.8 \text{ kN}$	$R_{A_{min}} = 28.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 11.7 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 8.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 30.8 \text{ kN}$	$R_{B_{min}} = 30.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 13.2 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 8.7 \text{ kN}$	



Section details
Section type

RH S 200x100x5.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade S355H



Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force	$V_{Ed} = 31 \text{ kN}$	Design shear resistance	$V_{c,Rd} = 392.6 \text{ kN}$
PASS - Design shear resistance exceeds design shear force			

Check bending moment - Section 6.2.5

Design bending moment	$M_{Ed} = 29.9 \text{ kNm}$	Des.bending resist.moment	$M_{c,Rd} = 65.7 \text{ kNm}$
PASS - Design bending resistance moment exceeds design bending moment			

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads			
Limiting deflection	$\delta_{lim} = 8.2 \text{ mm}$	Maximum deflection	$\delta = 4.947 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit			

Structural Calculations

Masonry design – central 1200mm pier

Loadings from floor and roof UDL:

Permanent $1.60+3.74=5.34\text{kN/m}$

Imposed $2.66+1.56=4.22\text{kN/m}$

Loadings from steel lintel:

LH Permanent 13.2kN

LH Imposed 8.7kN

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedd's calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	30.9 kN	39.8 kN	N/A	N/A	0.776	Pass

Masonry panel details

Panel length	L = 1250 mm	Panel height	h = 2500 mm
Load bearing leaf thickness	t = 100 mm	Effective height	$h_{ef} = 2500$ mm
Effective thickness	$t_{ef} = 126$ mm		

Masonry material details

Unit type	Autoclaved aerated concrete - Group 1		
Mean masonry strength	$f_b = 10.1$ N/mm ²	Specific weight of units	$\gamma = 18$ kN/m ³
Mortar type	M4 - General Purpose	Mortar strength	$f_m = 4.0$ N/mm ²
Characteristic strength	$f_k = 5.73$ N/mm ²		

Design compressive strength of masonry for inner (loaded) leaf

Category of manufacturing	Category I	Class of execution control	Class 2
Partial factor for material	$\gamma_M = 2.70$	Design strength of masonry	$f_d = 2.12$ N/mm ²

Partial safety factors for design loads

Permanent partial factor	$\gamma_{IG} = 1.35$	Variable partial factor	$\gamma_{IQ} = 1.50$
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Superimposed vertical loading details

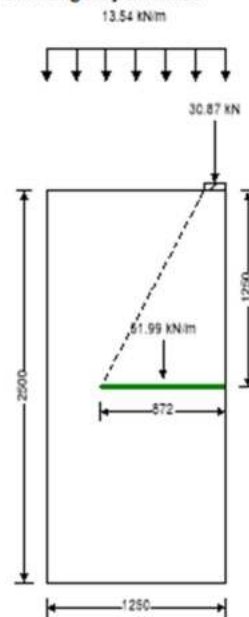
Permanent UDL at top of wall	$g_k = 5.34$ kN/m	Variable UDL at top of wall	$q_k = 4.22$ kN/m
Eccent. of permanent UDL	$e_{ga} = 0$ mm	Eccent. of variable UDL	$e_{qu} = 0$ mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	$\lambda_{lim} = 27$	Slenderness ratio	$\lambda = 19.8$
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PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - point load from large span lintel



Permanent load

 $G_{kic1} = 13.20 \text{ kN}$

Variable load

 $Q_{kic1} = 8.70 \text{ kN}$

Eccentricity of load

 $e_{c1} = 0 \text{ mm}$

Length of load

 $L_{c1} = 150 \text{ mm}$

Width of load

 $w_{c1} = 100 \text{ mm}$

Height of load

 $h_{c1} = 2500 \text{ mm}$

Distance to right edge

 $r_{11} = 0 \text{ mm}$

Distance to nearest edge

 $a_{11} = 0 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load $N_{Edc1} = 30.87 \text{ kN}$

Design resistance

 $N_{Rdc1} = 39.77 \text{ kN}$ **PASS - Design resistance exceeds applied concentrated load**

Walls subjected to mainly vertical loading - Section 6.1.2

Vertical load at mid-height $N_{Ed1} = 51.99 \text{ kN/m}$

Design resistance

 $N_{Rd1} = 131.51 \text{ kN/m}$ **PASS - Design value of vertical resistance exceeds applied vertical load**

Masonry design – central 2000mm pier with 140mm wide blockwork tied into inner leaf

Loadings from floor and roof UDL: Permanent $1.60+3.74=5.34\text{kN/m}$ Imposed $2.66+1.56=4.22\text{kN/m}$

Loadings from steel beam: Permanent 26.4kN Imposed 20.7kN

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedd's calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	66.7 kN	30.5 kN	1.53 N/mm ²	3.18 N/mm ²	0.482	Pass

Masonry panel details

Panel length	L = 2000 mm	Panel height	h = 2500 mm
Load bearing leaf thickness	t = 100 mm	Effective height	$h_{ef} = 2500$ mm
Effective thickness	$t_{ef} = 126$ mm		

Masonry material details

Unit type	Autoclaved aerated concrete - Group 1		
Mean masonry strength	$f_b = 10.1$ N/mm ²	Specific weight of units	$\gamma = 18$ kN/m ³
Mortar type	M4 - General Purpose	Mortar strength	$f_m = 4.0$ N/mm ²
Characteristic strength	$f_k = 5.73$ N/mm ²		

Design compressive strength of masonry for inner (loaded) leaf

Category of manufacturing	Category I	Class of execution control	Class 2
Partial factor for material	$\gamma_M = 2.70$	Design strength of masonry	$f_d = 2.12$ N/mm ²

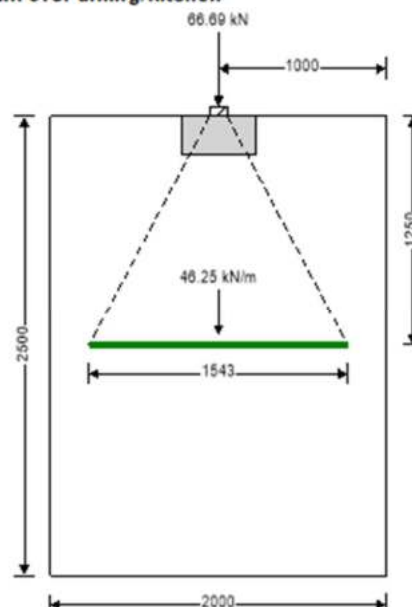
Partial safety factors for design loads

Permanent partial factor	$\gamma_{G1} = 1.35$	Variable partial factor	$\gamma_{Q1} = 1.50$
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Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	$\lambda_{lim} = 27$	Slenderness ratio	$\lambda = 19.8$
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PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - beam over dining/kitchen

Permanent load	$G_{kc1} = 26.40 \text{ kN}$	Variable load	$Q_{kc1} = 20.70 \text{ kN}$
Eccentricity of load	$e_{c1} = 0 \text{ mm}$	Length of load	$L_{c1} = 100 \text{ mm}$
Width of load	$w_{c1} = 100 \text{ mm}$	Height of load	$h_{c1} = 2500 \text{ mm}$
Distance to right edge	$r_{11} = 950 \text{ mm}$	Distance to nearest edge	$a_{11} = 950 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load	$N_{Edc1} = 66.69 \text{ kN}$	Design resistance	$N_{Rdc1} = 30.54 \text{ kN}$
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Applied concentrated load exceeds design resistance, spreader required!**Design of spreader beam**

Type of spreader	Concrete padstone	Type of load	Point load
Length of spreader	$L_{sp1} = 440 \text{ mm}$	Width of spreader	$w_{sp1} = 100 \text{ mm}$
Height of spreader	$h_{sp1} = 215 \text{ mm}$	Eccentricity of load	$e_{sp1} = 0 \text{ mm}$
Modulus of elasticity	$E_{sp1} = 29962 \text{ N/mm}^2$	Maximum moment	$M_{Edsp1} = 3.65 \text{ kNm}$
Maximum shear	$V_{Edsp1} = 33.34 \text{ kN}$	Allowable stress	$\sigma_{Rdsp1} = 3.18 \text{ N/mm}^2$
Design stress	$\sigma_{Edsp1} = 1.53 \text{ N/mm}^2$		

PASS - Design stress under spreader is less than the allowable bearing stress**Walls subjected to mainly vertical loading - Section 6.1.2**

Vertical load at mid-height	$N_{Ed1} = 46.25 \text{ kN/m}$	Design resistance	$N_{Rd1} = 131.51 \text{ kN/m}$
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PASS - Design value of vertical resistance exceeds applied vertical load

Adopt 100x215x440mm mass concrete padstone beneath beam bearing, ensure 140mm blockwork wall is toothed into inner leaf of cavity wall

Masonry design – bearing under lintel supporting kitchen/diner steel

Loadings from steel beam:

Permanent 26.1kN

Imposed 19.8

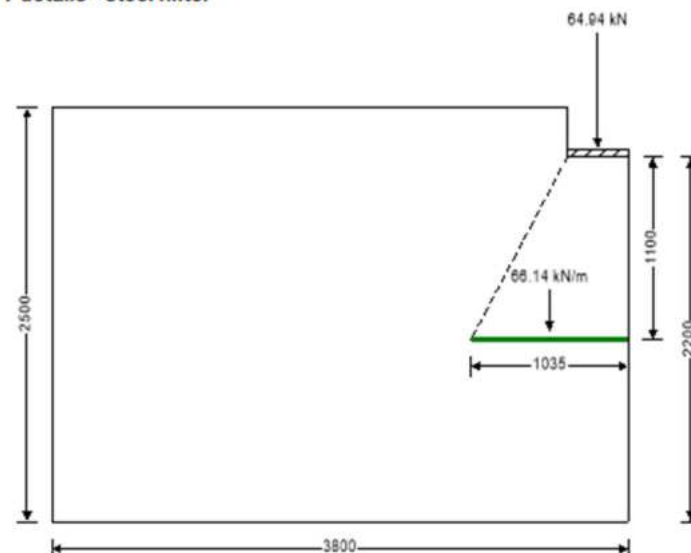
MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedd's calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	64.9 kN	91.2 kN	N/A	N/A	0.712	Pass

Masonry panel detailsPanel length $L = 3800$ mmPanel height $h = 2500$ mmLoad bearing leaf thickness $t = 100$ mmEffective height $h_{ef} = 2500$ mmEffective thickness $t_{ef} = 126$ mm**Masonry material details**Unit type **Autoclaved aerated concrete - Group 1**Mean masonry strength $f_b = 10.1$ N/mm²Specific weight of units $\gamma = 18$ kN/m³Mortar type **M4 - General Purpose**Mortar strength $f_m = 4.0$ N/mm²Characteristic strength $f_k = 5.73$ N/mm²**Design compressive strength of masonry for inner (loaded) leaf**Category of manufacturing **Category I**Class of execution control **Class 2**Partial factor for material $\gamma_M = 2.70$ Design strength of masonry $f_d = 2.12$ N/mm²**Partial safety factors for design loads**Permanent partial factor $\gamma_{G1} = 1.35$ Variable partial factor $\gamma_{Q1} = 1.50$ **Slenderness ratio of masonry wall - Section 5.5.1.4**Slenderness ratio limit $\lambda_{lim} = 27$ Slenderness ratio $\lambda = 19.8$ **PASS - Slenderness ratio is less than slenderness limit****Concentrated Load 1 details - steel lintel**

Permanent load	$G_{kc1} = 26.10 \text{ kN}$	Variable load	$Q_{kc1} = 19.80 \text{ kN}$
Eccentricity of load	$e_{c1} = 0 \text{ mm}$	Length of load	$L_{c1} = 400 \text{ mm}$
Width of load	$w_{c1} = 100 \text{ mm}$	Height of load	$h_{c1} = 2200 \text{ mm}$
Distance to right edge	$r_{11} = 0 \text{ mm}$	Distance to nearest edge	$a_{11} = 0 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load	$N_{Edc1} = 64.94 \text{ kN}$	Design resistance	$N_{Rdc1} = 91.20 \text{ kN}$
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PASS - Design resistance exceeds applied concentrated load**Walls subjected to mainly vertical loading - Section 6.1.2**

Vertical load at mid-height	$N_{Ed1} = 66.14 \text{ kN/m}$	Design resistance	$N_{Rd1} = 131.51 \text{ kN/m}$
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PASS - Design value of vertical resistance exceeds applied vertical load**Ensure Steel beam over lintel bears minimum 400mm on to the masonry.**

Structural Calculations

Foundation design**600mm wide footings along side elevation**

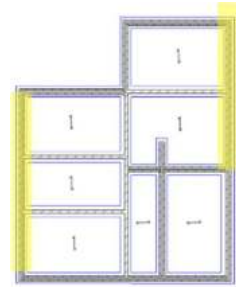
Worst case loading from above:

Loading from roof – permanent $1.44 \times 5.9 / 2 \times 11 / 2 = 23.4 \text{ kN}$

Spread over 11m of gable wall = 2.12 kN/m

Loading from roof – imposed $0.6 \times 5.9 / 2 \times 11 / 2 = 4.87 \text{ kN}$

Spread over 11m of gable wall = 0.89 kN/m



Lightweight cavity wall above ground $4.8 \text{ m} \times 2.1 \text{ kN/m}^2 = 10.08 \text{ kN/m}$

Brick outer leaf cavity wall below ground floor $1.1 \text{ m} \times 4.45 \text{ kN/m}^2 = 4.90 \text{ kN/m}$

Cavity wall below ground with concrete cavity infill $0.5 \times 6.0 = 3.0 \text{ kN/m}$

Trench foundation founded 2.5m below existing ground level – worst case increase in loading by replacing 20 kN/m^3 clay with 25 kN/m^3 concrete increases the load on the underside of the foundation by $(25-20) \times 2 \times 0.60 = 6.0 \text{ kN/m}$

Total load on the U/s of the footing $2.12 + 0.89 + 10.08 + 4.9 + 3.0 + 6.0 = 26.99 \text{ kN/m}$

Allowable ground bearing pressure is taken as 50 kN/m^2

$26.99 / 0.6 / 1.0 = 44.98 \text{ kN/m}^2 < 50 \text{ kN/m}^2$

THEREFORE OK

Adopt 600mm wide trench footing beneath the cavity wall on side elevation**Check footing by the garage**

Take 750mm wide footing

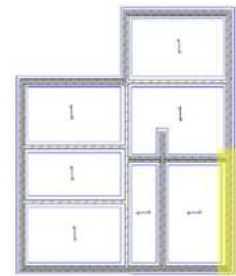
Worst case loading from above:

Loading from roof – permanent $1.44 \times 5.9 / 2 \times 11 / 2 = 23.4 \text{ kN}$

Spread over 11m of gable wall = 2.12 kN/m

Loading from roof – imposed $0.6 \times 5.9 / 2 \times 11 / 2 = 4.87 \text{ kN}$

Spread over 11m of gable wall = 0.89 kN/m



Allow for decreasing cavity wall height – take average as: $(3.3 \times 4.75 + (2.55 + 4.75) / 2 \times 3.0) / 6.3 = 4.23 \text{ m}$

Therefore loading from lightweight wall above ground $4.23 \times 2.1 = 8.88 \text{ kN/m}$

Ground floor DL $4.05 \times 3.55 / 2 = 7.20 \text{ kN/m}$

Ground floor IL $2.5 \times 3.55 / 2 = 4.43 \text{ kN/m}$

Brick outer leaf cavity wall below ground floor $1.1 \text{ m} \times 4.45 \text{ kN/m}^2 = 4.90 \text{ kN/m}$

Cavity wall below ground with concrete cavity infill $0.5 \times (25-20) \times 0.3 = 0.75 \text{ kN/m}$

(increase in loading compared to the weight of soil)

Trench foundation founded 2.5m below existing ground level – worst case increase in loading by replacing 20 kN/m^3 clay with 25 kN/m^3 concrete increases the load on the underside of the foundation by $(25-20) \times 2 \times 0.75 = 7.5 \text{ kN/m}$

Total load on the U/s of the footing $2.12 + 0.89 + 8.88 + 4.9 + 0.75 + 7.5 + 7.20 + 4.43 = 36.67 \text{ kN/m}$

Allowable ground bearing pressure is taken as 50 kN/m^2

$36.67 / 0.75 / 1.0 = 48.89 \text{ kN/m}^2 < 50 \text{ kN/m}^2$

THEREFORE OK

Adopt 750mm wide trench footing beneath the cavity wall on side elevation

750mm wide footings along front and rear

Worst case loading from above:

Roof DL $1.44 \times 11/4 = 3.96 \text{ kN/m}$

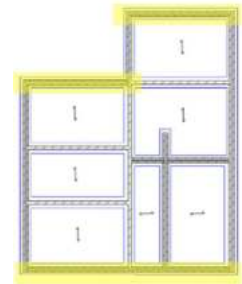
Roof IL $0.6 \times 11/4 = 1.65 \text{ kN/m}$

First floor DL $0.9 \times 3.65/2 = 1.64 \text{ kN/m}$

First floor IL $1.5 \times (3.65/2 - 1.2) + 0.5 \times 1.2 = 1.54 \text{ kN/m}$

Ground floor DL $4.05 \times 3.65/2 = 7.39 \text{ kN/m}$

Ground floor IL $1.5 \times 3.65/2 = 2.74 \text{ kN/m}$



Lightweight cavity wall above ground $2.4 \times 2.1 \text{ kN/m}^2 = 5.04 \text{ kN/m}$

Brick outer leaf cavity wall below ground floor $1.1 \times 4.45 \text{ kN/m}^2 = 4.90 \text{ kN/m}$

Cavity wall below ground with concrete cavity infill $0.5 \times (25 - 20) \times 0.3 = 0.75 \text{ kN/m}$
(increase in loading compared to the weight of soil)

Trench foundation founded 2.5m below existing ground level – worst case increase in loading by replacing 20 kN/m^3 clay with 25 kN/m^3 concrete increases the load on the underside of the foundation by $(25 - 20) \times 2 \times 0.75 = 7.5 \text{ kN/m}$

Total load on the U/s of the footing $3.96 + 1.65 + 1.64 + 1.54 + 7.39 + 2.74 + 5.04 + 4.9 + 0.75 + 7.5 = 37.11 \text{ kN/m}$

Allowable ground bearing pressure is taken as 50 kN/m^2

$37.11 / 0.75 / 1.0 = 49.48 \text{ kN/m}^2 < 50 \text{ kN/m}^2$

THEREFORE OK

Adopt 750mm wide trench footing beneath the cavity wall along front and rear elevations**900mm wide footings under internal walls**

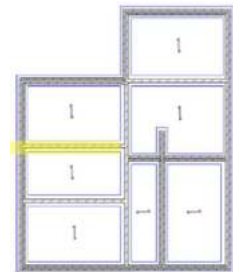
Worst case loading from above:

First floor DL $0.9 \times 6.7/2 = 3.02 \text{ kN/m}$

First floor IL $1.5 \times (6.7/2 - 0.1) = 4.88 \text{ kN/m}$

Ground floor DL $4.36 \times 6.7/2 = 14.61 \text{ kN/m}$

Ground floor IL $1.5 \times (6.7/2 - 0.14) = 4.82 \text{ kN/m}$



Lightweight internal wall above ground $2.4 \times 1.55 \text{ kN/m}^2 = 3.72 \text{ kN/m}$

Solid wall below ground floor $1.0 \times 4.3 \text{ kN/m}^2 = 4.30 \text{ kN/m}$

Solid wall below ground $0.5 \times (25 - 20) \times 0.215 = 0.54 \text{ kN/m}$

(increase in loading compared to the weight of soil)

Trench foundation founded 2.5m below existing ground level – worst case increase in loading by replacing 20 kN/m^3 clay with 25 kN/m^3 concrete increases the load on the underside of the foundation by $(25 - 20) \times 2 \times 0.9 = 9.0 \text{ kN/m}$

Total load on the U/s of the footing $3.02 + 4.88 + 14.61 + 4.82 + 3.72 + 4.30 + 0.54 + 9.0 = 44.89 \text{ kN/m}$

Allowable ground bearing pressure is taken as 50 kN/m^2

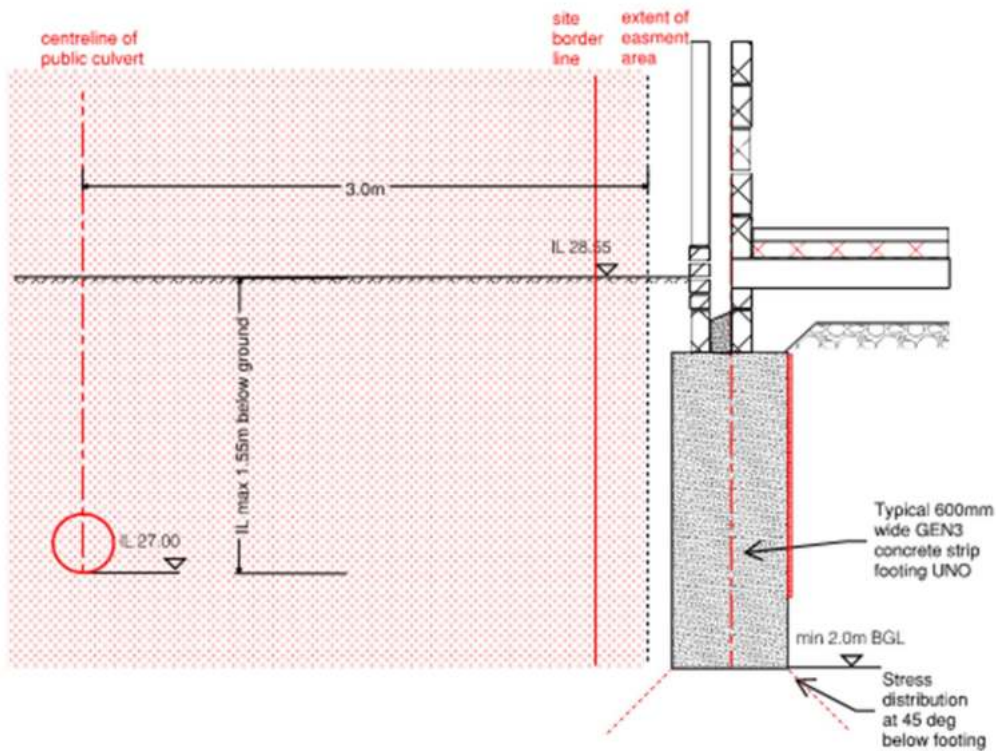
$44.89 / 0.9 / 1.0 = 49.88 \text{ kN/m}^2 < 50 \text{ kN/m}^2$

THEREFORE OK

Adopt 900mm wide trench footing beneath internal walls

6.0 Culvert Protection

In line with the requirements outlined in the Discharge Conditions letter dated 12th April 2023, the structural design was carried out with the intent of limiting the width of the footing on the side nearest to the culvert. After careful consideration, as shown in sketch 2230109-THE-ZZ-00-SK-S-0005 (extract below) the excavation in the area nearest to the culvert is outside of the easment area.



Section X through foundation nearest to the culvert

The underside of the footing is below the culvert invert level as illustrated in the section sketch. Any load distributed from the foundation on to the ground will not impact the culvert, as the load spread at 45 degrees is well below the culvert level.

No heavy machinery to be positioned on the outside of the trench to avoid disturbing the ground within the easment area.