LANAFRED CONSULTANTS

10 Abberton Road

Single Storey Rear Extension & Internal Alterations -
Structural Calculations

Calculation Ref - 000101-LFC-PW-XX-CA-W-0001-C01
Document Status - FOR CONSTRUCTION

Lanafred Consultants Ltd
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Registered Address: 24 Hooper Street, Hartford, Northwich, CW8 1TR

Document Control

Contract Title	10 Abberton Road
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Control Date	25/05/26

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Record of Issue

Issue	Status	Author	Date	Check	Date	Authorised	Date
C01	AFC	Craig Gelder	25/05/26	Charlie Stockley	25/05/26	Craig Gelder	25/05/26

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Appendix A Tedds Outputs

1.0 Standards and Drawing References

Standards, Codes of Practice & Design Guidance

BS EN 1990:2002+A1:2005 Eurocode. Basis of structural design
BS EN 1991-1-1:2002 Eurocode 1. Actions on structures - General actions - Densities, self-weight, imposed loads for buildings.
BS EN 1993-1-1:2005+A1:2014 Eurocode 3. Design of steel structures - General rules and rules for buildings
BS EN 1993-1-8:2005 Eurocode 3. Design of steel structures - Design of joints
BS EN 1996-1-1:2005+A1:2012 Eurocode 6. Design of masonry structures - General rules for reinforced and unreinforced masonry structures
BS EN 1997-1:2004+A1:2013 Eurocode 7. Geotechnical design - General rules

And associated National Annexes

Software

In-House Spreadsheets
Tekla Tedds

Other Technical References

Structural Engineers Pocket Book 2nd Edition
Simpson Strong tie technical literature

Design Drawings

Drawing Number	Rev
000101-LFC-PW-XX-DR-W-0001	A01
000101-LFC-PW-XX-DR-W-0002	A01
000101-LFC-PW-XX-DR-W-0003	A01

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2.0 Basis of Design

2.1 Introduction

The following calculations pertain to the structural design and associated modifications intended for 10 Abberton Road, Didsbury, Manchester, M20 1HU to facilitate a single storey rear extension and internal altercations.

The project involves:

- Design of 2No steel members and assessment of the remaining masonry walls for vertical and lateral support to enable removal of two walls (external and internal). The main beam supporting the external wall may be specified as a twin UB or single UC subject to contractor preference.

- Design of an additional steel beam in the new single-story extension over the proposed aprox 5.0m sliding doors

- The specification of suitable timber members to support a new proposed 1.5m x 3.0m roof light in the new flat roof.

- Associated connection design between structural steels.

- Typical anticipated foundation detail to improve current strip footings. Please note: No ground information is currently available and as such this detail will be based on assumed ground conditions and typical existing strip footing details. Given the existing single storey kitchen area has seen some settlement it is advised that the client is aware some form of improvement to the foundations are provisioned for. The contractor should investigate the existing foundations and ground conditions at the earliest opportunity. If the conditions are unfavourable additional modifications to the foundations may be required. If so, an additional fee may be required to undertake any further design work.

See below plan view showing the proposed structural modifications.

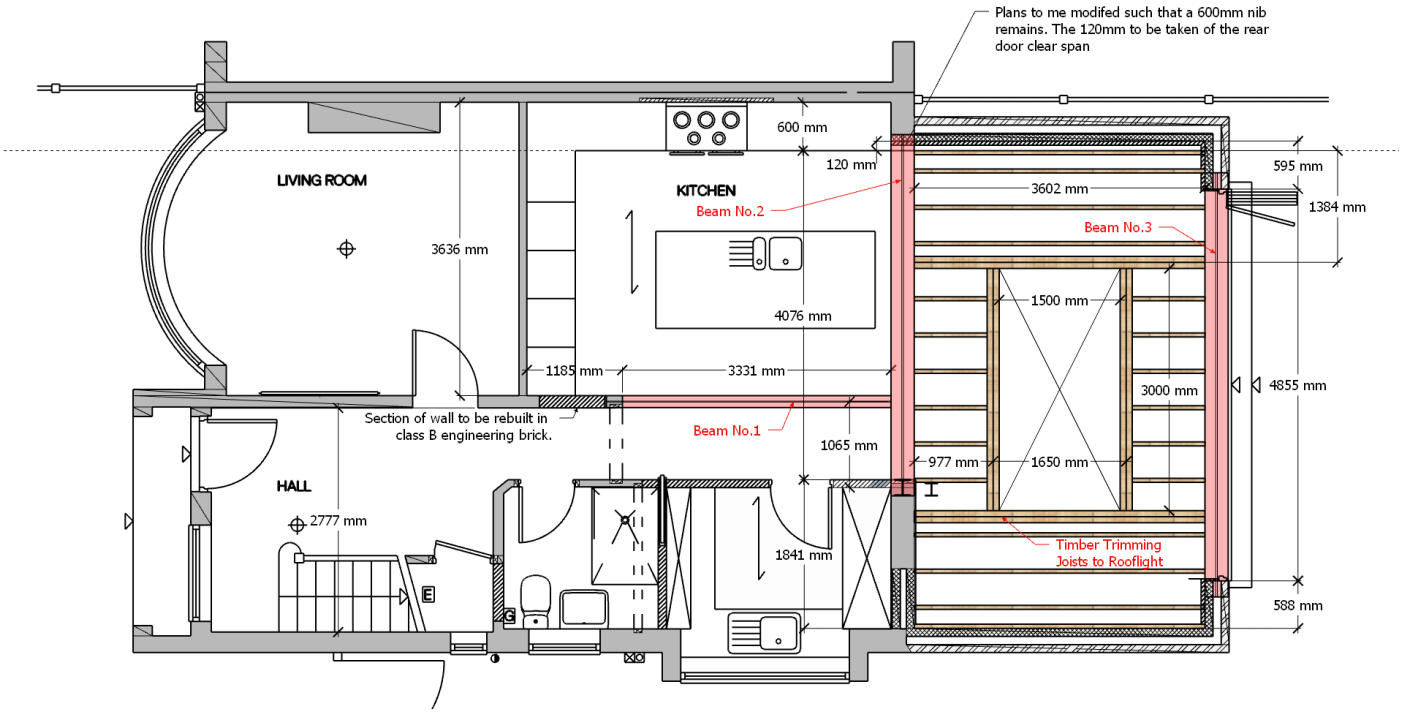


Figure 1 - Plan View of Proposed Structural Alterations

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2.2 Design Loads and Combinations

The following basic loads have been considered in this design:

Permanent Loads

Permanent loads are determined in accordance with BS EN 1991-1-1: 2002 Eurocode 1 Actions on structures - Part 1 General and UK NA to BS EN 1991-1-1 2002 Eurocode 1 Actions on structures - Part 1 General.

The following densities will be taken when deducing the self-weight of the existing structure:

- Masonry Walls 19kN/m^3 (London Stock)
- Blockwork Walls 18kN/m^3 (Medium Dense Block)
- Concrete roof tiles 0.51kN/m^2
- Timber Rafters 0.2kN/m^2
- Timber Battons and felt 0.2kN/m^2
- Timber Floor Timber boards 0.15kN/m^2
- Floor Joists 0.2kN/m^2
- Ceiling & services 0.15kN/m^2

Variable Loads

The following Variable loads will be considered in the design:

- Variable loading for domestic floors: 1.5 kN/m^2
- Snow Loading on Roofs: 0.6kN/m^2
- Snow Loading on flat Roofs: 0.75kN/m^2
- Access Loading on roofs: 0.6kN/m^2 & 0.9kN Point Load BS EN 1991-1-1 UK NA Table NA.7

Design Approach Partial Factors

Partial factors and load combinations as per NA to BS EN 1990:2002+A1:2005 Eurocode. Basis of structural design.

2.4 Ground Conditions

A review of the ground conditions and existing foundations will be undertaken by the contractor. Advice regarding modifications to the foundations can be made once information is received.

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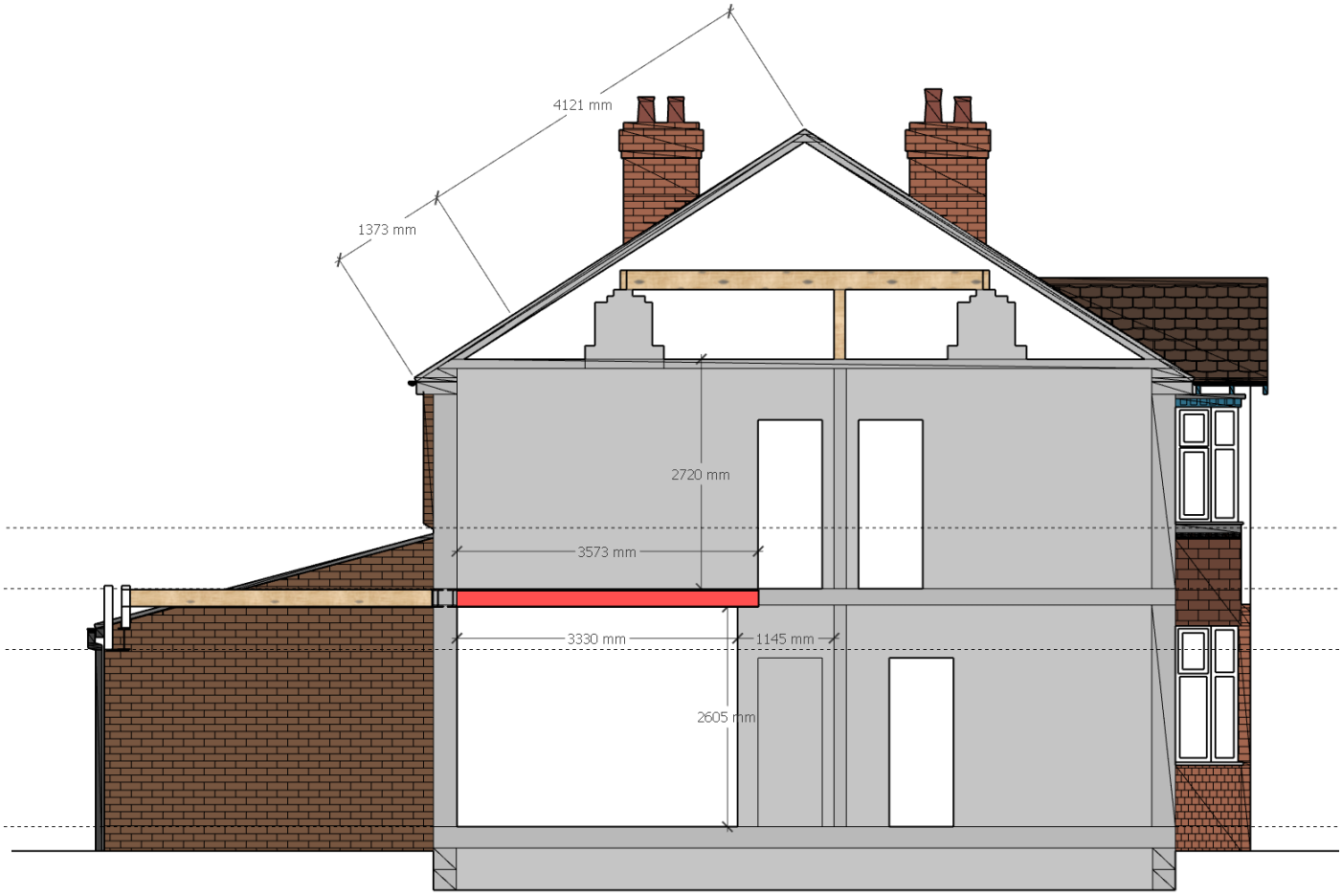
3.0 Analysis

3.1 Design of Beam No.1

Beam No.1 has a clear span of 3330mm supported on one end by the internal masonry load bearing wall and the new proposed Beam No.2 at the other. Propose to utilise a UB

The wall is currently loading bearing supporting the:

- Existing timber first floors formed with 3”x7” timbers (73x175mm)
- Loft ceiling
- Portion of the roof supported by the existing purlins.







Roof Load take down

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From the model file total area of roof supported by the masonry pier is 9.81+8.80 = 18.61m^2

Permanent load G_k = 18.61 x (0.2+0.2+0.50) = 16.75 kN (SLS)
- Concrete roof tiles 0.50kN/m^2
- Timber Rafters 0.2kN/m^2
- Timber Battons and felt 0.2kN/m^2

Variable load Q_k = 18.61 x (0.6) = 11.17kN (SLS)
- Snow / Access Loading on Roofs: 0.6kN/m^2

Load applied to masonry pier will disperse and at beam level at as UDL. Therefore:

Permanent load UDL g_k = 16.75/3.573 x 1.1 uplift factor for pier and prulins = 5.16kN/m (SLS)
Variable load UDL q_k = 11.17/3.573 = 3.13kN/m (SLS)

Masonry wall load

Height of masonry 2.72m above beam.
- Masonry Walls 19kN/m^3 (London Stock)

Permanent load UDL g_k = 2.72 x 19 x 0.1m wall thickness = 5.17kN/m (SLS)

Floor load

Span of floors 3.626 and 2.777m
- Timber Floor Timber boards 0.15kN/m^2
- Floor Joists 0.2kN/m^2
- Ceiling & services 0.15kN/m^2

Permanent load UDL g_k = (0.15+0.2+0.15) x (3.626+2.777)/2 = 1.60kN/m (SLS)
Variable load UDL q_k = 1.5 x (3.626+2.777)/2 = 4.80kN/m (SLS)

Total Loads

Permanent load UDL g_k = 5.16+5.17+1.60 = 11.93kN/m (SLS)
Variable load UDL q_k = 3.13+4.80 = 7.93kN/m (SLS)

SEE TEDDS CALC FOR BEAM NO 1

Max reactions at beam ends

Unfactored permanent load reaction at supports = 21.5 kN
Unfactored variable load reaction at supports = 14 kN
Factored load = 21.3 x 1.35 + 14 x 1.5 = 50kN
Deflection adheres to L/360 ok

SEE TEDDS CALC FOR BEAM NO 1 Masonry Bearing with Class B engineering brick
SEE TEDDS CALC FOR BEAM NO 1 to 2 Steel connection design
Note added a nominal 10kN tying force to connection design

3.2 Joist connection to Beam No.1

To secure joist to Beam no. 1 and ensure beam is flush within ceiling use Simpson strong-tie JHA270/75 Joist Hangers face fixed. Note some joists are wider therefore contractor to review and select appropriate sizes where required.

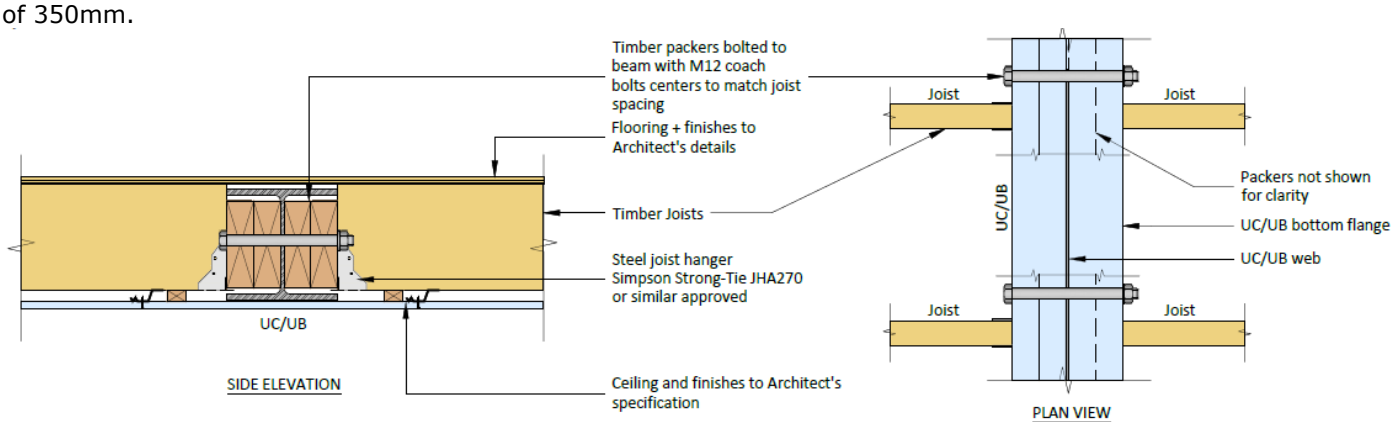
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Typical spacing of joists 350mm and max span is 3.626m therefore maximum unfactored reaction:
((0.15+0.2+0.15) + 1.5) x 3.626/2 x 0.35 = 1.269kN (SLS)

Published performance values are based upon 3.75 x 30mm square twist nails being used throughout yielding 3.2kN > 1.269kN therefore ok.

References	Header - Dimensions		Number of Fasteners			Characteristic capacities - Timber C16 [kN]		Safe Working Loads [kN]		
	Height		Header		Joist	$R_{1,k}$	$R_{2,k}$	$R_{1,SWL,Medium\ Term}$	$R_{1,SWL,Long\ term}$	Short Term Uplift
	Min	Max.	Face Qty	Top Qty	Qty	N3.75x30 CSA4.0x30	N3.75x30 CSA4.0x30	C16 header N3.75x30 CSA4.0x30	C16 header N3.75x30 CSA4.0x30	C16 Joist N3.75x30 CSA4.0x30
JHA270/38	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/44	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/47	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/50	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/63	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/75	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/91	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1
JHA270/100	200	250	20	-	4	7.6	2.1	3.6	3.2	1.1

Joist Hanger with Adjustable Height Strap - JHA | Simpson Strong-Tie
Proposed connection of timber packing to UB beam as per detail below with M12 coach bolts to match joist centres of 350mm.



3.3 Design of Beam No.2 (Twin UB)

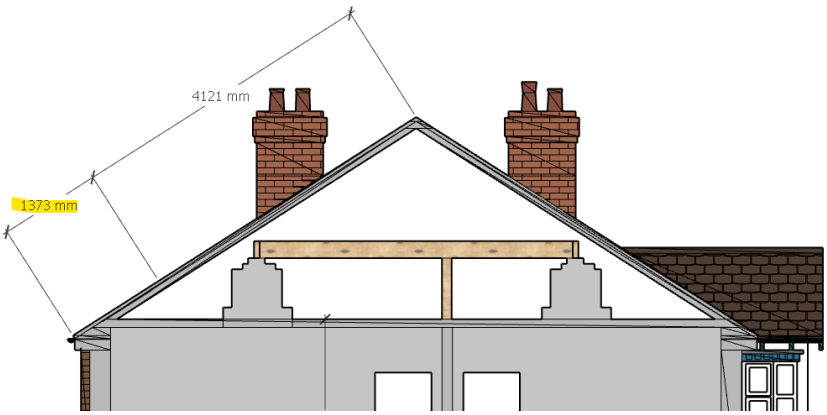
Beam No.2 has a clear span of 4075mm supported at both ends by the masonry load bearing walls. Propose to utilise a UBs that will be bolted together.

The internal wall is currently loading bearing supporting a portion of the roof.

Roof Load take down

From the model file total area of roof supported by the masonry pier is 1.373m^2/m run of wall

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Permanent load g_k = 1.373 x (0.2+0.2+0.50) = 1.24 kN/m (SLS)
- Concrete roof tiles 0.50kN/m^2
- Timber Rafters 0.2kN/m^2
- Timber Battons and felt 0.2kN/m^2

Variable load Q_k = 1.373 x (0.6) = 0.82kN/m (SLS)
- Snow Loading on Roofs: 0.6kN/m^2

Masonry wall load

Height of masonry 2.50m above beam
- Masonry Walls 19kN/m^3 (London Stock)

Permanent load UDL g_k = 2.5 x 19 x 0.1m wall thickness = 4.75kN/m (SLS)

Each 0.1m leaf taken by one beam.

Note: Conservative due to window openings

Floor load

N/a

Reaction load from beam No.1

Applied to the inner beam applied 1.065m from the end.

Unfactored Permanent load = 21.5 kN
Unfactored Variable Load = 14.0 kN

Loads from flat roof

Applied to the outer beam applied as equivalent UDL

Flat roof loading based on below

Permanent load G_k = (0.2+0.45+0.54+0.15) = 1.34kN/m^2 (SLS)
- Timber joists 0.2kN/m^2
- Waterproofing 0.45kN/m^2
- Sedum trays for green roof weight: 55kg/m² = Equivalent to 0.54kN/m^2
- Ceiling & services 0.15kN/m^2

Note sky light weight: 46kg/m² = Equivalent to 0.45kN/m^2 < 1.34kN/m^2 conservative to just assumed green roof throughout. Specification for Flat Roof Skylights

1000x3000mm Panoramic Rooflight | Anti-Pooling | KingsView

Whilst some load will be concentrated on the trimmers just consider applied as equivalent UDL. The span of Flat roof of 3.60m therefore;

Permanent load UDL g_k = 1.34 x 3.60/2 = 2.41kN/m (SLS)

Variable load q_k = 0.75 x 3.60/2 = 1.35kN/m (SLS)
- Snow/access Loading on flat Roofs: 0.75kN/m^2

Total Loads Inner Beam

Permanent load UDL g_k = 1.24+4.75 = 5.99kN/m (SLS)
Variable load UDL q_k = 0.82kN/m (SLS)

Unfactored Permanent Point load = 21.5 kN applied 1.065m from the end
Unfactored Variable Point Load = 14 kN applied 1.065m from the end

SEE TEDDS CALC FOR BEAM NO 2 Inner

Total Loads outer Beam

Permanent load UDL g_k = 2.41+4.75 = 7.16kN/m (SLS)
Variable load UDL q_k = 1.35kN/m (SLS)

Whilst calculations show outer beam may be dropped to UB203x133x25 specify both at UB203x133x30 S275 Prefabricated with 18mm Holes centre Drilled sandwiched close together to form 311mm overall on the assumption the existing cavity is 75mm. Contractor to measure and confirm prior to fabrication.

SEE TEDDS CALC FOR BEAM NO 2 Outer

Inner Beam Reactions

Unfactored permanent load reaction at support A = 29.6 kN
Unfactored variable load reaction at support A = 12.3 kN

Unfactored permanent load reaction at support B = 18.8 kN
Unfactored variable load reaction at support B = 5.2 kN

Outer Beam Reactions

Unfactored permanent load reaction at support A = 15.9 kN
Unfactored variable load reaction at support A = 2.9 kN

Unfactored permanent load reaction at support B = 15.9 kN
Unfactored variable load reaction at support B = 2.9 kN

Total loads at support A

Unfactored permanent load = 15.9 + 29.6 = 45.5 kN
Unfactored variable load = 2.9 + 12.3 = 15.2 kN

SEE TEDDS CALC FOR BEAM NO 2 Masonry check Support A

Total loads at support B

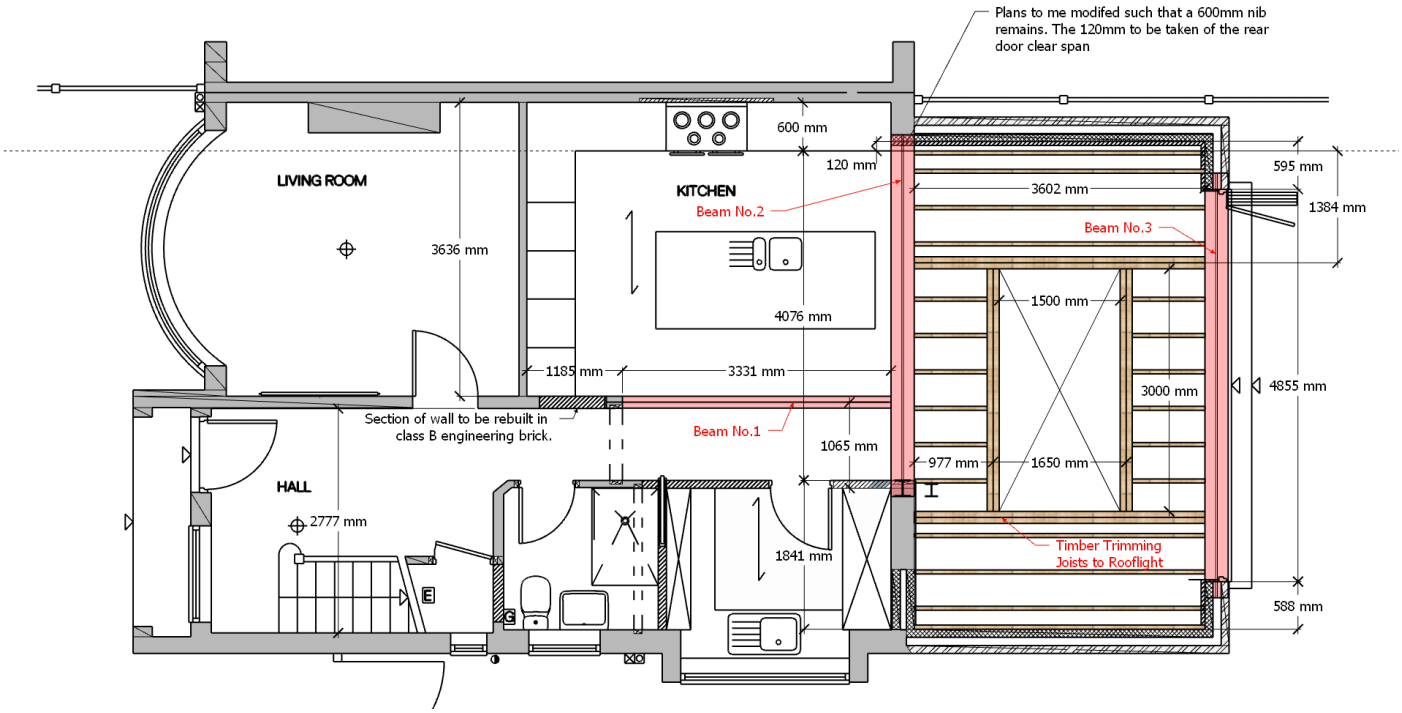
Unfactored permanent load = 15.9 + 18.8 = 34.7 kN
Unfactored variable load = 2.9 + 5.2 = 8.1 kN
Bearing on Class B Engineering brick wall

SEE TEDDS CALC FOR BEAM NO 2 Masonry check Support B

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3.4 Design of Trimming joists to support Roof light

Loads from flat roof



Flat roof loading based on below

Permanent load G_k = (0.2+0.45+0.54+0.15) = 1.34kN/m² (SLS)
- Timber joists 0.2kN/m²
- Waterproofing 0.45kN/m²
- Sedum trays for green roof weight: 55kg/m² = Equivalent to 0.54kN/m²
- Ceiling & services 0.15kN/m²

Note sky light weight: 46kg/m² = Equivalent to 0.45kN/m² < 1.34kN/m²

To be less conservative don't assume green roof selfweight throughout. Load taken by trimming timbers to roof light is a 2.45m wide section 3.0m long. Roof light is 1.5m with so (1.5 x 0.45 + 1.45 x 1.34)/2.45 = 0.8 kN/m²

Say 1.0 kN/m²

The span of flat roof of 3.60m therefore;

Permanent load on timbers g_k = 1.0 x (2.45x3x0.5)/2 = 1.84kN (SLS)

Variable load q_k = 0.75 x (2.45x3x0.5)/2 = 1.38kN (SLS)
- Snow/access Loading on flat Roofs: 0.75kN/m²

Trimming joists to take 0.225m worth of roof load as well as loading from roof light trimmers.

Variable load q_k = 0.75 x 0.225 = 0.17 kN/m (SLS)
Permanent load g_k = 1.35 x 0.225 = 0.30 kN/m (SLS)

Propose to use 2No C24 200x72mm timbers

SEE TEDDS CALC FOR Trimming Joists

Double member trimmer joists and trimming joists should be securely fixed together. min. 4mm x 48mm M16 washers under head & nut. Bolt centres for double member trimmers at 1200mm with minimum of two bolts at 1/3 span positions. Min end distance 7d = 7 x 16 = 112mm.

Connection from trimmers to trimming joists

Proposed to connect trimming timbers to trimming joists with Heavy Duty Face Fix Hangers SAE500/146/2 with full nailing. Capacity 14.8kN > 1.84+1.38 = 3.22kN therefore OK.

Heavy Duty Face Fix Hangers - SAE-SAEL | Simpson Strong-Tie

References	Characteristic capacities - Timber to timber - Full nailing							
	Number of Fasteners		Product characteristic capacities - Timber C24 [kN]			Safe Working Load [kN]		
	Header	Joist	$R_{1,k}$		$R_{2,k}$	$R_{1,SWL,Long\ term}$		$R_{2,SWL,Short\ term}$
	Qty	Qty	N3.75x30 - C16 Header	N3.75x30 - C24 Header	N3.75x30 - C16 Joist	C16 header N3.75x30	C24 header N3.75x30	C16 Joist N3.75x30
SAE500/146/2	32	16	33.3	35.6	12.2	13.9	14.8	6.1

To ensure that the load from the trimmers is distributed equally between the members of the trimming joists at, or immediately adjacent to the connection 2No M16 bolts are required one either side of hanger.

Ref BS 5298-2:2002 cl 6.6.4 and 6.6.6

Fadm = F × K56 × K57; Both K factors are 1.0 in this instance

Table 71 shear per bolt = 3.30kN long term in C24 giving a total of 6.60kN permissible per joint > 3.22kN therefore OK.

Place 2No M16 bolts with one either side of hanger which will ensure min 4d between bolts.

Table 71 — Basic single shear loads in kilonewtons (kN) for one 4.6 grade steel bolt in a two member timber-to-timber joint: C24 timber

Load duration	Minimum member thickness	Direction of loading											
		Parallel to grain						Perpendicular to grain					
		Bolt diameter						Bolt diameter					
	mm	M8	M10	M12	M16	M20	M24	M8	M10	M12	M16	M20	M24
Long-term	16	0.48	0.59	0.69	0.88	1.05	1.20	0.43	0.52	0.60	0.73	0.84	0.93
	22	0.66	0.81	0.95	1.21	1.44	1.65	0.60	0.72	0.82	1.01	1.16	1.27
	35	1.06	1.29	1.52	1.93	2.30	2.62	0.95	1.14	1.31	1.61	1.84	2.03
	44	1.29	1.62	1.91	2.43	2.89	3.29	1.20	1.43	1.65	2.02	2.32	2.55
	47	1.33	1.74	2.04	2.59	3.09	3.52	1.23	1.53	1.76	2.16	2.47	2.72
	60	1.52	2.07	2.60	3.31	3.94	4.49	1.40	1.89	2.25	2.75	3.16	3.47
	72	1.55	2.28	2.93	3.97	4.73	5.39	1.47	2.06	2.64	3.30	3.79	4.17
	97	1.55	2.40	3.42	5.05	6.37	7.26	1.47	2.26	3.07	4.43	5.11	5.62
	147	1.55	2.40	3.42	5.94	8.40	10.63	1.47	2.26	3.18	5.42	7.09	8.51

Table 81 — Minimum bolt and dowel distances

Direction of loading [a]	End distance		Edge distance		Distances between fasteners or rows of fasteners	
	Loaded	Unloaded	Loaded	Unloaded	Measured across the grain	Measured parallel to the grain
Loading parallel to grain	$7d$	$4d$	$1.5d$	$1.5d$	$4d$	$4d$ [b]
Loading perpendicular to grain	$4d$	$4d$	$4d$	$1.5d$	$4d$	$4d$

[a] When a load is applied at an angle to the grain between 0° and 90°, or when a group of fasteners resists a moment, the larger of the distances for parallel or perpendicular to the grain loading should be used.

[b] Greater loads may be obtained with a spacing of $7d$ (see Annex G).

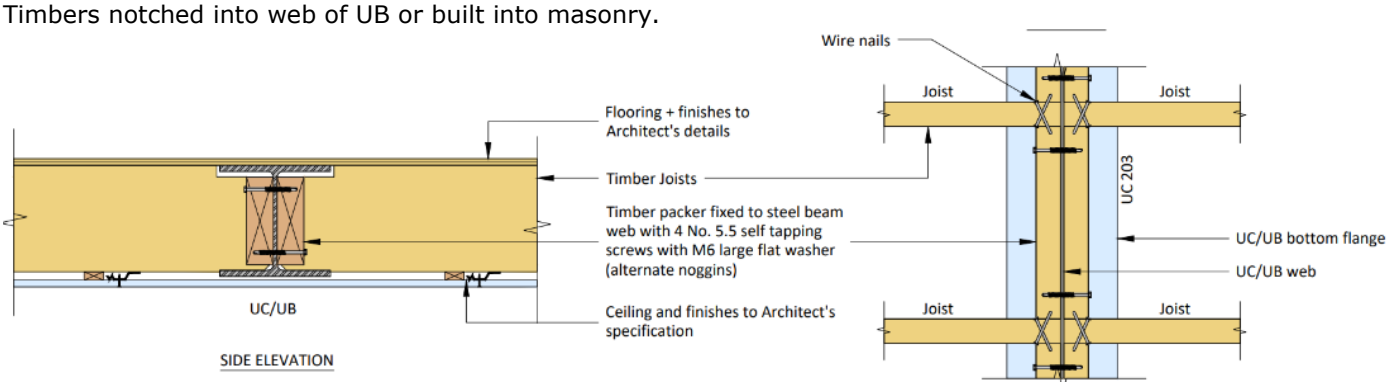
For remaining section of roof specify 200x72mm C24 timber joists at 450mm c/c

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Variable load q_k = 0.75 x 0.45 = 0.34 kN/m (SLS)
Permanent load g_k = 1.35 x 0.45 = 0.61 kN/m (SLS)

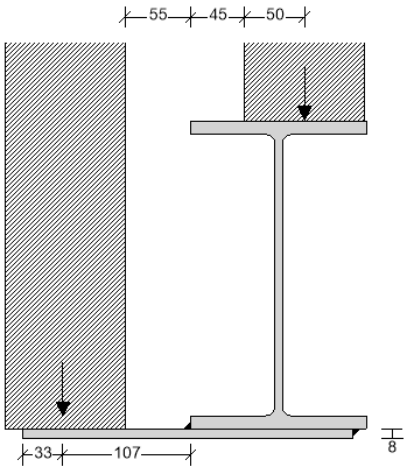
SEE TEDDS CALC FOR Normal Joists

Timbers notched into web of UB or built into masonry.



3.5 Design of Beam No.3

Propose to use a plate welded to the underside of the beam to support outer leaf of masonry. Due to exposure of the plate the member will need to be galvanised.



Loads from flat roof

Flat roof loads as per earlier calculation.

Permanent load UDL g_k = 2.41kN/m (SLS)
Variable load q_k = 1.35kN/m (SLS)

Whilst some load will be concentrated on the trimmers just consider applied as equivalent UDL.

Loads from masonry and beam selfweight

Load on plate will include circa 0.5m and 0.75m of masonry on the beam and plate, respectively. Additional line loads are:

Plate masonry Permanent load UDL g_k = 0.1 x 19 x 0.5 = 0.95 kN/m (SLS)
Beam masonry Permanent load UDL g_k = 0.1 x 19 x 0.75 = 1.43 kN/m (SLS)

Beam section size UB 254x146x37

Selfweight of beam 37 x 9.81×10^{-3} x 1.25 uplift for plate = 0.45 kN/m (SLS)

Total Loads on Beam

Permanent load UDL g_k = 2.41+0.95+1.43+0.45 = 5.24kN/m (SLS)
Variable load UDL q_k = 1.35kN/m (SLS)

SEE TEDDS CALC FOR BEAM NO. 3

Max reaction at end of beam on the basis of a clear span of 4.735m with 200mm bearing.

Permanent load UDL g_k = 5.24kN/m x 4.935 x 0.5 = 12.9kN (SLS)
Variable load UDL q_k = 1.35kN/m x 4.935 x 0.5 = 3.33kN (SLS)
Total = 16.23kN (SLS)

Factored = 12.9 x 1.35 + 3.33 x 1.5
= 17.41 + 5.0 =22.42 kN

SEE TEDDS CALC FOR BEAM NO. 3 Masonry Support

Total load of load bearing wall at ends of Beam no. 3

Total unfactored reaction = 16.23kN
Applied over 0.6m nib therefore 27.05kN/m

Weight of masonry = 3.5 x 0.1 x 19 x 2no leafs = 13.3kN/m

Total = 13.3 + 27.05 = 40.35kN/m

C3 interior: high humidity, some air pollution exterior: urban inland or mild coastal 0.7 to 2

Hot dip galvanize to BS EN ISO 1461 with a minimum average coating of 85µm on steel

4.0 Conclusions

Design requirements presented in the following drawings:

Drawing Number	Rev
000101-LFC-PW-XX-DR-W-0001	A01
000101-LFC-PW-XX-DR-W-0002	A01
000101-LFC-PW-XX-DR-W-0003	A01

Some key points to flag up:

- These drawings are intended to outline the structural requirements only and do not provide the level of detail normally included in architectural working drawings. Architectural drawings for a flat roof typically set out the intended roof build-up, drainage strategy, edge and parapet details, penetrations, thermal and moisture control measures, and any access or maintenance requirements, providing the full design intent for how the roof should be constructed. In contrast, our structural drawing relates only to the supporting structure for the roof and confirms the elements required to safely carry the roof's self-weight, access loads, and snow loads.
- Where no separate architectural drawings are supplied, the Contractor is responsible for coordinating all dimensional setting out, construction details, and finishes, and for ensuring full compliance with the Building Regulations. Any discrepancies, omissions, or uncertainties must be referred to the Designer or Structural Engineer before construction proceeds.
- The chimney breast is bonded into the external masonry wall immediately adjacent to the window. At its base, previous alterations appear to have been carried out to remove the lower section of the chimney,

with a "makeshift" corbel formed using a mix of brick and blockwork. While the bond into the wall and bodged corbel does provide some inherent stability, sections of the external masonry, both the internal leaf and the outer masonry below the rafter wall plate, are in relatively poor condition. As an additional precaution, it is recommended that an acrow prop with a strongboy is installed beneath the outer edge of the stack to provide temporary support during demolition of the existing single-storey structure. It is further suggested that the Client's bricklayer removes the redundant chimney stack entirely and rebuilds this portion of the external wall so it is flush with the surrounding masonry before any new steelwork is installed.

- Additionally, the internal masonry leaf above the proposed Beam no. 2 steelwork is also noted to be in poor condition, as are other areas. During inspection, masonry units were easily dislodged, and there were noticeable cracks and gaps in the mortar. It is recommended that the Client's bricklayer inspects these areas thoroughly and undertakes any necessary repairs or rebuilding to ensure the wall is structurally sound prior to installing steelwork or carrying out major alterations.
- When demolishing the lean-to, this should be done in a controlled, piecemeal manner. Particular care should be taken at the junction with the existing external wall of the main house to avoid damaging the remaining masonry, especially given its condition. Using safe access, the structure should be dismantled progressively from the top down.
- Some information has been included regarding the foundations; however, this element was not included within our original scope. As noted in the original quote, this cannot be taken on without confirmation of the ground conditions and the nature of the existing foundations. Given the subsidence affecting the existing lean-to, it is recommended—consistent with discussions on site—that the Client pays particular attention to this area. Once the ground conditions and existing foundations are confirmed, we are happy to review the information if required.
- The full calculation pack will be issued once confirmation is received that the Client is satisfied with the design as it stands. If no changes are required, a brief confirmation will allow us to forward everything to Building Control.
- Finally, the Client will need to ensure that the planning department signs off the proposed changes to the extension, as the previous approval was for a lean-to roof rather than a flat roof. The Client's preferred parapet heights and any other alterations will also need to be taken into account.

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Health, Safety and Environment Information.
Notes below are additional to hazard/risks normally associated with this type of work:



All works shall be carried out with due consideration to health and safety in accordance with the CDM Regulations 2015, and the following key risks shall be identified, managed, and mitigated by the Contractor:

a. **Structural instability risk:** Temporary loss of support during removal of loadbearing elements (e.g. walls/chimneys) may lead to partial or full structural collapse. Adequate temporary works (e.g. props, needles) must be specified and installed prior to alteration.
b. **Incorrect installation of steel beams (RSJs)**: Risks include inadequate bearing, misalignment, insufficient end support, or omission of padstones, which may result in structural failure. All steelwork shall be installed in accordance with structural engineer's details and inspected prior to loading.
c. **Inadequate temporary works:** Failure or improper installation of propping systems during construction may cause sudden collapse. Temporary works and sequencing must be planned and implemented by a competent person.
d. **Ground instability and excavation collapse:** Excavations for foundations may collapse or undermine adjacent structures or services. Suitable excavation support and safe working practices must be adopted.
e. **Service strikes:** Risk of damaging existing underground or concealed services (gas, electric, water, drainage) during excavation or structural alterations. All services must be identified prior to works commencing.
f. **Manual handling injuries:** Lifting and positioning heavy steel beams and materials may result in injury. Mechanical lifting equipment and safe handling techniques shall be used.
g. **Working at height:** Installation of beams and associated works may require working at height, posing a risk of falls. Appropriate access equipment (e.g. scaffolding, platforms) must be provided.
h. **Fire risk:** Hot works (cutting, welding) associated with steel installation may present a fire hazard. Hot works procedures and fire prevention measures must be implemented.
i. **Dust and airborne contaminants:** Cutting, demolition, and excavation may generate dust, including silica, posing health risks. Suitable dust suppression and PPE shall be used.
j. **Interface with existing structure:** Unknown construction methods or hidden defects in the existing building may pose risks during alteration works. Careful investigation and monitoring are required.
k. **Weather conditions:** Adverse weather (rain, frost) may affect ground conditions, concrete curing, and safety of excavations and structural works.

All risks shall be assessed by a suitably competent Contractor(s) prior to commencement, and appropriate control measures implemented. Any unforeseen conditions shall be reported immediately and works paused until safe solutions are agreed.

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Appendix A – Tedds Outputs

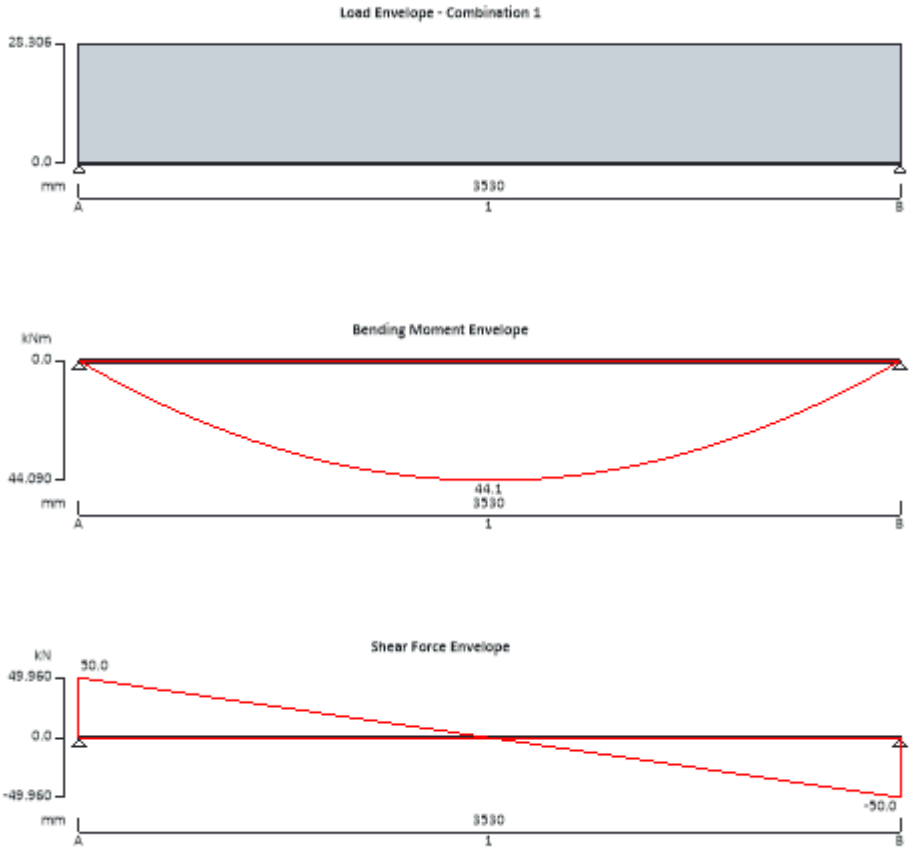
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Tekla Tedds	Project			Job no.	
	10 Abberton			000101	
	Calcs for			Start page no./Revision	
				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	CG	03/06/2026	CS	25/05/2026	CG	25/05/2026

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 3.0.15



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 11.93 kN/m
	Variable full UDL 7.93 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

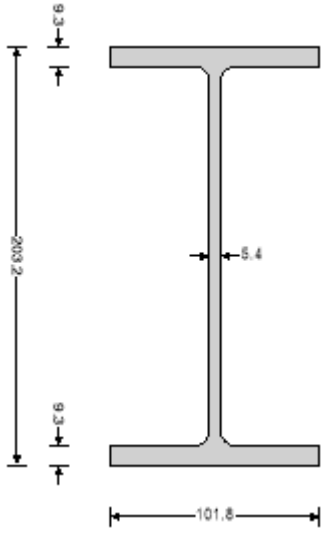
Maximum moment	M_{max} = **44.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **50** kN	V_{min} = **-50** kN
Deflection	δ_{max} = **9.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **50** kN	R_{A_min} = **50** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **21.5** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **14** kN	
Maximum reaction at support B	R_{B_max} = **50** kN	R_{B_min} = **50** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **21.5** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **14** kN	

Section details

Section type	**UKB 203x102x23 (Tata Steel Advance)**
Steel grade	**S275**

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

Nominal thickness of element	t = max(t_f, t_w) = **9.3** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **0.92**

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

Width of section: $c = d =$ **169.4** mm

$c / t_w = 33.9 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2 =$ **40.6** mm

$c / t_f = 4.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f =$ **184.6** mm

Shear area factor: $\eta =$ **1.000**

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

Shear buckling resistance can be ignored

Design shear force: $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) =$ **50** kN

Shear area - cl 6.2.6(3): $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1238** mm^2

Design shear resistance - cl 6.2.6(2): $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **196.6** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment: $M_{Ed} = \max(abs(M_{s1_max}), abs(M_{s1_min})) =$ **44.1** kNm

Design bending resistance moment - eq 6.13: $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **64.4** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection: $\delta_{lim} = L_{s1} / 360 =$ **9.8** mm

Maximum deflection span 1: $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) =$ **9.187** mm

PASS - Maximum deflection does not exceed deflection limit

MASONRY BEARING DESIGN

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

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	50.0 kN	87.2 kN	N/A	N/A	0.574	Pass

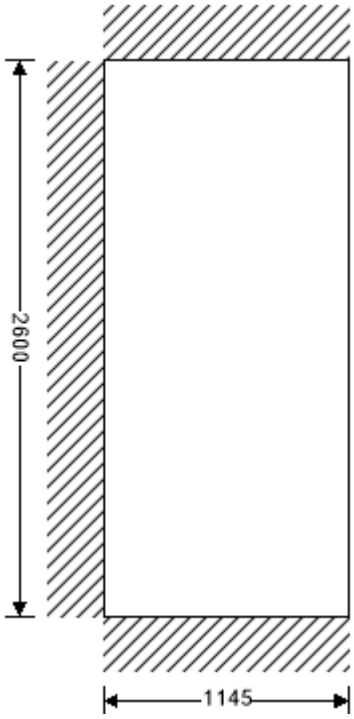
Masonry panel details

Panel length — L = **1145** mm

Panel height — h = **2600** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints — $L < 15 \times t_{ef}$ (Wall restrained at the top and bottom and stiffened on one vertical edge (with one free edge))

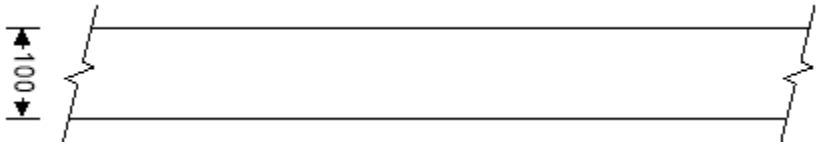
Horizontal restraints — **Simply supported**

Reduction factor - eq. 5.5 — ρ_2 = **1.00**

Reduction factor - eq. 5.6 — $\rho_3 = 1/(1 + [(\rho_2 \times h) / (3 \times L)]^2) \times \rho_2$ = **0.64**

Effective height of wall - eq. 5.2 — $h_{ef} = \rho_3 \times h$ = **1653** mm

Wall construction details



Wall type	**Single leaf panel**
Overall wall thickness	t = **100.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness	t_{ef} = t = **100.0** mm

Masonry material details

Unit type	**Clay - Group 1**
Compressive strength of masonry unit	f_c = **50** N/mm^2
Height of unit	h_u = **65** mm
Width of unit	w_u = **100** mm
Conditioning factor	k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1	d_{sf} = **0.85**
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf}$ = **42.5** N/mm^2
Specific weight of units	γ = **18** kN/m^3
Mortar type	**M4 - General Purpose**
Compressive strength of mortar	f_m = **4.0** N/mm^2
Compressive strength factor - Tbl. NA 4	K = **0.50**
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **10.46** N/mm^2
Short term secant modulus of elasticity factor	K_E = **1000**
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k$ = **10458** N/mm^2

Design compressive strength of masonry

Category of manufacturing control	**Category II**
Class of execution control	**Class 2**
Partial factor for compressive strength	γ_M = **3.00**
Cross-sectional area of wall	A = L × t = **0.11** m^2
Design compressive strength of masonry	$f_d = f_k / \gamma_M$ = **3.49** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = **1.35**
Partial safety factor for variable load	γ_{fQ} = **1.50**

Superimposed vertical loading details

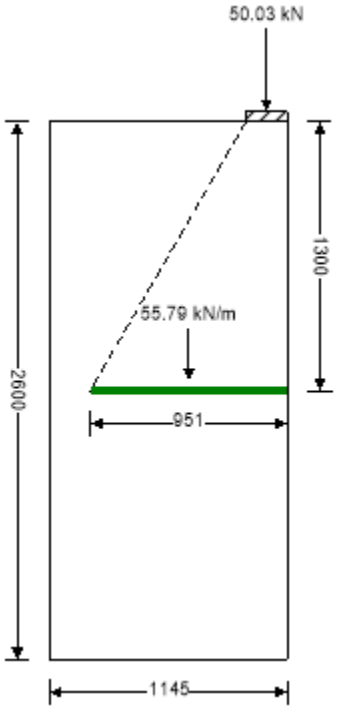
Permanent UDL at top of wall	g_k = **0.00** kN/m
Variable UDL at top of wall	q_k = **0.00** kN/m
Eccentricity of permanent UDL load	e_{gu} = **0** mm
Eccentricity of variable UDL load	e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	λ_{lim} = **27**
Slenderness ratio	$\lambda = h_{ef} / t_{ef}$ = **16.5**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	G_{kc1} = **21.50** kN
Variable concentrated load	Q_{kc1} = **14.00** kN
Eccentricity of concentrated load	e_{c1} = **0** mm
Length of concentrated load	L_{c1} = **200** mm
Width of concentrated load	w_{c1} = **100** mm
Height of concentrated load	h_{c1} = **2600** mm
Distance of load to right vertical edge	r_{11} = **0** mm
Distance of load to nearest vertical edge	a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing	$A_{b1} = L_{c1} \times w_{c1}$ = **20000** mm²
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11}$ = **951** mm
Effective bearing area	$A_{ef1} = l_{efm1} \times t$ = **95056** mm²
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.21**
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0)$ = **1.27**
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5)$ = **1.25**
Enhancement factor for concentrated loads	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1})$ = **1.25**
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **50.03** kN
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **87.15** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

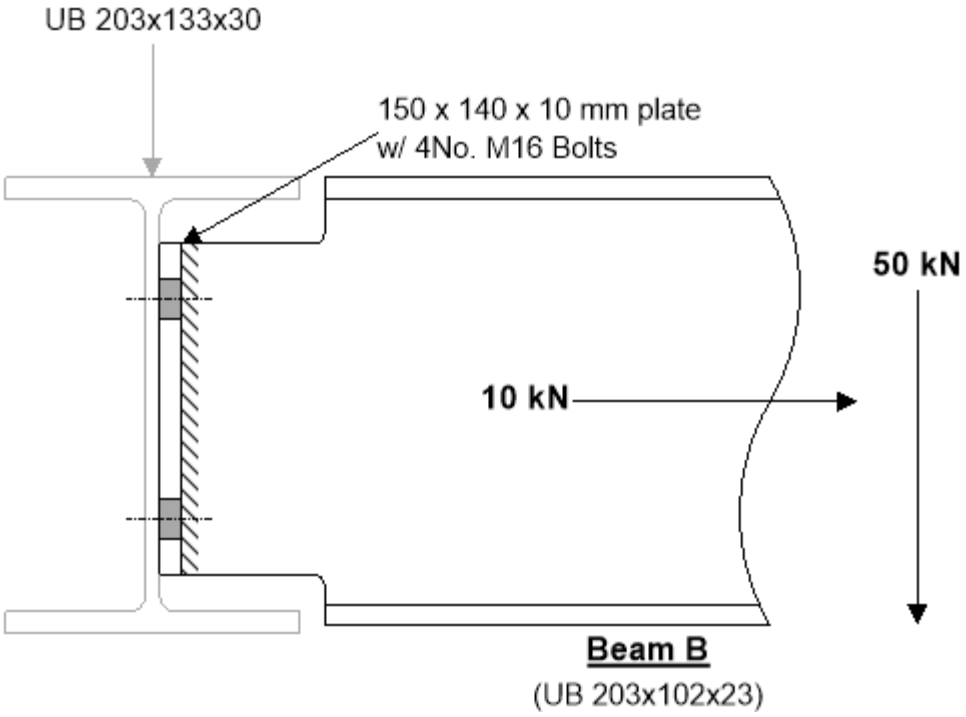
	$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) =$ **0.0** mm
Eccentricity of concentrated load at mid-height	$e_{mc1} = e_{c1} / 2 =$ **0.0** mm
Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h_{ef} / 450 =$ **3.7** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1} =$ **52.63** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} =$ **55.79** kN/m
Design moment at mid-height	$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} =$ **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} =$ **3.7** mm
Slenderness ratio limit for creep eccentricity	$\lambda_c =$ **27**
Eccentricity due to creep	$e_{k1} =$ **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) =$ **5.0** mm
From eq. G2	$A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.90**
From eq. G3	$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **0.68**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.71**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **248.20** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

STEEL CONNECTION DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK National Annex.

Tedds calculation version 1.2.04



Connection details

Connection type	**Partial depth end plate**
Number of supported beams	**1 supported beam**

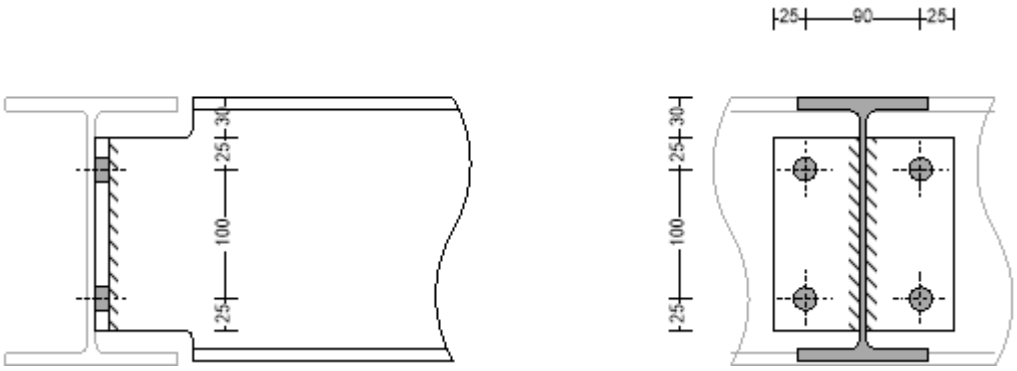
Partial factors

Resistance of cross-section	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Cross-sections in tension to fracture	$\gamma_{M2,c}$ = **1.10**
Resistance of bolts	$\gamma_{M2,b}$ = **1.25**
Structural integrity	$\gamma_{M,u}$ = **1.10**

Supporting beam details

Section name	**UB 203x133x30**
Steel grade	**S275**
Yield strength	f_y = **275** N/mm^2
Ultimate strength	f_u =**410** N/mm^2

Supported Beam B



Summary Results

Check	Description	Units	Design Force	Design Resistance	Utilisation	
1	Recommended detailing practices					PASS
2	Supported beam - Welds	kN	25.5	212.7	0.120	PASS
4	Supported beam - Web in shear	kN	50	115.7	0.432	PASS
5	Supported beam - Resistance at notch	kNm	3.8	5.6	0.673	PASS
6	Supported beam - Local stability notch					PASS
8	Connection - Bolt group	kN	50	192.9	0.259	PASS
9	Connection - End plate in shear	kN	50	375.1	0.133	PASS
10	Supporting beam - Shear	kN	25	203.2	0.123	PASS
11	Tying resistance - Plate and bolts	kN	10	170	0.059	PASS
12	Tying resistance - Supported beam web	kN	10	301.9	0.033	PASS

Design forces

Design shear V_{Ed1} = **50** kN

Design tying force F_{Ed1} = **10** kN

Supported beam details

Section name **UB 203x102x23**

Steel grade **S275**

Yield strength $f_{y,b}$ = **275** N/mm^2

Ultimate strength $f_{u,b}$ = **410** N/mm^2

Correlation factor $\beta_{w,b}$ = **0.85**

End plate details

Plate height h_p = **150** mm

Plate width b_p = **140** mm

Plate thickness t_p = **10** mm

Plate grade **S275**

Yield strength $f_{y,p}$ = **275** N/mm^2

Ultimate strength $f_{u,p}$ = **410** N/mm^2

Correlation factor	$\beta_{w,p}$ = **0.85**
Bolt details	
Number of bolt rows	$n_{1,1}$ = **2**
Total number of bolts	n_b = **4**
End distance	e_1 = **25** mm
Edge distance	e_2 = **25** mm
Pitch	p_1 = **100** mm
Gauge	p_3 = **90** mm
Bolt hole	d_0 = **18** mm
Bolt size	**M16**
Bolt grade	**8.8**
Yield strength	$f_{y,bolt}$ = **640** N/mm^2
Ultimate strength	$f_{u,bolt}$ = **800** N/mm^2
Check 1: Recommended detailing practice	
Minimum plate height	0.6 × h_b = **121.9** mm
Actual plate height	h_p = **150** mm
Maximum depth to plate	**50 mm**
Actual depth to plate	d_p = **30** mm
Maximum plate thickness	**10 mm**
Actual plate thickness	t_p = **10** mm
Minimum bolt gauge	**90 mm**
Actual bolt gauge	p_3 = **90** mm
Top Notch	
Depth of notch	d_{nt} = **30** mm
Length of notch	l_n = **65** mm
Minimum vertical clearance	Max($t_{f,b}$ + r_b, t_f + r) = **17.2** mm
Actual vertical clearance	d_{nt} = **30** mm
Minimum horizontal clearance	**10 mm**
Actual horizontal clearance	l_n - (b - t_w) / 2 + t_p = **11.2** mm
Bottom Notch	
Depth of notch	d_{nb} = **23.2** mm
Length of notch	l_n = **65** mm
Minimum vertical clearance	t_{f,b,A_c1} + r_b = **16.9** mm
Actual vertical clearance	d_{nb} = **23.2** mm

PASS - Recommended detailing practices are met

Check 2: Supported beam - Welds	
Weld leg size	s_w = **8.0** mm
Minimum weld throat thickness	0.4 × $t_{w,b}$ = **2.2** mm
Effective weld throat thickness	a_w = 0.7 × s_w = **5.6** mm
Correlation factor	β_w = Min($\beta_{w,b}$, $\beta_{w,p}$) = **0.85**
Design shear strength	$f_{vw,d}$ = Min($f_{u,b}$,$f_{u,p}$) / √(3) / (β_w × $\gamma_{M2,c}$) = **253.17** N/mm^2
Design resistance	$F_{w,Rd}$ = $f_{vw,d}$ × a_w × h_p = **212.66** kN
Design weld force	$F_{w,Ed}$ = √(V_{Ed1}^2 + F_{Ed1}^2) / 2 = **25.5** kN
Utilisation	$F_{w,Ed}$ / $F_{w,Rd}$ = **0.120**

PASS - Weld throat thickness greater than required

Check 4: Supported beam - Web in shear

Shear area	$A_v = 0.9 \times h_p \times t_{w,b} =$ **729** mm²
Plastic shear resistance of beam web	$V_{pl,Rd} = A_v \times (f_{y,b} / \sqrt{(3)}) / \gamma_{M0} =$ **115.74** kN
Design shear resistance	$V_{c,Rd} = V_{pl,Rd} =$ **115.74** kN
Utilisation	$V_{Ed1} / V_{c,Rd} =$ **0.432**

PASS - Web shear resistance greater than design shear

Check 5: Supported beam - Resistance at notch

Double Notch (low shear, $V_{Ed} <= 0.5V_{pl,DN,Rd}$)

Shear area at notch	$A_{v,DN} = 0.9 \times (h_b - d_{nt} - d_{nb}) \times t_{w,b} =$ **729** mm²
Shear resistance at notch	$V_{pl,DN,Rd} = (A_{v,DN} \times f_{y,b}) / (\sqrt{(3)} \times \gamma_{M0}) =$ **115.74** kN
Moment resistance at notch	$M_{v,DN,Rd} = f_{y,b} \times t_{w,b} / (6 \times \gamma_{M0}) \times (h_b - d_{nt} - d_{nb})^2 =$ **5.57** kNm
Design moment at notch	$M_{v,Ed} = V_{Ed1} \times (t_p + l_n) =$ **3.75** kNm
Utlilisation	$M_{v,Ed} / M_{v,N,Rd} = 0.673$

PASS - Notch resistance is greater than design force

Check 6: Supported beam - Local stability of notched beam

Double notch

Maximum notch depth	$h_b / 5 =$ **40.6** mm
Actual notch depth	$Max(d_{nt}, d_{nb}) =$ **30** mm
Maximum notch length ($h_b/t_{w,b} <= 54.3$)	$h_b =$ **203.2** mm
Actual notch length	$l_n =$ **65** mm

PASS - Local stability is accounted for

Check 8: Connection - Bolt group

Bolt tensile stress area	$A_s =$ **245** mm²
Bolt shear stress factor	$\alpha_v =$ **0.6**
Bolt shear resistance	$F_{v,Rd} = \alpha_v \times f_{u,bolt} \times A_s / \gamma_{M2,b} =$ **94.08** kN
For the end plate	$\alpha_{b,p} = Min(e_1 / (3 \times d_0), p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_{u,p}, 1) =$ **0.46**
	$k_{1,p} = Min(2.8 \times e_2 / d_0 - 1.7, 1.4 \times p_3 / d_0 - 1.7, 2.5) =$ **2.19**
For the supporting member	$\alpha_{b,2} = Min(p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_u, 1) =$ **1**
	$k_{1,2} = Min(1.4 \times p_3 / d_0 - 1.7, 2.5) =$ **2.5**
Bearing on the end plate	$F_{b,Rd,p} = k_{1,p} \times \alpha_{b,p} \times f_{u,p} \times d_b \times t_p / \gamma_{M2,b} =$ **53.18** kN
Bearing on the supporting member	$F_{b,Rd,2} = k_{1,2} \times \alpha_{b,2} \times f_u \times d_b \times t_w / \gamma_{M2,b} =$ **83.97** kN
Minimum bearing resistance	$F_{b,Rd1} = Min(F_{b,Rd,p}, F_{b,Rd,2}) =$ **53.18** kN
Resistance of the bolt group	$F_{Rd} = n_b \times F_{b,Rd1} =$ **212.73** kN
Utilisation	$V_{Ed1} / F_{Rd} =$ **0.235**

PASS - Bolt group resistance is greater than design force

Check 9: Connection - End plate in shear

Net shear area	$A_{v,net} = t_p \times (h_p - n_{1,1} \times d_0) =$ **1140** mm²
Edge shear area	$A_{nt} = t_p \times (e_2 - d_0 / 2) =$ **160** mm²
Shear area from end bolt	$A_{nv} = t_p \times (h_p - e_1 - (n_{1,1} - 0.5) \times d_0) =$ **980** mm²
Gross section shear resistance	$V_{Rd,g} = (2 \times h_p \times t_p) / 1.27 \times f_{y,p} / (\sqrt{(3)} \times \gamma_{M0}) =$ **375.05** kN
Net section shear resistance	$V_{Rd,n} = 2 \times A_{v,net,A_c1} \times f_{u,plate,A_c1} / (\sqrt{(3)} \times \gamma_{M2,c}) =$ **490.64** kN
Block tearing resistance	$V_{Rd,b} = 2 \times (f_{u,p} \times A_{nt} / \gamma_{M2,c} + f_{y,p} \times A_{nv} / (\sqrt{(3)} \times \gamma_{M0})) =$ **430.46** kN
End plate in-plane bending resistance	$h_p < 1.36p_3$ - No additional requirements
End plate shear resistance	$V_{Rd,pl,min} = Min(V_{Rd,g}, V_{Rd,n}, V_{Rd,b}) =$ **375.05** kN

Utilisation	_PlateShearUtilisationA_c1 = **0.133**
	PASS - Shear resistance of end plate greater than design force

Check 10: Supporting beam - Shear

Distance from top bolt to flange	$e_{1,t}$ = **55** mm
Distance from bottom bolt to flange	$e_{1,b}$ = **52** mm
Minimum top distance	$e_t = \text{Min}(e_{1,t}, 5 \times d_b)$ = **55** mm
Minimum bottom distance	$e_b = \text{Min}(e_{1,b}, p_3 / 2, 5 \times d_b)$ = **45** mm
Shear area of supporting member	$A_v = t_w \times (e_t + (n_{1,1} - 1) \times p_1 + e_b)$ = **1280** mm^2
Net shear area of supporting member	$A_{v,net} = A_v - n_{1,1} \times d_0 \times t_w$ = **1050** mm^2
Local shear resistance	$V_{Rd,min} = \text{Min}(A_v \times f_y / (\sqrt{(3)} \times \gamma_{M0}), A_{v,net} \times f_u / (\sqrt{(3)} \times \gamma_{M2,c}))$ = **203.23** kN
Utilisation	$V_{Ed1}/2 / V_{Rd,min}$ = **0.123**
	PASS - Beam shear resistance is greater than design force

Check 11: Tying resistance - Plate and bolts

Effective end distance	$e_{1A} = \text{Min}(e_1, 0.5 \times (p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)}) + d_0/2)$ = **25** mm
Effective bolt pitch	$p_{1A} = \text{Min}(p_1, p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)} + d_0)$ = **87** mm
Minimum end distance	$e_{min} = e_2$ = **25** mm
Bolt factor	k_2 = **0.9**
Distance from weld throat to bolt	$m_w = (p_3 - t_{w,b} - 2 \times 0.8 \times a_w \times \sqrt{(2)}) / 2$ = **36** mm
	$n_w = \text{Min}(e_{min}, 1.25 \times m_w)$ = **25** mm
Width across bolt head points	d_w = **33** mm
	$e_w = d_w / 4$ = **8.3** mm
Effective length of equivalent T-stub	$\Sigma l_{eff} = 2 \times e_{1A} + (n_{1,1} - 1) \times p_{1A}$ = **136.8** mm
Moment resistance of plate	$M_{pl,1,Rd,u} = (0.25 \times \Sigma l_{eff} \times t_p^2 \times f_{u,p}) / \gamma_{M,u}$ = **1.27** kNm
	$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = **1.27** kNm
Mode 1 plate failure	$F_{Rd,u,1} = (8 \times n_w - 2 \times e_w) \times M_{pl,1,Rd,u} / (2 \times m_w \times n_w - e_w \times (m_w + n_w))$ = **180.54** kN
Individual bolt resistance	$F_{t,Rd,u} = k_2 \times f_{u,bolt} \times A_s / \gamma_{M,u}$ = **160.36** kN
Group bolt resistance	$\Sigma F_{t,Rd,u} = n_b \times F_{t,Rd,u}$ = **641.45** kN
Mode 2 bolt and plate failure	$F_{Rd,u,2} = (2 \times M_{pl,2,Rd,u} + n_w \times \Sigma F_{t,Rd,u}) / (m_w + n_w)$ = **304.85** kN
Mode 3 bolt failure	$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = **641.45** kN
Minimum resistance	$F_{Rd,u,min} = \text{Min}(F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$ = **180.54** kN
Utilisation	$F_{Ed1} / F_{Rd,u,min}$ = **0.055**
	PASS - Tying resistance of plate and bolts is greater than design force

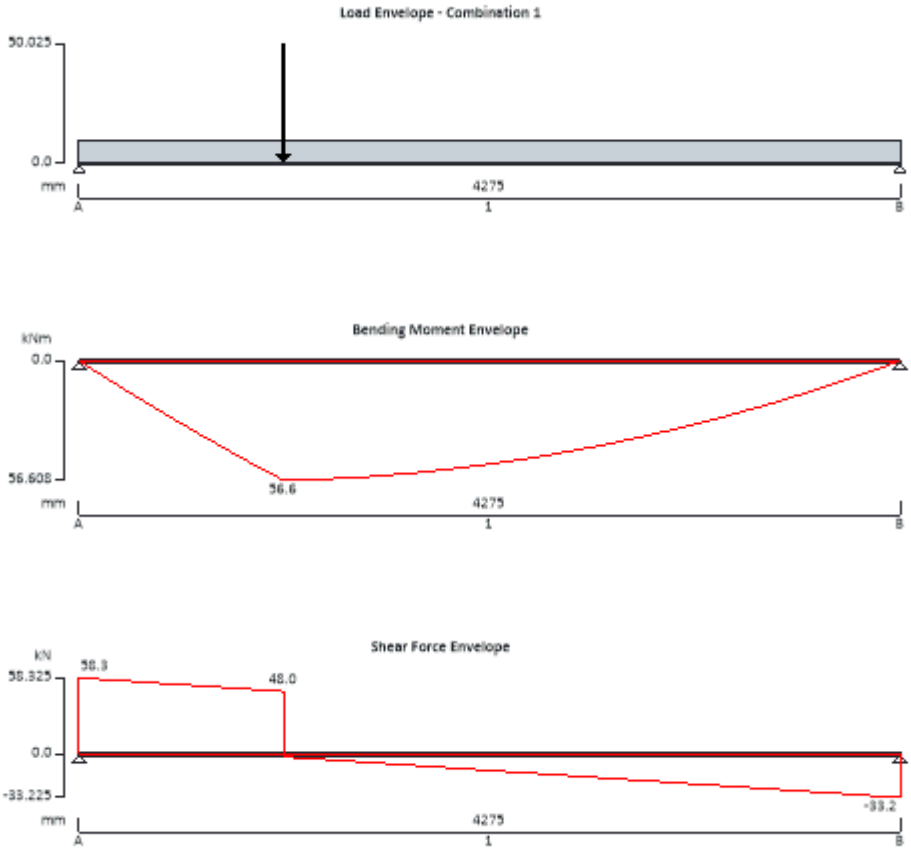
Check 12: Tying resistance - Supported beam web

Web resistance	$F_{Rd,u} = (t_{w,b} \times h_p \times f_{u,b}) / \gamma_{M,u}$ = **301.91** kN
Utilisation	$F_{Ed1} / F_{Rd,u}$ = **0.033**
	PASS - Supported beam web tying resistance is greater than design force

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 3.0.15



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 5.99 kN/m
	Variable full UDL 0.82 kN/m
	Permanent point load 21.5 kN at 1065 mm
	Variable point load 14 kN at 1065 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50

	Support B	Permanent × 1.35 Variable × 1.50

Analysis results

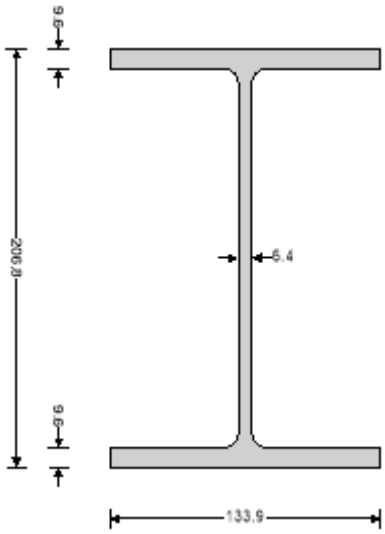
Maximum moment	M_{max} = **56.6** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1_seg1_max}$ = **56.6** kNm	$M_{s1_seg1_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1_seg2_max}$ = **56.6** kNm	$M_{s1_seg2_min}$ = **0** kNm
Maximum shear	V_{max} = **58.3** kN	V_{min} = **-33.2** kN
Maximum shear span 1 segment 1	$V_{s1_seg1_max}$ = **58.3** kN	$V_{s1_seg1_min}$ = **-2** kN
Maximum shear span 1 segment 2	$V_{s1_seg2_max}$ = **0** kN	$V_{s1_seg2_min}$ = **-33.2** kN
Deflection segment 3	δ_{max} = **11.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **58.3** kN	R_{A_min} = **58.3** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **29.6** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **12.3** kN	
Maximum reaction at support B	R_{B_max} = **33.2** kN	R_{B_min} = **33.2** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **18.8** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **5.2** kN	

Section details

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

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

Nominal thickness of element	t = max(t_f, t_w) = **9.6** mm
Nominal yield strength	f_y = **275** N/mm²
Nominal ultimate tensile strength	f_u = **410** N/mm²
Modulus of elasticity	E = **210000** N/mm²



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports plus 1065 mm

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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.92**

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

Width of section: $c = d$ = **172.4** mm

$c / t_w = 29.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2$ = **56.2** mm

$c / t_f = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f$ = **187.6** mm

Shear area factor: η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force: $V_{Ed} = \max(abs(V_{max}), abs(V_{min}))$ = **58.3** 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** mm^2

Design shear resistance - cl 6.2.6(2): $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **231.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 2 major (y-y) axis - Section 6.2.5

Design bending moment: $M_{Ed} = \max(abs(M_{s1_seg2_max}), abs(M_{s1_seg2_min}))$ = **56.6** kNm

Design bending resistance moment - eq 6.13: $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **86.5** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: k_c = **0.861**

$C_1 = 1 / k_c^2$ = **1.349**

Curvature factor: $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.931**

Poissons ratio: ν = **0.3**

Shear modulus: $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length: $L = 1.0 \times L_{s1_seg2}$ = **3210** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **160.4** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.734**

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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.759**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.852**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.931}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.915}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{79.1}$ 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 $\delta_{lim} = L_{s1} / 360 = \mathbf{11.9}$ mm

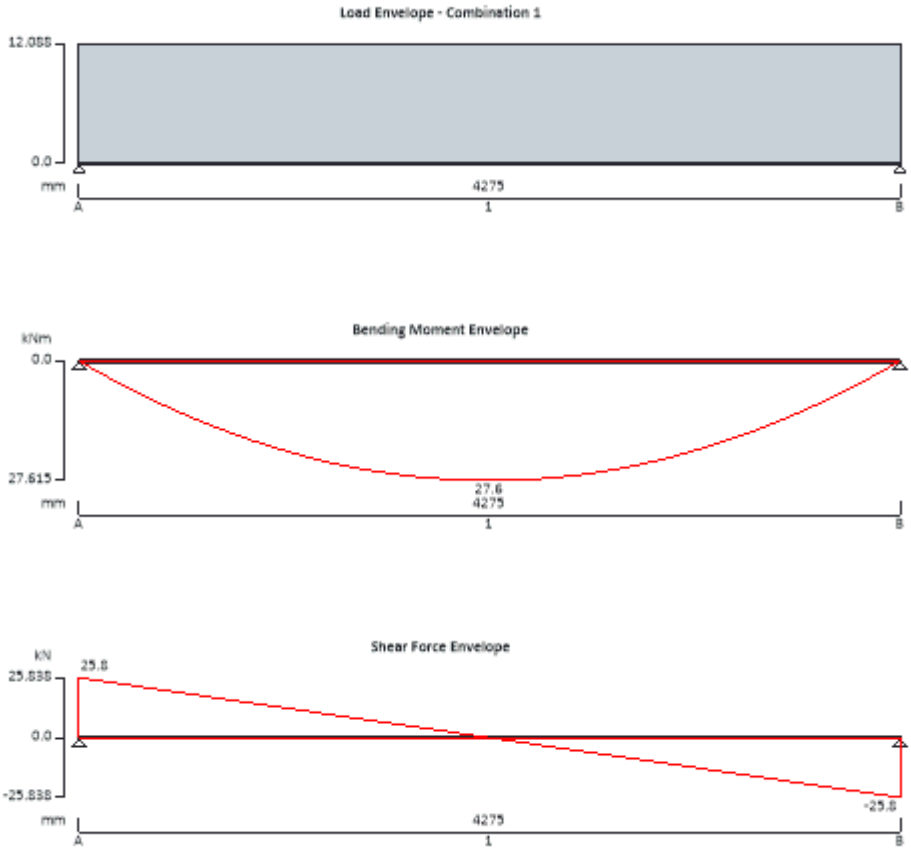
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{11.659}$ mm

PASS - Maximum deflection does not exceed deflection limit

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 3.0.15



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 7.16 kN/m
	Variable full UDL 1.35 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

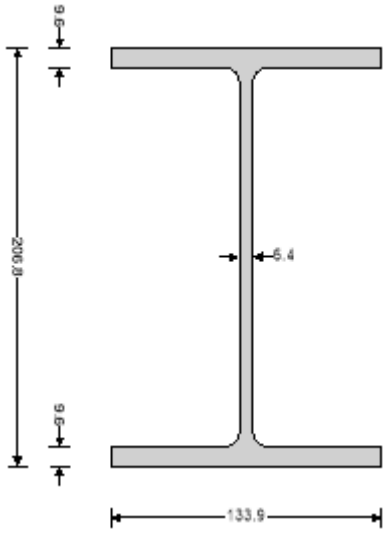
Maximum moment	M_{max} = **27.6** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1_seg1_max}$ = **24.5** kNm	$M_{s1_seg1_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1_seg2_max}$ = **27.6** kNm	$M_{s1_seg2_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1_seg3_max}$ = **24.5** kNm	$M_{s1_seg3_min}$ = **0** kNm
Maximum shear	V_{max} = **25.8** kN	V_{min} = **-25.8** kN
Maximum shear span 1 segment 1	$V_{s1_seg1_max}$ = **25.8** kN	$V_{s1_seg1_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1_seg2_max}$ = **8.6** kN	$V_{s1_seg2_min}$ = **-8.6** kN
Maximum shear span 1 segment 3	$V_{s1_seg3_max}$ = **0** kN	$V_{s1_seg3_min}$ = **-25.8** kN
Deflection segment 4	δ_{max} = **6.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **25.8** kN	R_{A_min} = **25.8** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **15.9** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **2.9** kN	
Maximum reaction at support B	R_{B_max} = **25.8** kN	R_{B_min} = **25.8** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **15.9** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **2.9** kN	

Section details

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

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

Nominal thickness of element	t = max(t_f, t_w) = **9.6** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports plus third points

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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.92**

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

Width of section: $c = d$ = **172.4** mm

$c / t_w = 29.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2$ = **56.2** mm

$c / t_f = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f$ = **187.6** mm

Shear area factor: η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force: $V_{Ed} = \max(abs(V_{max}), abs(V_{min}))$ = **25.8** 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** mm^2

Design shear resistance - cl 6.2.6(2): $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **231.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 2 major (y-y) axis - Section 6.2.5

Design bending moment: $M_{Ed} = \max(abs(M_{s1_seg2_max}), abs(M_{s1_seg2_min}))$ = **27.6** kNm

Design bending resistance moment - eq 6.13: $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **86.5** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: k_c = **0.99**

$C_1 = 1 / k_c^2$ = **1.02**

Curvature factor: $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.931**

Poissons ratio: ν = **0.3**

Shear modulus: $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length: $L = 1.0 \times L_{s1_seg2}$ = **1425** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **468.1** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.43**

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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.574**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.988**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.996}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.992}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{85.8}$ 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 $\delta_{lim} = L_{s1} / 360 = \mathbf{11.9}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{6.297}$ mm

PASS - Maximum deflection does not exceed deflection limit

MASONRY BEARING DESIGN

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

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	59.0 kN	83.7 kN	N/A	N/A	0.705	Pass

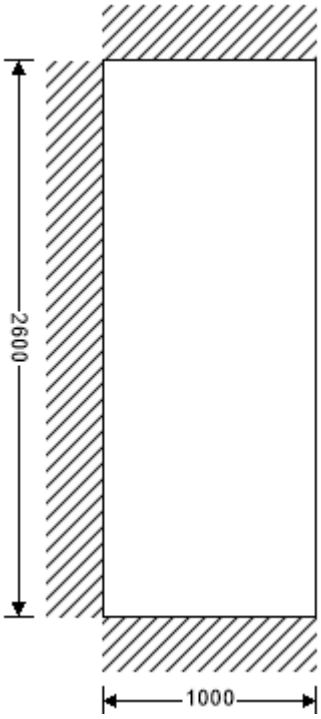
Masonry panel details

Panel length — L = **1000** mm

Panel height — h = **2600** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints — L < 15 × t_{ef} (Wall restrained at the top and bottom and stiffened on one vertical edge (with one free edge))

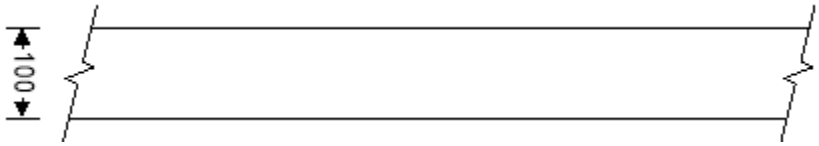
Horizontal restraints — **Simply supported**

Reduction factor - eq. 5.5 — ρ_2 = **1.00**

Reduction factor - eq. 5.6 — $\rho_3 = 1/(1 + [(\rho_2 \times h) / (3 \times L)]^2) \times \rho_2$ = **0.57**

Effective height of wall - eq. 5.2 — $h_{ef} = \rho_3 \times h$ = **1485** mm

Wall construction details



Wall type	**Single leaf panel**
Overall wall thickness	t = **100.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness	t_{ef} = t = **100.0** mm

Masonry material details

Unit type	**Clay - Group 2**
Compressive strength of masonry unit	f_c = **50** N/mm^2
Height of unit	h_u = **65** mm
Width of unit	w_u = **100** mm
Conditioning factor	k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1	d_{sf} = **0.85**
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf}$ = **42.5** N/mm^2
Specific weight of units	γ = **18** kN/m^3
Mortar type	**M4 - General Purpose**
Compressive strength of mortar	f_m = **4.0** N/mm^2
Compressive strength factor - Tbl. NA 4	K = **0.40**
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **8.37** N/mm^2
Short term secant modulus of elasticity factor	K_E = **1000**
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k$ = **8367** N/mm^2

Design compressive strength of masonry

Category of manufacturing control	**Category II**
Class of execution control	**Class 2**
Partial factor for compressive strength	γ_M = **3.00**
Cross-sectional area of wall	A = L × t = **0.10** m^2
Design compressive strength of masonry	$f_d = f_k / \gamma_M$ = **2.79** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = **1.35**
Partial safety factor for variable load	γ_{fQ} = **1.50**

Superimposed vertical loading details

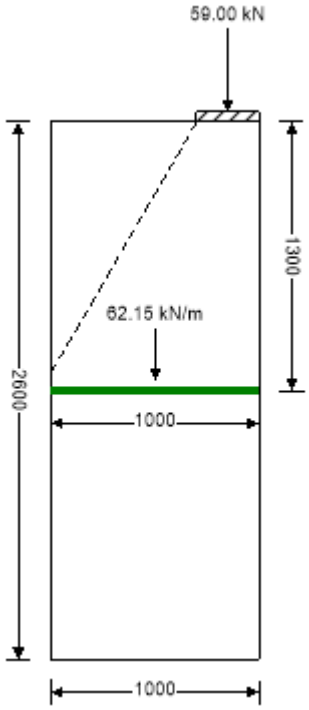
Permanent UDL at top of wall	g_k = **0.00** kN/m
Variable UDL at top of wall	q_k = **0.00** kN/m
Eccentricity of permanent UDL load	e_{gu} = **0** mm
Eccentricity of variable UDL load	e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	λ_{lim} = **27**
Slenderness ratio	$\lambda = h_{ef} / t_{ef}$ = **14.8**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	G_{kc1} = **34.70** kN
Variable concentrated load	Q_{kc1} = **8.10** kN
Eccentricity of concentrated load	e_{c1} = **0** mm
Length of concentrated load	L_{c1} = **300** mm
Width of concentrated load	w_{c1} = **100** mm
Height of concentrated load	h_{c1} = **2600** mm
Distance of load to right vertical edge	r_{11} = **0** mm
Distance of load to nearest vertical edge	a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing	$A_{b1} = L_{c1} \times w_{c1}$ = **30000** mm^2
Effective length of bearing at mid-height	l_{efm1} = L = **1000** mm
Effective bearing area	$A_{ef1} = l_{efm1} \times t$ = **100000.00** mm^2
Bearing area ratio check	A_{ratio1} = Min(A_{b1} / A_{ef1}, 0.45) = **0.30**
Enhancement factor - cl.6.1.3(3)	β_1 = **1.00**
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **59.00** kN
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **83.67** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of concentrated load at mid-height $e_{mc1} = e_{c1} / 2$ = **0.0** mm

Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h_{ef} / 450 =$ **3.3** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1} =$ **59.00** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} =$ **62.15** kN/m
Design moment at mid-height	$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} =$ **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} =$ **3.3** mm
Slenderness ratio limit for creep eccentricity	$\lambda_c =$ **27**
Eccentricity due to creep	$e_{k1} =$ **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) =$ **5.0** mm
From eq. G2	$A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.90**
From eq. G3	$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **0.61**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.75**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **208.97** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

MASONRY BEARING DESIGN

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

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	84.2 kN	174.8 kN	N/A	N/A	0.482	Pass

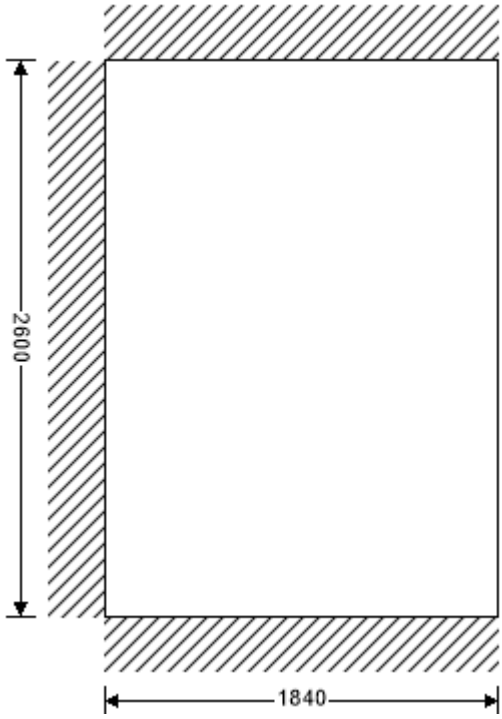
Masonry panel details

Panel length — L = **1840** mm

Panel height — h = **2600** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints — $L < 15 \times t_{ef}$ (Wall restrained at the top and bottom and stiffened on one vertical edge (with one free edge))

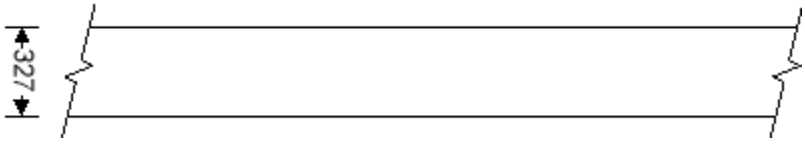
Horizontal restraints — **Simply supported**

Reduction factor - eq. 5.5 — ρ_2 = **1.00**

Reduction factor - eq. 5.6 — $\rho_3 = 1/(1 + [(\rho_2 \times h) / (3 \times L)]^2) \times \rho_2$ = **0.82**

Effective height of wall - eq. 5.2 — $h_{ef} = \rho_3 \times h$ = **2128** mm

Wall construction details



Wall type	**Single leaf panel**
Overall wall thickness	t = **327.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness	t_{ef} = t = **327.0** mm

Masonry material details

Unit type	**Aggregate concrete - Group 2**
Compressive strength of masonry unit	f_c = **7.3** N/mm^2
Height of unit	h_u = **215** mm
Width of unit	w_u = **100** mm
Conditioning factor	k = **1.0**
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	d_{sf} = **1.38**
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf}$ = **10.07** N/mm^2
Specific weight of units	γ = **18** kN/m^3
Mortar type	**M4 - General Purpose**
Compressive strength of mortar	f_m = **4.0** N/mm^2
Compressive strength factor - Tbl. NA 4	K = **0.70**
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **5.35** N/mm^2
Short term secant modulus of elasticity factor	K_E = **1000**
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k$ = **5345** N/mm^2

Design compressive strength of masonry

Category of manufacturing control	**Category II**
Class of execution control	**Class 2**
Partial factor for compressive strength	γ_M = **3.00**
Cross-sectional area of wall	$A = L \times t$ = **0.60** m^2
Design compressive strength of masonry	$f_d = f_k / \gamma_M$ = **1.78** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = **1.35**
Partial safety factor for variable load	γ_{fQ} = **1.50**

Superimposed vertical loading details

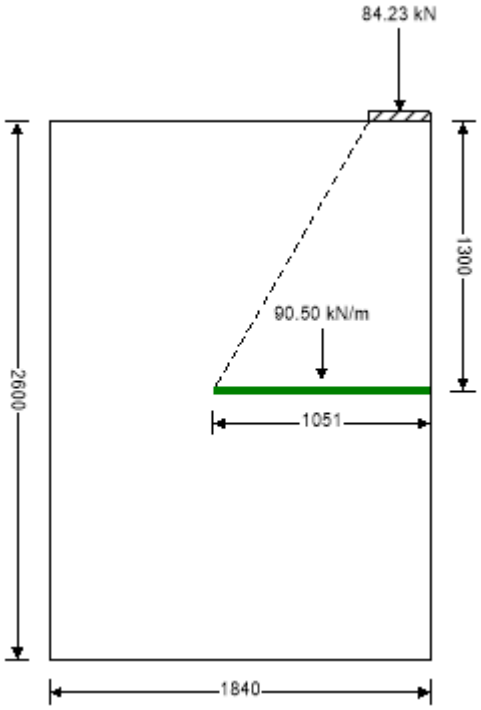
Permanent UDL at top of wall	g_k = **0.00** kN/m
Variable UDL at top of wall	q_k = **0.00** kN/m
Eccentricity of permanent UDL load	e_{gu} = **0** mm
Eccentricity of variable UDL load	e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	λ_{lim} = **27**
Slenderness ratio	$\lambda = h_{ef} / t_{ef}$ = **6.5**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	G_{kc1} = **45.50** kN
Variable concentrated load	Q_{kc1} = **15.20** kN
Eccentricity of concentrated load	e_{c1} = **0** mm
Length of concentrated load	L_{c1} = **300** mm
Width of concentrated load	w_{c1} = **327** mm
Height of concentrated load	h_{c1} = **2600** mm
Distance of load to right vertical edge	r_{11} = **0** mm
Distance of load to nearest vertical edge	a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing	$A_{b1} = L_{c1} \times w_{c1}$ = **98100** mm²
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11}$ = **1051** mm
Effective bearing area	$A_{ef1} = l_{efm1} \times t$ = **343531.60** mm²
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.29**
Enhancement factor - cl.6.1.3(3)	β_1 = **1.00**
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **84.23** kN
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **174.79** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of concentrated load at mid-height $e_{mc1} = e_{c1} / 2$ = **0.0** mm

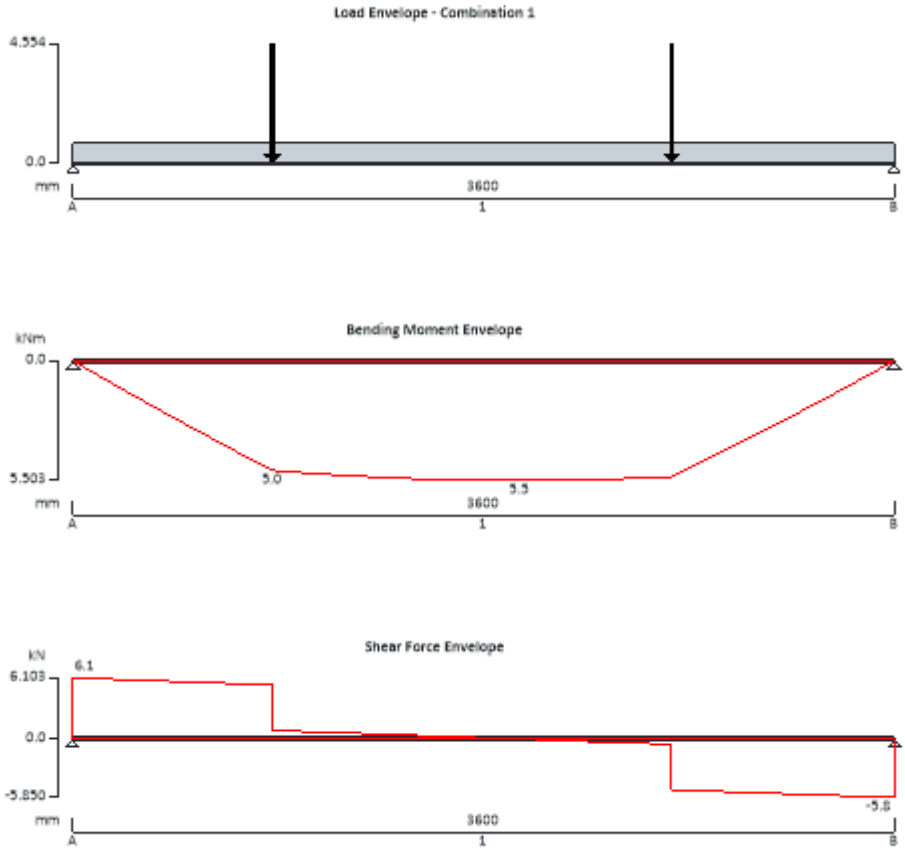
Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h_{ef} / 450 =$ **4.7** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1} =$ **80.17** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} =$ **90.50** kN/m
Design moment at mid-height	$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} =$ **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} =$ **4.7** mm
Slenderness ratio limit for creep eccentricity	$\lambda_c =$ **27**
Eccentricity due to creep	$e_{k1} =$ **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) =$ **16.3** mm
From eq. G2	$A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.90**
From eq. G3	$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **0.21**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.88**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **512.64** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

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 1.7.05



Applied loading

Beam loads

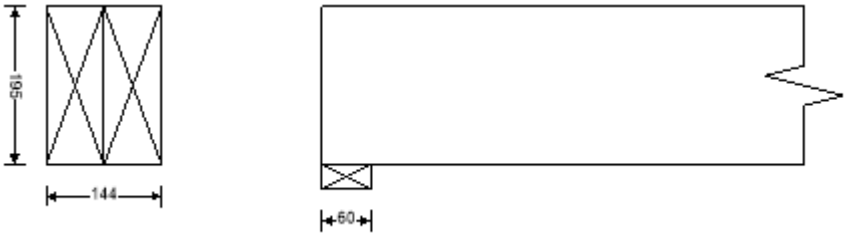
Permanent self weight of beam × 1
Permanent point load 1.840 kN at 875 mm
Permanent point load 1.840 kN at 2625 mm
Variable point load 1.380 kN at 875 mm
Variable point load 1.380 kN at 2625 mm
Permanent full UDL 0.300 kN/m
Variable full UDL 0.170 kN/m

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

Maximum moment	M_{max} = **5.503** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **5.503** kNm	
Maximum shear	F_{max} = **6.103** kN	F_{min} = **-5.850** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **6.103** kN	
Total load on beam	W_{tot} = **11.952** kN	
Reactions at support A	R_{A_max} = **6.103** kN	R_{A_min} = **6.103** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **2.605** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **1.724** kN	
Reactions at support B	R_{B_max} = **5.850** kN	R_{B_min} = **5.850** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **2.502** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **1.648** kN	



Timber section details

Breadth of timber sections	b = **72** mm
Depth of timber sections	h = **195** mm
Number of timber sections in member	N = **2**
Overall breadth of timber member	b_b = N × b = **144** mm
Timber strength class - EN 338:2016 Table 1	**C24**

Member details

Load duration - cl.2.3.1.2	**Long-term**
Service class of timber - cl.2.3.1.3	**1**
Length of span	L_{s1} = **3600** mm
Length of bearing	L_b = **60** mm

Section properties

Cross sectional area of member	A = N × b × h = **28080** mm^2
Section modulus	W_y = N × b × h^2 / 6 = **912600** mm^3
	W_z = h × (N × b)^2 / 6 = **673920** mm^3
Second moment of area	I_y = N × b × h^3 / 12 = **88978500** mm^4
	I_z = h × (N × b)^3 / 12 = **48522240** mm^4
Radius of gyration	r_y = √(I_y / A) = **56.3** mm
	r_z = √(I_z / A) = **41.6** mm

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3	γ_M = **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1	
	k_{mod} = **0.700**
Deformation factor for service classes - Table 3.2	k_{def} = **0.600**

Depth factor for bending - exp.3.1	$k_{h.m}$ = **1.000**
Depth factor for tension - exp.3.1	$k_{h.t}$ = **1.000**
Bending stress re-distribution factor - cl.6.1.6(2)	k_m = **0.700**
Crack factor for shear resistance - cl.6.1.7(2)	k_{cr} = **0.670**
Load configuration factor - exp.6.4	$k_{c.90}$ = **1.000**
System strength factor - cl.6.6	k_{sys} = **1.000**
Lateral buckling factor - cl.6.3.3(5)	k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress	$\sigma_{c.90.d} = R_{A_max} / (N \times b \times L_b)$ = **0.706** N/mm^2
Design compressive strength	$f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.346** N/mm^2
	$\sigma_{c.90.d} / f_{c.90.d}$ = **0.525**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress	$\sigma_{m.d} = M / W_y$ = **6.030** N/mm^2
Design bending strength	$f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **12.923** N/mm^2
	$\sigma_{m.d} / f_{m.d}$ = **0.467**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.487** N/mm^2
Permissible shear stress	$f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.154** N/mm^2
	$\tau_d / f_{v.d}$ = **0.226**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

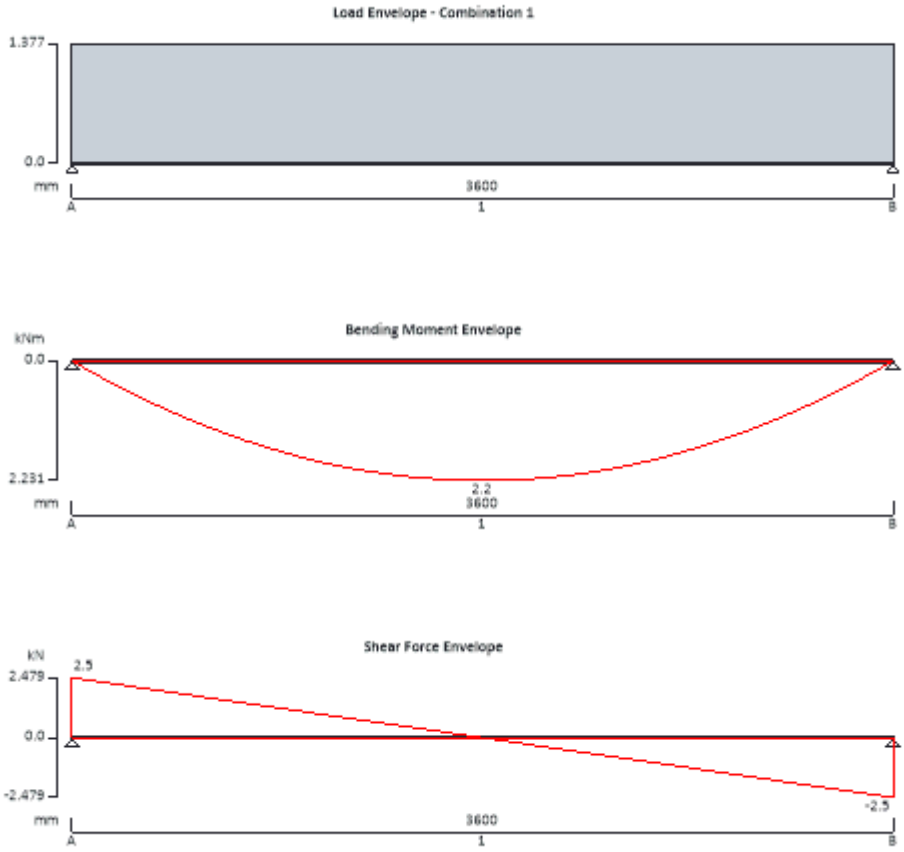
Deflection limit	$\delta_{lim} = \min(10\text{ mm}, 0.004 \times L_{s1})$ = **10.000** mm
Instantaneous deflection due to permanent load	δ_{instG} = **3.598** mm
Final deflection due to permanent load	$\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **5.757** mm
Instantaneous deflection due to variable load	δ_{instQ} = **2.401** mm
Factor for quasi-permanent variable action	ψ_2 = **0.3**
Final deflection due to variable load	$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **2.834** mm
Total final deflection	$\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **8.590** mm
	$\delta_{fin} / \delta_{lim}$ = **0.859**

PASS - Total final deflection is less than the deflection limit

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 1.7.05



Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 0.610 kN/m
Variable full UDL 0.340 kN/m

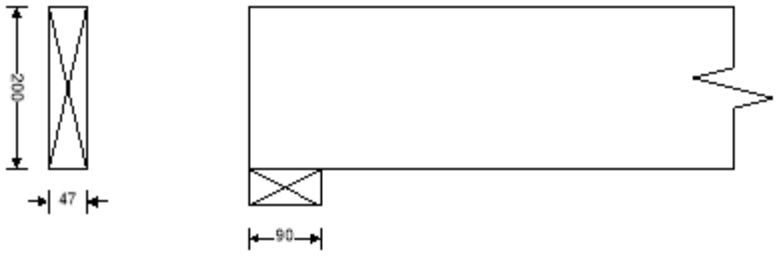
Load combinations

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

Maximum moment	M_{max} = **2.231** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **2.231** kNm	
Maximum shear	F_{max} = **2.479** kN	F_{min} = **-2.479** kN

Design shear	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min})) =$ **2.479** kN	
Total load on beam	$W_{tot} =$ **4.957** kN	
Reactions at support A	$R_{A_max} =$ **2.479** kN	$R_{A_min} =$ **2.479** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent} =$ **1.156** kN	
Unfactored variable load reaction at support A	$R_{A_Variable} =$ **0.612** kN	
Reactions at support B	$R_{B_max} =$ **2.479** kN	$R_{B_min} =$ **2.479** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent} =$ **1.156** kN	
Unfactored variable load reaction at support B	$R_{B_Variable} =$ **0.612** kN	



Timber section details

Breadth of timber sections	$b =$ **47** mm
Depth of timber sections	$h =$ **200** mm
Number of timber sections in member	$N =$ **1**
Overall breadth of timber member	$b_b = N \times b =$ **47** mm
Timber strength class - EN 338:2016 Table 1	**C24**

Member details

Load duration - cl.2.3.1.2	**Long-term**
Service class of timber - cl.2.3.1.3	**1**
Length of span	$L_{s1} =$ **3600** mm
Length of bearing	$L_b =$ **90** mm

Section properties

Cross sectional area of member	$A = N \times b \times h =$ **9400** mm^2
Section modulus	$W_y = N \times b \times h^2 / 6 =$ **313333** mm^3
	$W_z = h \times (N \times b)^2 / 6 =$ **73633** mm^3
Second moment of area	$I_y = N \times b \times h^3 / 12 =$ **31333333** mm^4
	$I_z = h \times (N \times b)^3 / 12 =$ **1730383** mm^4
Radius of gyration	$r_y = \sqrt{(I_y / A)} =$ **57.7** mm
	$r_z = \sqrt{(I_z / A)} =$ **13.6** mm

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3	$\gamma_M =$ **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1	
	$k_{mod} =$ **0.700**
Deformation factor for service classes - Table 3.2	$k_{def} =$ **0.600**
Depth factor for bending - exp.3.1	$k_{h.m} =$ **1.000**
Depth factor for tension - exp.3.1	$k_{h.t} =$ **1.000**
Bending stress re-distribution factor - cl.6.1.6(2)	$k_m =$ **0.700**
Crack factor for shear resistance - cl.6.1.7(2)	$k_{cr} =$ **0.670**
Load configuration factor - exp.6.4	$k_{c.90} =$ **1.000**

System strength factor - cl.6.6 k_{sys} = **1.000**

Lateral buckling factor - cl.6.3.3(5) k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{A_max} / (N \times b \times L_b)$ = **0.586** N/mm²

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.346** N/mm²

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.435**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = M / W_y$ = **7.120** N/mm²

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **12.923** N/mm²

$\sigma_{m.d} / f_{m.d}$ = **0.551**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.590** N/mm²

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.154** N/mm²

$\tau_d / f_{v.d}$ = **0.274**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(10\text{ mm}, 0.004 \times L_{s1})$ = **10.000** mm

Instantaneous deflection due to permanent load δ_{instG} = **4.268** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **6.829** mm

Instantaneous deflection due to variable load δ_{instQ} = **2.259** mm

Factor for quasi-permanent variable action ψ_2 = **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **2.666** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **9.495** mm

$\delta_{fin} / \delta_{lim}$ = **0.949**

PASS - Total final deflection is less than the deflection limit

STEEL MASONRY SUPPORT

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

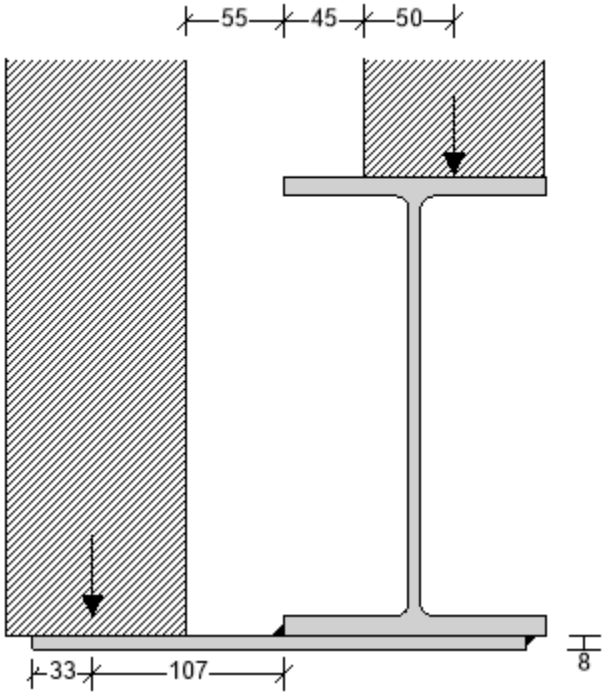
Tedds calculation version 1.0.06

Design summary

Overall design status PASS

Overall design utilisation 0.650

Description	Unit	Allowable	Applied	Utilisation	Result
Heel moment	kNm/m	2.933	0.205	0.070	PASS
Deflection	mm	1.9	0.1	0.051	PASS
Weld capacity	kN/m	945.2	247.0	0.261	PASS
Shear force (major axis)	kN	268.2	22.7	0.085	PASS
Bending (major-axis)	kNm	79.5	28.0	0.352	PASS
Bending (minor axis)	kNm	32.8	1.3	0.039	PASS
Warping	kNm	16.1	1.3	0.080	PASS
Bending and torsion				0.472	PASS
Plastic interaction				0.164	PASS
Torsion beam rotation	deg	3.00	1.95	0.650	PASS
Torsion beam deflection	mm	10.0	4.4	0.443	PASS



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resist. of members to instability γ_{M1} = **1**

Resistance of joints γ_{M2} = **1.25**

Partial factor for permanent action γ_{G} = **1.35**

Partial factor for variable action γ_{Q} = **1.50**

Partial factor for permanent action (favourable) γ_{G_fav} = **1.00**

Partial factor for variable action (favourable) γ_{Q_fav} = **0.00**

Steel beam section details

Torsion beam section type	UKB 254x146x37
Nominal yield strength	$f_y = f_{y,tb} =$ **275** N/mm^2
Nominal ultimate tensile strength	$f_u = f_{u,tb} =$ **410** N/mm^2

Masonry support section details

Section type	Plate 275x8(140)
Steel grade - EN 10025-2:2004	S275
Nominal thickness	$t_{nom,sb} = t_{plate} =$ **8** mm
Nominal yield strength	$f_{y,sb} =$ **275** N/mm^2
Nominal ultimate tensile strength	$f_{u,sb} =$ **410** N/mm^2
Modulus of elasticity	$E_{sb} =$ **210000** N/mm^2
Total length of plate	$l_{plate} =$ **275** mm
Length of plate beyond outer edge of torsion beam	$l_h =$ **140** mm

Supported materials detail

Density of masonry on torsion beam	$\rho_{m,tb} =$ **19.0** kN/m^3
Width of masonry on torsion beam	$b_{m,tb} =$ **100** mm
Height of masonry on torsion beam	$h_{m,tb} =$ **500** mm
Eccentricity of torsion beam masonry	$e_{load,tb} =$ **95** mm
Eccentricity of torsion beam material	$e_{tb} =$ **45** mm
Add perm. force torsion beam (not masonry)	$G_{k,add,tb} =$ **2.4** kN/m
Add var. force torsion beam (not masonry)	$Q_{k,add,tb} =$ **1.4** kN/m
Density of masonry on support beam	$\rho_{m,sb} =$ **19.0** kN/m^3
Width of masonry on support beam	$b_{m,sb} =$ **100** mm
Height of masonry on support beam	$h_{m,sb} =$ **750** mm
Eccentricity of support beam masonry	$e_{load,sb} =$ **106.8** mm

Geometry

Cavity width	$b_{cavity} =$ **100** mm
Supported width of masonry	$d_m = l_h + t_{shim} + e_{tb} - b_{cavity} =$ **85** mm

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

Maximum overall bending moment	$M_{y,Ed} =$ **28.0** kNm
Dist to NA combined section (CoG torsion beam)	$z_{na,all} = (h_{tb} + t_{plate}) \times A_{pl} / (2 \times (A_{tb} + A_{pl})) =$ **42** mm
Second moment of area of combined section	$I_{y,all} = (I_{ytb} + A_{tb} \times z_{na,all}^2) + A_{pl} \times (h_{tb} / 2 + t_{plate} / 2 - z_{na,all})^2 =$ **8151** cm^4
Elastic section modulus of combined section	$Z_{y,all} = I_{y,all} / (h_{tb} / 2 + t_{plate} - z_{na,all}) =$ **866.98** cm^3
Section modulus of plate	$Z_{y,plate} = 1\text{m} \times t_{plate}^2 / (6 \times 1\text{m}) =$ **10.67** cm^3/m
Force of masonry on support plate	$F_1 = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) \times \gamma_G + Q_{k,add,sb} \times \gamma_Q =$ **1.9** kN/m
Bending at heel	$M_{y,Ed,plate} = F_1 \times e_{load,sb} =$ **0.2** kNm/m
Moment capacity of plate	$M_{y,Rd,plate} = Z_{y,plate} \times f_{y,sb} / \gamma_{M0} =$ **2.9** kNm/m
	PASS - Moment capacity of plate exceeds applied moment
Longitudinal stress due to overall bending	$\sigma_1 = M_{y,Ed} / Z_{y,all} =$ **32.3** N/mm^2
Constant relating to Von Mises curve	$c_{fp} = (4 \times f_{y,sb}^2 - 3 \times \sigma_1^2)^{0.5} =$ **547.1** N/mm^2
Transverse bending stress ratio limit	$\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times f_{y,sb}) =$ **0.991**
Transverse bending stress ratio	$\alpha_{ls} = M_{y,Ed,plate} / M_{y,Rd,plate} =$ **0.070**
	PASS - Transverse bending stress ratio less than allowable limit

Deflection of plate

Unfactored force on support angle	$F_{1ser} = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) + Q_{k,add,sb} =$ **1.4** kN/m
Distance from weld to load position	$a_m = e_{load,sb} =$ **107** mm

Length of load resultant to edge of plate	$b_m = l_h - e_{load,sb}$ = **33** mm
Dist from weld to load position as ratio of length	$a_l = a_m / (a_m + b_m)$ = **0.763**
Effective second moment of area	$I_{eff_def} = t_{plate}^3 / 12$ = **42667** mm⁴/m
Deflection at end of plate	$\delta = (a_l^2 \times (3 - a_l) / 6) \times (F_{1ser} \times (a_m + b_m)^3) / (E_{sb} \times I_{eff_def})$ = **0.09** mm
Deflection limit	$\delta_{lim} = \min((1 + d_m / b_{cavity}) \times 1mm, 2mm)$ = **1.85** 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	$F_A = F_1 \times \max((1 + (3 \times e_{load,sb}) / (2 \times b_{tb} / 2)), 1.4)$ = **6.1** 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** kN
Longitudinal shear between beam and plate	$F_l = 2 \times F_p / L$ = **245.2** kN/m
Horizontal shear between beam and plate	$F_h = F_1 \times e_{load,sb} / (s_{weld} / 2 + t_{plate} / 2)$ = **29.4** kN/m
Resultant weld force	$F_{w,Ed} = (F_A^2 + F_l^2 + F_h^2)^{0.5}$ = **247.0** kN/m
Leg length of weld	s_{weld} = **6.00** mm
Throat thickness of weld	$a_{weld} = 1 / \sqrt{(2)} \times s_{weld}$ = **4.24** mm
Length of weld per metre run	l_{weld} = **1000** mm/m
Ultimate tensile strength used for weld	$f_{u,weld} = \min(f_{u,sb}, f_{u,tb})$ = **410.0** N/mm²
Correlation factor (table 4.1)	β_w = **0.85**
Design shear strength	$f_{vw,d} = f_{u,weld} / (\sqrt{(3)} \times \beta_w \times \gamma_{M2})$ = **222.8** N/mm²
Design resistance of weld	$F_{w,Rd} = f_{vw,d} \times a_{weld}$ = **945.2** kN/m

PASS - weld capacity exceeds applied force

Eccentricities

Distance to shear centre of torsion beam	$e_{0,tb}$ = **0** mm
Eccentricity of support beam masonry	$e_{m,sb} = e_{load,sb} + b_{tb} / 2$ = **180** mm
Eccentricity of torsion beam masonry	$e_{m,tb} = b_{tb} / 2 - e_{load,tb}$ = **-22** mm
Eccentricity of support beam	$e_{b,sb} = c_{zsb} + b_{tb} / 2$ = **76** mm
Eccentricity of torsion beam	$e_{b,tb}$ = **0** mm

Torsional loading ULS (unfavourable)

Loading of support beam masonry	$w_{sb} = (h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}) \times \gamma_G$ = **1.92** kN/m
Loading of torsion beam masonry	$w_{tb} = (h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb}) \times \gamma_G + Q_{k,add,tb} \times \gamma_Q$ = **6.56** kN/m
Self weight of support beam	$w_{sw,sb} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_G$ = **0.23** kN/m
Self weight of torsion beam	$w_{sw,tb} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_G$ = **0.49** kN/m

Torsional loading ULS (favourable)

Loading of support beam masonry	$w_{sb_fav} = (h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}) \times \gamma_{G_fav}$ = **1.43** kN/m
Loading of torsion beam masonry	$w_{tb_fav} = (h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb}) \times \gamma_{G_fav} + Q_{k,add,tb} \times \gamma_{Q_fav}$ = **3.36** 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** kN/m
Self weight of torsion beam	$w_{sw,tb_fav} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_{G_fav}$ = **0.36** kN/m

Torsional loading SLS (unfavourable)

Loading of support beam masonry	$w_{sb,ser} = h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}$ = **1.43** kN/m
Loading of torsion beam masonry	$w_{tb,ser} = h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb} + Q_{k,add,tb}$ = **4.71** kN/m
Self weight of support beam	$w_{sw,sb,ser} = A_{pl} \times \rho_{SEC3} \times g_{acc}$ = **0.17** kN/m
Self weight of torsion beam	$w_{sw,tb,ser} = A_{tb} \times \rho_{SEC3} \times g_{acc}$ = **0.36** kN/m

Torsional effects

Applied torque (ULS +ve ecc. unfav, -ve ecc. fav) $T_{d,w,fav1} = abs(w_{sb} \times e_{m,sb} + w_{tb_fav} \times e_{m,tb} + w_{sw,sb} \times e_{b,sb} + w_{sw,tb} \times e_{b,tb}) =$ **0.29** kNm/m

Applied torque (ULS +ve ecc. fav, -ve ecc. unfav) $T_{d,w,fav2} = abs(w_{sb_fav} \times e_{m,sb} + w_{tb} \times e_{m,tb} + w_{sw,sb_fav} \times e_{b,sb} + w_{sw,tb_fav} \times e_{b,tb}) =$ **0.13** kNm/m

Applied torque (ULS all unfavourable) $T_{d,w} = abs(w_{sb} \times e_{m,sb} + w_{tb} \times e_{m,tb} + w_{sw,sb} \times e_{b,sb} + w_{sw,tb} \times e_{b,tb}) =$ **0.22** kNm/m

Total torque (ULS) $T_d = max(T_{d,w}, T_{d,w,fav1}, T_{d,w,fav2}) \times L =$ **1.43** kNm

Applied torque (SLS) $T_{d,w,ser} = abs(w_{sb,ser} \times e_{m,sb} + w_{tb,ser} \times e_{m,tb} + w_{sw,sb,ser} \times e_{b,sb} + w_{sw,tb,ser} \times e_{b,tb}) =$ **0.17** kNm/m

Total torque (SLS) $T_{d,ser} = T_{d,w,ser} \times L =$ **0.82** 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

Tedds calculation version 1.0.06

Partial factors - Section 6.1

Resistance of cross-sections $\gamma_{M0} =$ **1**

Resistance of members to instability $\gamma_{M1} =$ **1**

Section details

Section type: UKB 254x146x37 (Tata Steel Advance)

Steel grade - EN 10025-2:2004: S275

Nominal thickness of element $t_{nom} = max(t_f, t_w) =$ **10.9** mm

Nominal yield strength $f_y =$ **275** N/mm^2

Nominal ultimate tensile strength $f_u =$ **410** N/mm^2

Modulus of elasticity $E =$ **210000** N/mm^2

Shear centre

Distance between flange shear centres $h_s = h - t_f =$ **245.1** mm

Shear centre (above bottom flange centroid) $e_{s,bf} = h_s / 2 =$ **122.6** mm

Analysis results

Design bending moment - major axis $M_{y,Ed} =$ **28.0** kNm

Design shear force - major axis $V_{y,Ed} =$ **22.7** kN

Classification

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

Width of section: $c = d =$ **219** mm

$c / t_w = 34.8 = 37.6 \times \varepsilon <= 72 \times \varepsilon$ Class 1

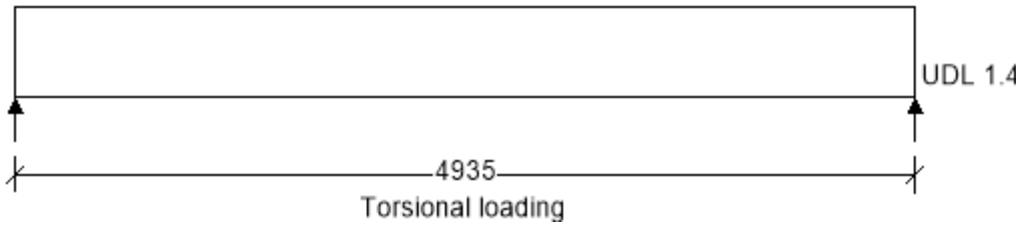
Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2 =$ **62.5** mm

$c / t_f = 5.7 = 6.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Torsional loads

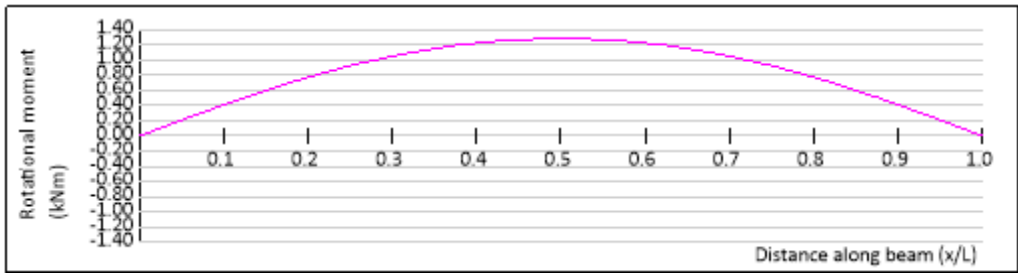


Load No.	Load type	Load (kNm)	Distance along beam (mm)
1	UDL	1.4	-

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, $M_{z,add,Ed} = M_{y,Ed} \times \phi$

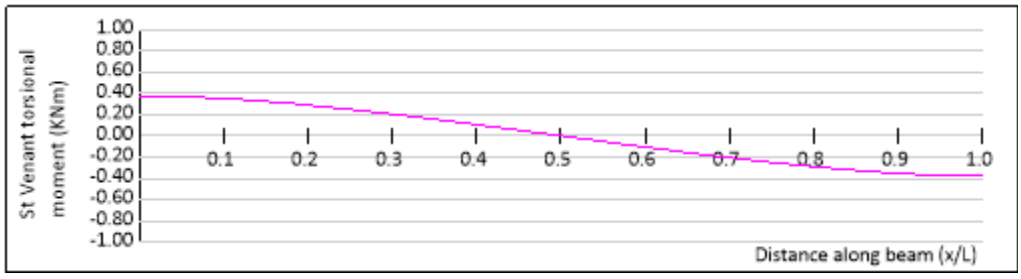


Additional minor design moment due to rotation $M_{z,add,Ed}$ = **1.29** kNm

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 $T_{t,Ed} = G_{SEC3} \times I_t \times \phi'$

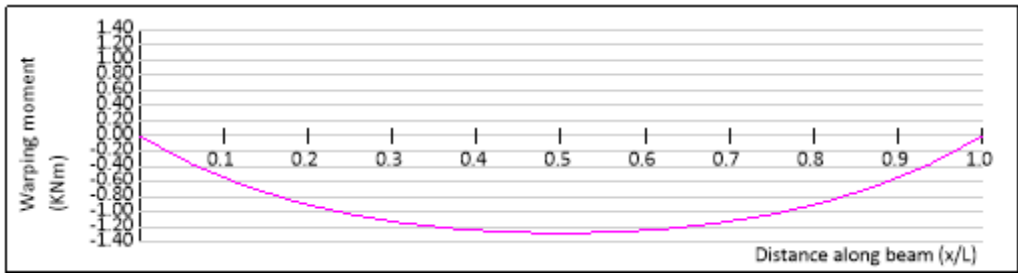


St Venant torsion design moment $T_{t,Ed}$ = **0.38** kNm

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, $M_{w,Ed} = E_{SEC3} \times I_w \times \phi'' / h_s$

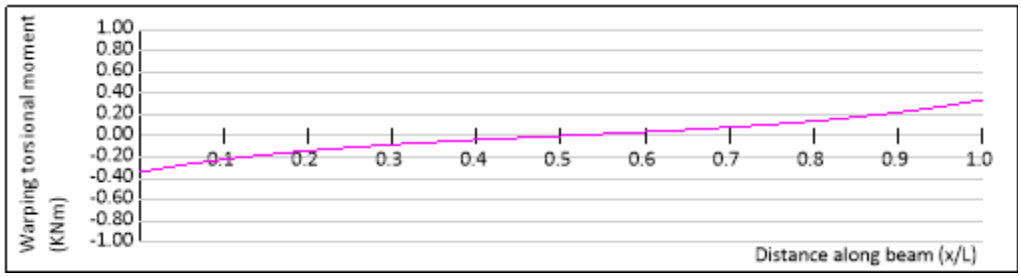


Warping design moment $M_{w,Ed}$ = **1.28** kNm

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 $T_{w,Ed}$ = **0.34** kNm

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **234.2** mm

η = **1.000**

$h_w / t_w = 37.2 = 40.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **22.71** 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** mm²

Design shear resistance - cl 6.2.6(2) $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **279.3** kN

Shear stress due to St Venant torsion $\tau_{t,Ed} = T_{t,Ed} \times t_w / I_t$ = **15.53** N/mm²

Reduced shear resistance due to torsion - eq 6.26 $V_{c,y,Rd} = V_{pl,T,y,Rd} = \sqrt{(1 - \tau_{t,Ed} / (1.25 \times (f_y / \sqrt{(3)}) / \gamma_{M0}))} \times V_{pl,y,Rd}$ = **268.2** kN

$V_{y,Ed} / V_{pl,T,y,Rd}$ = **0.085**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **28** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **132.9** kNm

$M_{y,Ed} / M_{pl,y,Rd}$ = **0.211**

PASS - Design bending resistance moment exceeds design bending moment

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 ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained effective length L = **4935** mm

Distance from shear centre to level of load $z_g = (h_{tb} / 2 \times w_{tb} - h_{tb} / 2 \times w_{sb}) / (w_{tb} + w_{sb})$ = **70.0** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (k_{z,cr} \times L^2) \times (\sqrt{((k_{z,cr} / k_{w,cr})^2 \times I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z) + (C_2 \times z_g)^2)} - (C_2 \times z_g))$ = **94.1** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **1.188**

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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **1.163**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.586**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.980**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.598**
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **79.5** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.352**
	PASS - Design buckling resistance exceeds design buckling moment

Check bending moment - Section 6.2.5

Design bending moment	$M_{z,Ed,total} = M_{z,Ed} + M_{z,add,Ed}$ = **1.3** kNm
Design bending resistance moment - eq 6.13	$M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0}$ = **32.8** kNm
	$M_{z,Ed,total} / M_{pl,z,Rd}$ = **0.039**
	PASS - Design bending resistance moment exceeds design bending moment

Check warping moment

Warping moment in flange	$M_{w,Ed}$ = **1.28** kNm
Plastic modulus of flange	$W_{pl.f} = t_f \times b^2 / 4$ = **58.40** cm^3
Design warping resistance of flange	$M_{w,Rd} = W_{pl.f} \times f_y / \gamma_{M0}$ = **16.06** kNm
	$M_{w,Ed} / M_{w,Rd}$ = **0.08**
	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	$M_{y,Rk} = W_{pl.y} \times f_y$ = **132.9** kNm
Characteristic moment resistance - z axis	$M_{z,Rk} = W_{pl.z} \times f_y$ = **32.8** kNm
Characteristic warping resistance	$M_{w,Rk} = W_{pl.f} \times f_y$ = **16.1** kNm
Interaction factors	$k_w = 0.7 - 0.2 \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1})$ = **0.68**
	$k_{zw} = 1 - M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1})$ = **0.96**
	$k_\alpha = 1 / (1 - M_{y,Ed} / M_{cr})$ = **1.42**
Interaction formula eqn A.1	$M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + C_{mz} \times M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1}) + k_w \times k_{zw} \times k_\alpha \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1})$ = **0.472**
	PASS - Combined bending and torsion check satisfied

Plastic verification - Exp.6.41

	$(M_{y,Ed} / M_{pl,y,Rd})^2 + M_{z,Ed,total} / M_{pl,z,Rd} + M_{w,Ed} / M_{w,Rd}$ = **0.164**
	PASS - Plastic interaction criterion is less than 1.0

Serviceability limit checks

Rotation limit	$\phi_{ser,lim}$ = **3.00** deg
Rotation of torsion beam	$\phi_{ser} = M_{z,add,Ed} \times 180 / (M_{y,Ed} \times \gamma_G \times \pi)$ = **1.95** deg
	PASS - Rotation limit exceeds rotation in torsion beam
Vertical deflection limit	$\delta_{v,lim}$ = **10.0** mm
SLS loading on beam	$f_{d,ser} = w_{sb,ser} + w_{tb,ser} + w_{sw,sb,ser} + w_{sw,tb,ser}$ = **6.67** kN/m
Vertical deflection of torsion beam	$\delta_v = 5 \times f_{d,ser} \times L^4 / (384 \times E_{sb} \times I_{ytb})$ = **4.4** mm
	PASS - Vertical deflection limit exceeds vertical deflection in torsion beam

MASONRY BEARING DESIGN

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

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	28.5 kN	31.4 kN	N/A	N/A	0.909	Pass

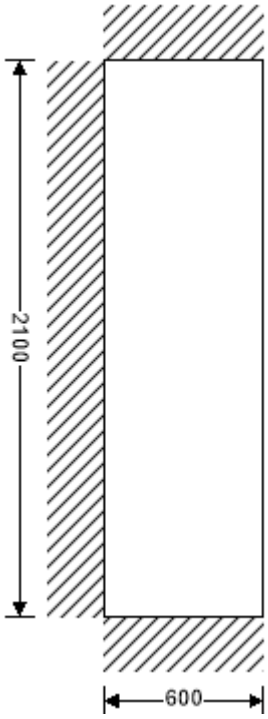
Masonry panel details

Panel length — L = **600** mm

Panel height — h = **2100** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints — $L < 15 \times t_{ef}$ (Wall restrained at the top and bottom and stiffened on one vertical edge (with one free edge))

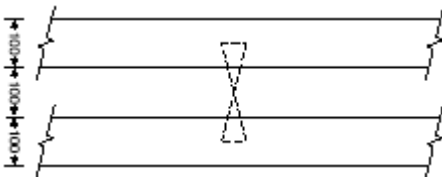
Horizontal restraints — **Simply supported**

Reduction factor - eq. 5.5 — ρ_2 = **1.00**

Reduction factor - eq. 5.6 — $\rho_3 = 1/(1 + [(\rho_2 \times h) / (3 \times L)]^2) \times \rho_2$ = **0.42**

Effective height of wall - eq. 5.2 — $h_{ef} = \rho_3 \times h$ = **889** mm

Wall construction details



Wall type	**Cavity wall**
Inner leaf thickness	t_2 = **100.0** mm
Cavity thickness	t_c = **100.0** mm
Outer leaf thickness	t_1 = **100.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness of inner leaf	$t_{ef2} = t_2$ = **100.0** mm
Effective thickness of outer leaf	$t_{ef1} = \text{Min}(t_1, t_2)$ = **100.0** mm
Factor for the relative E values	k_{tef} = **1.00**
Effective thickness - eq. 5.11	$t_{ef} = (k_{tef} \times t_{ef1}^3 + t_{ef2}^3)^{1/3}$ = **126.0** mm

Masonry material details

Unit type	**Aggregate concrete - Group 2**
Compressive strength of masonry unit	f_c = **7.3** N/mm^2
Height of unit	h_u = **215** mm
Width of unit	w_u = **100** mm
Conditioning factor	k = **1.0**
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	d_{sf} = **1.38**
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf}$ = **10.07** N/mm^2
Specific weight of units	γ = **18** kN/m^3
Mortar type	**M4 - General Purpose**
Compressive strength of mortar	f_m = **4.0** N/mm^2
Compressive strength factor - Tbl. NA 4	K = **0.70**
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **5.35** N/mm^2
Short term secant modulus of elasticity factor	K_E = **1000**
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k$ = **5345** N/mm^2

Design compressive strength of masonry for inner (loaded) leaf

Category of manufacturing control	**Category II**
Class of execution control	**Class 2**
Partial factor for compressive strength	γ_M = **3.00**
Cross-sectional area of wall	A = L × t = **0.06** m^2 (A < 0.1 m^2)
Design compressive strength of masonry	$f_d = f_k / \gamma_M \times (0.7 + 3 \times A / 1\text{ m}^2)$ = **1.57** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = **1.35**
Partial safety factor for variable load	γ_{fQ} = **1.50**

Superimposed vertical loading details

Permanent UDL at top of wall	g_k = **0.00** kN/m
Variable UDL at top of wall	q_k = **0.00** kN/m
Eccentricity of permanent UDL load	e_{gu} = **0** mm

Eccentricity of variable UDL load e_{qu} = **0** mm

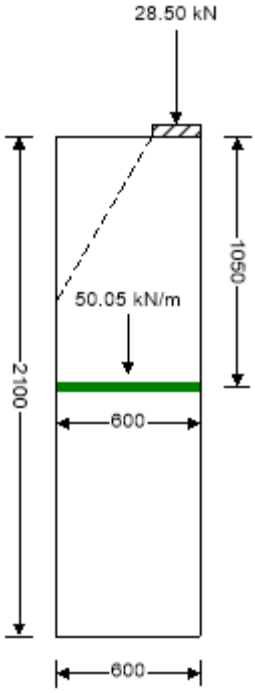
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit λ_{lim} = **27**

Slenderness ratio $\lambda = h_{ef} / t_{ef}$ = **7.1**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load G_{kc1} = **17.41** kN

Variable concentrated load Q_{kc1} = **3.33** kN

Eccentricity of concentrated load e_{c1} = **0** mm

Length of concentrated load L_{c1} = **200** mm

Width of concentrated load w_{c1} = **100** mm

Height of concentrated load h_{c1} = **2100** mm

Distance of load to right vertical edge r_{11} = **0** mm

Distance of load to nearest vertical edge a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing $A_{b1} = L_{c1} \times w_{c1}$ = **20000** mm^2

Effective length of bearing at mid-height $l_{efm1} = L$ = **600** mm

Effective bearing area $A_{ef1} = l_{efm1} \times t$ = **60000.00** mm^2

Bearing area ratio check $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.33**

Enhancement factor - cl.6.1.3(3) β_1 = **1.00**

Design value of the concentrated load $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **28.50** kN

Design value concentrated load resistance $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **31.36** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) =$ **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) =$ **0.0** mm

Eccentricity of concentrated load at mid-height	$e_{mc1} = e_{c1} / 2 =$ **0.0** mm
Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h_{ef} / 450 =$ **2.0** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1} =$ **47.50** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} =$ **50.05** kN/m
Design moment at mid-height	$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} =$ **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} =$ **2.0** mm
Slenderness ratio limit for creep eccentricity	$\lambda_c =$ **27**
Eccentricity due to creep	$e_{k1} =$ **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) =$ **5.0** mm
From eq. G2	$A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.90**
From eq. G3	$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **0.24**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.87**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **137.15** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load