IKT CONSULTING
STRUCTURAL ENGINEERS LIMITED

Client: Mr A. Client

Project: Mapperley, Nottingham

Report: Structural Calculations

Job No.: IKT0000

Date: 1 April 2024

Document History

REVISION	DATE	DESCRIPTION	PREPARED BY	CHECKED BY
	01 April 2024	Structural Calculations	S. Engineer	An Engineer

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Structural Calculations

Introduction

The following calculations have been produced for the proposed structural alteration referred to as No. 1 Mapperley, Nottingham.

The existing property appeared to be a solid brick wall construction, with a traditional timber joist for floor level and cut timber hipped roof formed with rafters and purlins.

Scope of Design /work

IKT Consulting Limited design was limited to loose steel beams with padstones, steel column and a pad foundation, timber joists required to support roof, floors and wall.

General Notes

The Engineer has carried out the design in accordance with the information provided to him during the initial site visit and drawing provided by the client.

However, unless and until the structural fabric of the building is fully exposed, these should be treated as assumptions and not certainties and should be confirmed or otherwise by the contractor on site. Should the contractor's site discoveries indicate that these assumptions are incorrect he should advise the Engineer immediately and await the Engineers advice on how to proceed.

Sketches are to demonstrate certain features of the design and are not intended as working drawings. Where shown, details are intended to identify the main structural features. It is assumed that the work will be carried out by experienced and competent personnel, therefore exhaustive detailing is not required.

The fabricator/supplier will normally bear responsibility for the structural members up to the point where they are off-loaded onto the site: thereupon they become the responsibility of the contractor.

The delivery should be checked to ensure that it complies with the specification and that the quantities and dimensions are correct. Any discrepancies must immediately be brought to the attention of the supplier.

Contractor/builder appointed to carry out the construction work must carefully assess our proposed layouts, proposed structural specifications and existing site before undertaking construction work.

If the contractor is unsure about the length or size of any design structural element, he must contact the structural engineer for clarification before undertaking construction work.

The contractor must demonstrate a full understanding of the project before starting deconstruction/construction work, and if unclear about any part of the design the contractor must contact us before undertaking the work.

If needed client or contractor must get the local authority approval, i.e. submit the proposed design and layout for approval before undertaking any construction work.

The contractor must provide the client with details of the construction process and risks involved (i.e. damage to existing decoration, existing features and fixtures etc.) before carrying out the construction work.

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Fire protection to be in accordance with relevant Building Regulations and Architect's details. New steel beams to be fire protected using British Gypsum Gyproc Fireline Pink plasterboard or 2 layers of plasterboard and skim to achieve a minimum of 30 minutes to 1-hour fire protection.

All dimensions are to be confirmed by the contractor on-site prior to construction.

All bolts to be minimum M16, Grade 8.8 u.n.o.

All internal steelwork to be shot blast to SA2.5 Standard and painted with 2 coats of zinc phosphate min 120 microns or Red Oxide Primer except as noted on drawings.

All steel beam ends are to be painted with 2No. coats bituminous paint where embedded in the external wall.

Steelwork in the cavity to receive 2No. coats of bitumastic paint.

All temporary works to the contractor's design and details.

Amendments to the design

Before placing an order or commencing work on site the contractor should be satisfied that the design brief is correct and that he has sufficient information to perform the works safely. IKT Consulting Ltd cannot be held responsible for any incorrect or incomplete design brief.

Codes of Practice

This project was generally designed using the following standards:

The Building Regs. – Approved Document A (2010)

BS EN 1991-1-1 : 2002 General actions: Densities, self-weight, imposed loads for buildings

BS EN 1991-1-1 : 2002 Actions on structures: Dead and Imposed Loads

BS EN 1993-1-1 : 2005 Design of steel structures: General rules and rules for buildings

BS EN 1996-1-1 : 2005 Masonry Unreinforced & Reinforced

BS EN 1996-1-2 : 2006 Masonry Materials

P202 Steelwork Design Guide to BS5950-1:2000 Section Properties and member capacities

Calculation Method

Trimble Tekla TEDDS design software used to assist with these calculations (printouts are included) to Eurocode Standards.

<u>Structural Consideration</u>

All ground floor internal walls are assumed to be solid brickwork walls and are to be confirmed on site.

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The condition and adequacy of existing structures and foundations to support additional loads should be confirmed onsite before commencing construction works.

<u>Design Notes</u>

1. **<u>All dimensions are to be confirmed by the contractor on-site prior to construction.</u>**
2. Steelwork to be grade S355, execution class 2and CE marked unless otherwise noted.
3. To minimise deflections of the existing structure, new beams must be pinned uptight to existing construction with slate or dry-pack mortar, and all mortar must be allowed to cure prior to de-propping.
4. All work to be undertaken in accordance with the current Building Regulations Part A, Eurocode and good building practice.
5. Beams and lintels are to have a minimum bearing length of 100mm when perpendicular to the wall, and 150mm when parallel to the wall unless noted otherwise.
6. Due to significant structural works, minor post-construction deflection of brittle finishes may be expected in the existing building.
7. All load-bearing inner skin walls are to be minimum 100mm thick medium density (3.6N) concrete blockwork wall unless noted otherwise.
8. All steels that support timber work are to have the flanges pre-drilled @ 500mm centres to accept timber plates.
9. Drawings are not to scale.

Timber

10. Floor joists to be doubled up under stud partitions where parallel and noggins under partitions where perpendicular and fixed to joists underneath.
11. New structural timber is to be a minimum grade C24 in accordance with the latest edition of BS5268 unless otherwise noted.
12. All bolting to be as noted in 2mm maximum clearance holes. Where bolts are fitted with timber connectors, washers of the appropriate size and thickness must be used in accordance with BS 5268.
13. Multiple timber members are to be bolted together using M12 grade 4.6 bolts with 50x50x3mm thick washers at 800mm maximum centres.
14. All screwing operations are to be installed in predrilled holes.
15. Wall plates are to be generally 100 x 75 unless noted otherwise fixed to walls using 30 x 2.5 galvanised mild steel vertical restraint straps 900mm long at maximum 1250mm centres with 6no screws in polymide plugs equally spaced and to current Building Regulations.

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Design Summary - Member Sizes

Beam (B1):	1No 203x203 UC 46, Grade S355, weld 6mm thick 230mm wide plate to top flange. Span dimension to be confirmed by builder on site.
Beam (B2):	1No 203x203 UC 46, Grade S355, weld 6mm thick 230mm wide plate to top flange. Span dimension to be confirmed by builder on site.
Beam (B3):	1Nos 254x254 UC 89, Grade S355. Span dimension to be confirmed by builder on site.
Beam (B4):	1No 180x90x26 PFC, Grade S355 galvanised, with 10mm thick, 300mm (TBC) wide plate welded to bottom. Span dimension to be confirmed by builder on site.
Column (C1):	1No 152x152 UC 30, Grade S355. Height dimension to be confirmed by builder on site.
Trimmer (T1):	1No 150x90x24 PFC, Grade S355 galvanised. Span dimension to be confirmed by builder on site.
Trimmer (T2):	1No 150x90x24 PFC, Grade S355 galvanised. Span dimension to be confirmed by builder on site.
Floor Joist (FJ):	50 x 175, Grade C24 Timber, at 400mm centres.
Roof Joist (RJ):	50 x 150, Grade C24 Timber, at 400mm centres.
L 1:	1No. Naylor R6 or similar 100 (W) x 140 (H) Prestress Concrete Lintel.
L 2:	2No. Naylor R6 or similar 100 (W) x 140 (H) Prestress Concrete Lintel.

Padstones

PS 1: 1No 440 (L) x 100 (W) x 215 (H), C35 Mass concrete Padstone.

PS 2: 1No 300 (L) x 100 (W) x 140 (H), C35 Mass concrete Padstone.

Foundations

F 1: 1000 (L) x 1000 (W) x 600 (Deep) min., C35 Mass fill concrete pad.

F 2: 1200 (L) x 1200 (W) x 600 (Deep) min., C35 Mass fill concrete pad.

Our site visit was limited to visual inspection and the contractor/builder appointed to carry out the construction work must carefully assess our proposed layouts, proposed structural specifications against the existing site by removing the building covering and plasterboards to expose the structure before ordering materials or commencing work on-site and undertaking construction work. If unclear about any part of the design the contractor must contact IKT Consulting before ordering materials and allow a sufficient time scale of no less than 48 hours to resolve any discrepancy.

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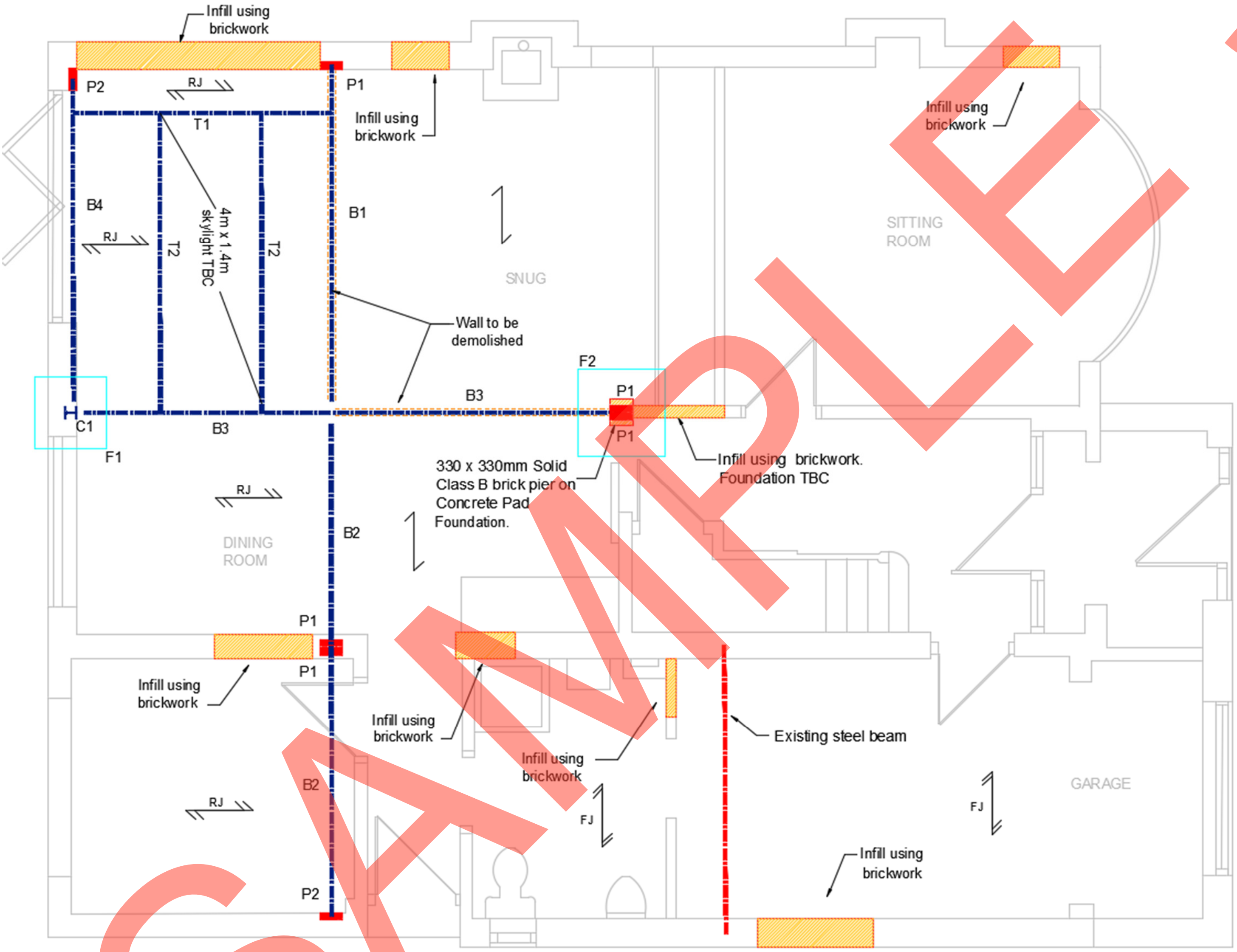
KEY PLAN

ALL TOP OF STEEL BEAMS TO BE CONFIRMED ON SITE



Ground Floor Showing Structure Above

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First Floor Level Structure

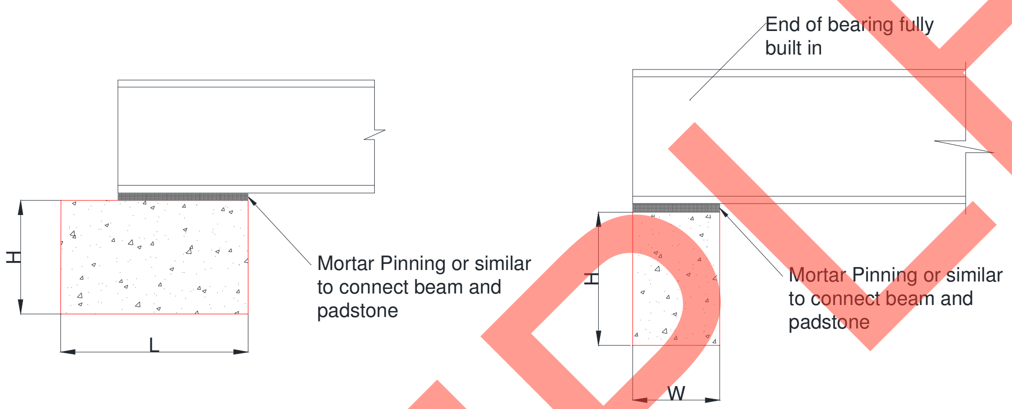
NOTE:
Do not scale from this drawing.
Existing structures must be adequately propped during beam installation.

All site works shall be in accordance with the health & safety Act & associated regulations issued by the Health & Safety Executive & the Construction Regulations.

Materials in excess of 20kg must be 2 man lift or machine lift

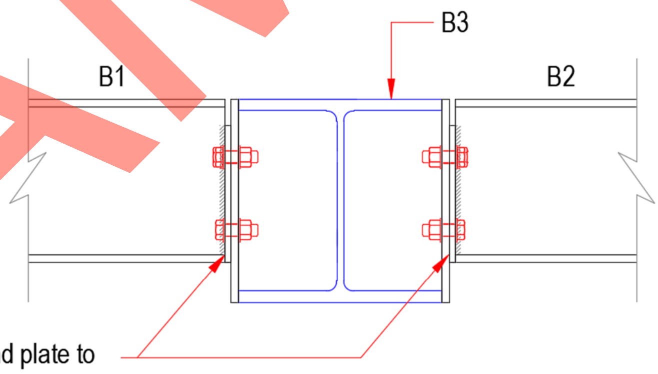
All dimensions are to be confirmed by the contractor on site prior to placing an order or commencing work on site.

Indicates floor span @ first floor



Minimum bearing length of 100mm when perpendicular to the wall, and 200mm when parallel to the wall unless noted otherwise.

Detail 1 – Typical Padstone



Weld 10Thk. End plate to suit existing beams and connect to Proposed Beam with 4No.18Ø holes for M16 grade 8.8 bolts with 6mm full profile fillet weld. + 6mm Thk web stiffeners.

Detail 2 - Typical Beam-To-Beam Connection

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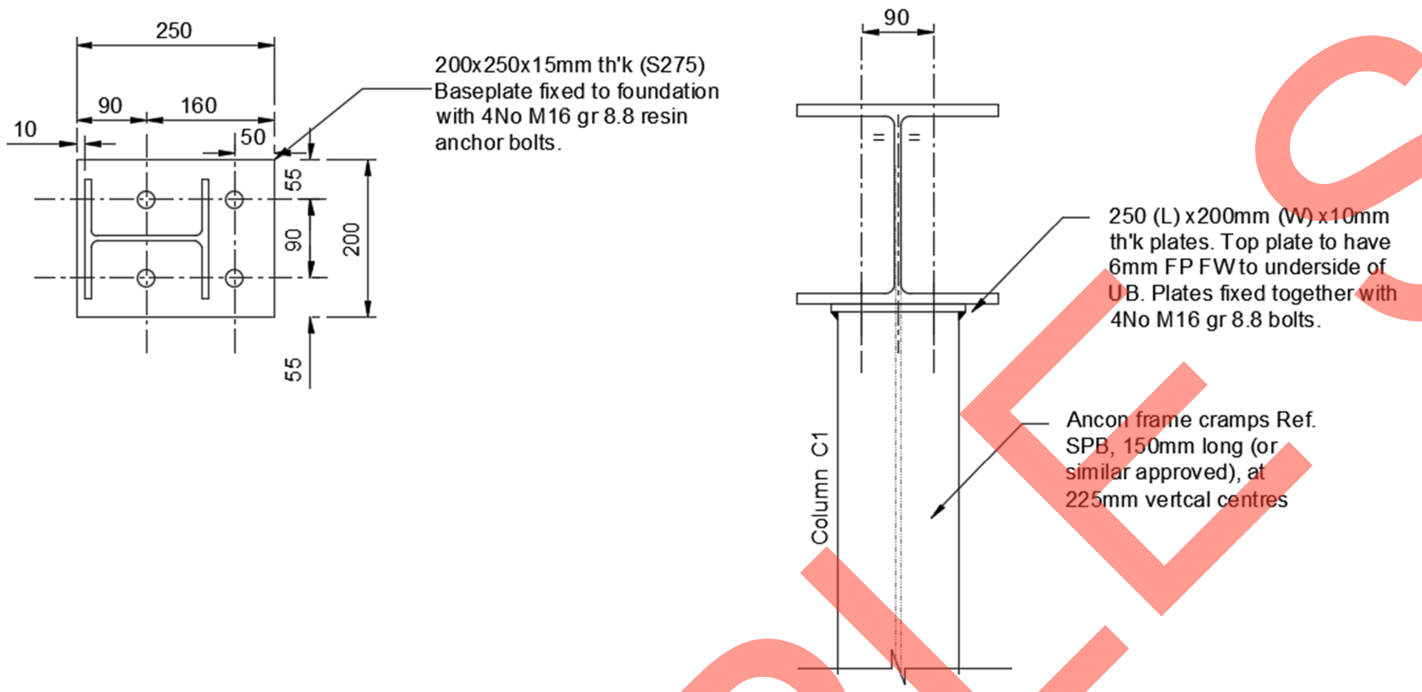
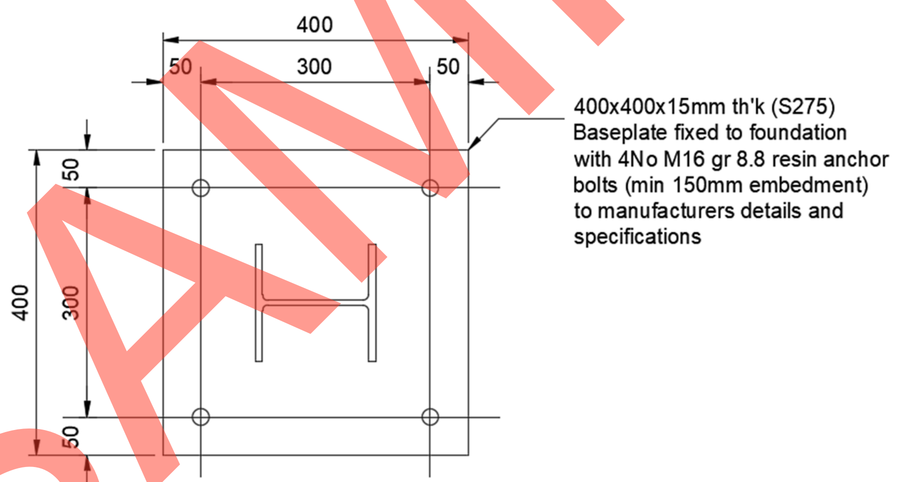
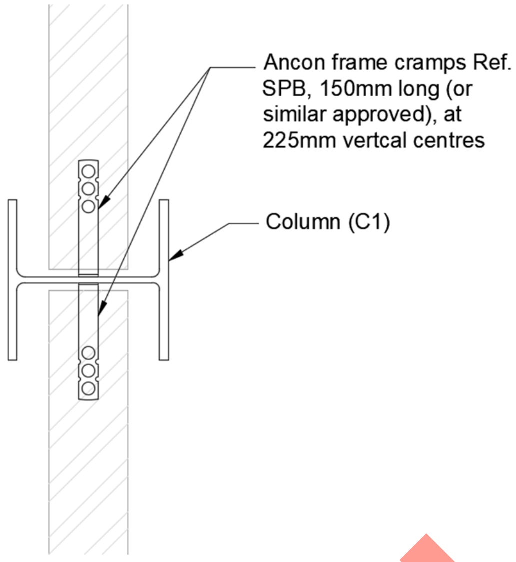


Detail 3 - Typical Beam-To-Column Connection



Detail 4 - Typical Column Baseplate



Detail 5 - Typical Column Frame Cramps

Load Schedule

Unfactored working loads.

		Dead, kN/m^2		Imposed, kN/m^2
Roof (sloping rafters)				
	Tiles	0.70	Snow	0.60
	Batton,felt, rafters	0.15	Attic storage	0.25
	Insulation	0.05	Total imposed,	**0.85**
	Ceiling	0.15		
	Total dead, on plan	**1.48**		
Roof (Flat rafters)				
	Asphalt waterproof	0.45	Snow (drift)	1.00
	Joists	0.15		
	Insulation	0.05	Total imposed,	**1.00**
	Ceiling & services	0.20		
	Total dead	**0.85**		
First floor				
	Plywood	0.20	Live	1.50
	Joist	0.20	Partitions	0.50
	Ceiling	0.15	Total imposed	**2.00**
	Total dead	**0.55**		

		Dead, kN/m^2			Dead, kN/m^2
Exterior solid wall			**Int partition wall**		
	215 solid brick leaf	3.87		Brickwork	2.20
	Plaster	0.30		Plaster - both sides	0.60
				Total dead	**2.80**
	Total dead	**4.17**			

Beam – B1

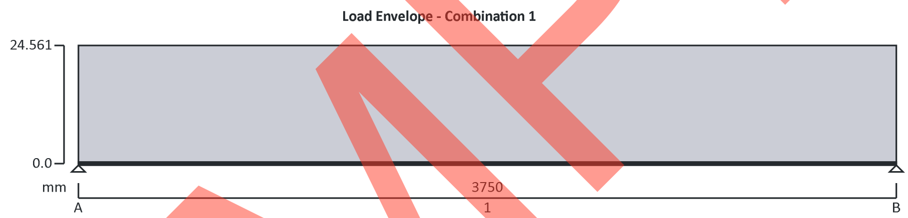
Beam span = 3.75 m

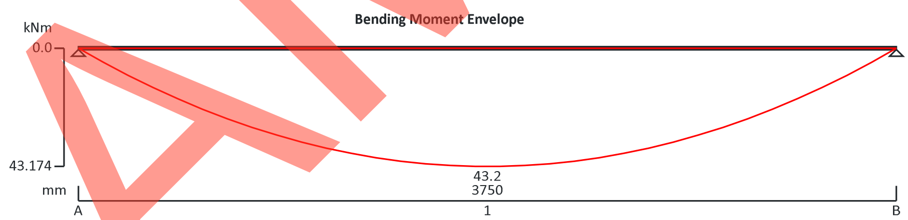
		Area Load	Loaded Width/Height	UDL, unfactored
Roof				
Sloping rafters	Dead	1.48 kN/m^2	2.00 m	2.97 kN/m
	Live	0.85 kN/m^2	2.00 m	1.70 kN/m
Roof				
Flat rafters	Dead	0.85 kN/m^2	1.25 m	1.06 kN/m
	Live	1.00 kN/m^2	1.25 m	1.25 kN/m
Wall				
Ext Solid wall	Dead	4.17 kN/m^2	2.50 m	10.43 kN/m
	Live	0.00 kN/m^2	2.50 m	0.00 kN/m

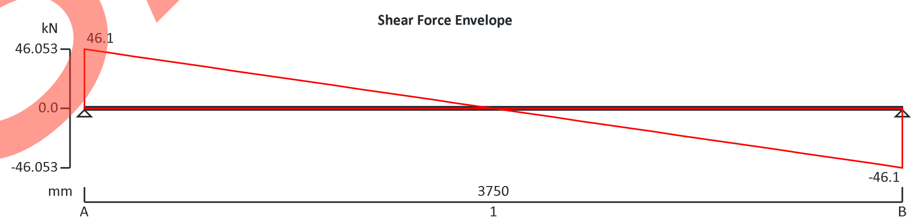
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14







Support conditions

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

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Applied loading

Beam loads	Permanent self weight of beam × 1	
	Permanent full UDL 14.46 kN/m	
	Variable full UDL 2.95 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

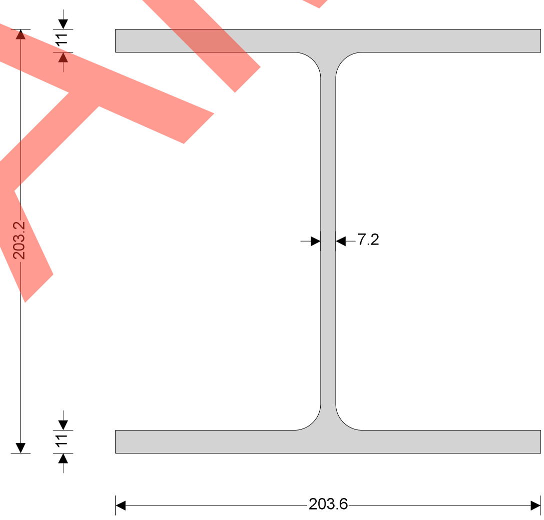
Maximum moment	M_{max} = **43.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **46.1** kN	V_{min} = **-46.1** kN
Deflection	δ_{max} = **4.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **46.1** kN	R_{A_min} = **46.1** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **28** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **5.5** kN	
Maximum reaction at support B	R_{B_max} = **46.1** kN	R_{B_min} = **46.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **28** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.5** kN	

Section details

Section type	**UC 203x203x46 (British Steel Section Range 2022 (BS4-1))**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **11.0** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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 only

Effective length factors

Effective length factor in major axis: K_y = **1.000**

Effective length factor in minor axis: K_z = **1.000**

Effective length factor for torsion: $K_{LT.A}$ = **1.200** + 2 × h

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

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

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

Width of section: $c = d$ = **155.8** mm

$c / t_w = 26.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2$ = **85.5** mm

$c / t_f = 9.6 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor: η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force: $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **46.1** kN

Shear area - cl 6.2.6(3): $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1802** mm²

Design shear resistance - cl 6.2.6(2): $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **369.3** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment: $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **43.2** kNm

Design bending resistance moment - eq 6.13: $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **178.1** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: k_c = **0.94**

$C_1 = \min(1 / k_c^2, 3.5)$ = **1.132**

Curvature factor: $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.815**

Poissons ratio: ν = **0.3**

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

Unrestrained length: $L = (1.2 \times L_{s1} + 2 \times h + 1.0 \times L_{s1}) / 2$ = **4328** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **338.6** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.725**

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: $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.753**

LTB reduction factor - eq 6.57: $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.857**

Modification factor: $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.970**

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Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.883}$
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{157.3}$ 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	$\delta_{lim} = L_{s1} / 360 = \mathbf{10.4}$ mm
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{4.757}$ mm

PASS - Maximum deflection does not exceed deflection limit

Beam – B2

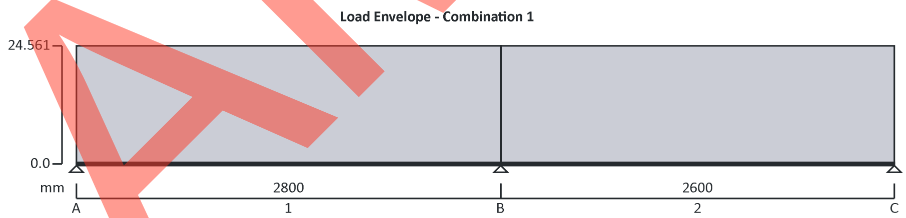
Beam span = 5.40 m

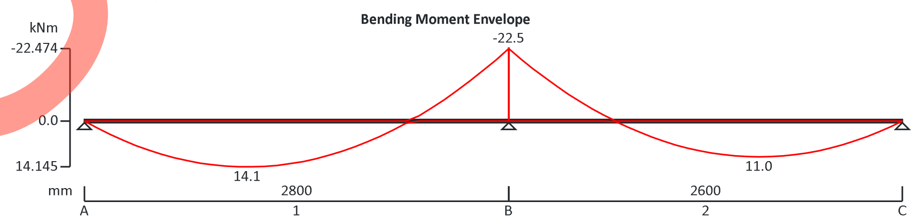
		Area Load	Loaded Width/Height	UDL unfactored
Roof				
Sloping rafters	Dead	1.48 kN/m^2	2.00 m	2.97 kN/m
	Live	0.85 kN/m^2	2.00 m	1.70 kN/m
Roof				
Flat rafters	Dead	0.85 kN/m^2	1.25 m	1.06 kN/m
	Live	1.00 kN/m^2	1.25 m	1.25 kN/m
Wall				
Ext Solid wall	Dead	4.17 kN/m^2	2.50 m	10.43 kN/m
	Live	0.00 kN/m^2	2.50 m	0.00 kN/m

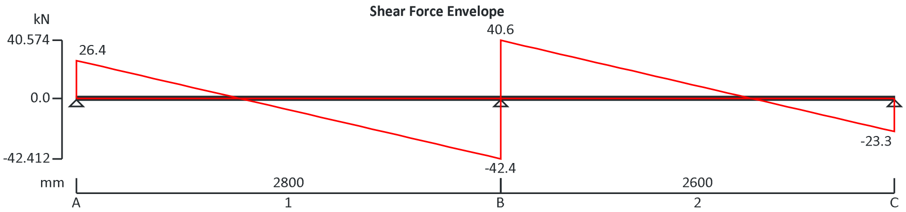
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 14.46 kN/m
	Variable full UDL 2.95 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
		Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50

Analysis results

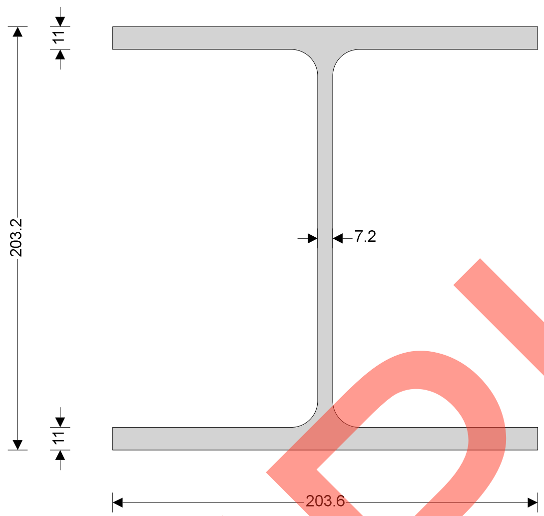
Maximum moment	M_{max} = **14.1** kNm	M_{min} = **-22.5** kNm
Maximum moment span 1	M_{s1_max} = **14.1** kNm	M_{s1_min} = **-22.5** kNm
Maximum moment span 2	M_{s2_max} = **11** kNm	M_{s2_min} = **-22.5** kNm
Maximum shear	V_{max} = **40.6** kN	V_{min} = **-42.4** kN
Maximum shear span 1	V_{s1_max} = **26.4** kN	V_{s1_min} = **-42.4** kN
Maximum shear span 2	V_{s2_max} = **40.6** kN	V_{s2_min} = **-23.3** kN
Deflection	δ_{max} = **0.7** mm	δ_{min} = **0** mm
Deflection span 1	δ_{s1_max} = **0.7** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0.4** mm	δ_{s2_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **26.4** kN	R_{A_min} = **26.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **16** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.2** kN	
Maximum reaction at support B	R_{B_max} = **83** kN	R_{B_min} = **83** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **50.4** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **10** kN	
Maximum reaction at support C	R_{C_max} = **23.3** kN	R_{C_min} = **23.3** kN
Unfactored permanent load reaction at support C	$R_{C\_Permanent}$ = **14.1** kN	
Unfactored variable load reaction at support C	$R_{C\_Variable}$ = **2.8** kN	

Section details

Section type	**UC 203x203x46 (British Steel Section Range 2022 (BS4-1))**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w) =$ **11.0** mm
Nominal yield strength	$f_y =$ **355** N/mm^2
Nominal ultimate tensile strength	$f_u =$ **470** N/mm^2
Modulus of elasticity	$E =$ **210000** N/mm^2



Partial factors - Section 6.1

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

Lateral restraint

Span 1 has lateral restraint at supports only

Span 2 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	$K_{LT.A} =$ **1.200** $+ 2 \times h$
	$K_{LT.B} =$ **1.000**
	$K_{LT.C} =$ **1.000**

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d =$ **155.8** mm	
	$c / t_w = 26.6 \times \varepsilon <= 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 =$ **85.5** mm	
	$c / t_f = 9.6 \times \varepsilon <= 10 \times \varepsilon$	Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **181.2** mm
Shear area factor	$\eta =$ **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{42.4}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1802}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{369.3}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{22.5}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{178.1}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.706}$

$C_1 = \min(1 / k_c^2, 3.5) = \mathbf{2.004}$

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.815}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

Unrestrained length — $L = (1.2 \times L_{s1} + 2 \times h + 1.0 \times L_{s1}) / 2 = \mathbf{3283}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{913.7}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.441}$

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections — $\beta = \mathbf{0.75}$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.580}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.984}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.891}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{1.000}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{178.1}$ 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 — $\delta_{lim} = L_{s1} / 360 = \mathbf{7.8}$ mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.67}$ mm

PASS - Maximum deflection does not exceed deflection limit

Padstone – P1

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	82.0 kN	37.0 kN	1.88 N/mm^2	3.19 N/mm^2	0.590	Pass

Masonry panel details

Panel length	L = **1000** mm	Panel height	h = **2700** mm
Load bearing leaf thickness	t = **100** mm	Effective height	h_{ef} = **2700** mm
Effective thickness	t_{ef} = **100** mm		

Masonry material details

Unit type	**Clay - Group 1**		
Mean masonry strength	f_b = **21.0** N/mm^2	Specific weight of units	γ = **22** kN/m^3
Mortar type	**M4 - General Purpose**	Mortar strength	f_m = **4.0** N/mm^2
Characteristic strength	f_k = **6.38** N/mm^2		

Design compressive strength of masonry

Category of manufacturing	**Category II**	Class of execution control	**Class 2**
Partial factor for material	γ_M = **3.00**	Design strength of masonry	f_d = **2.13** N/mm^2

Partial safety factors for design loads

Permanent partial factor	γ_{fG} = **1.35**	Variable partial factor	γ_{fQ} = **1.50**

Superimposed vertical loading details

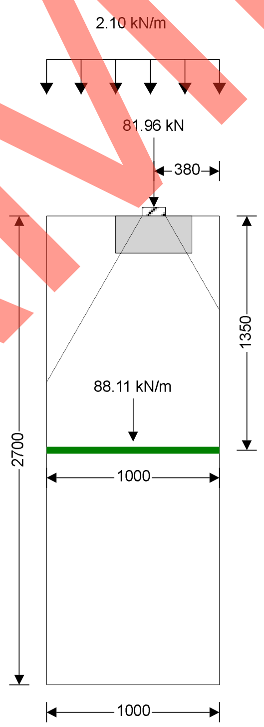
Permanent UDL at top of wall	g_k = **1.00** kN/m	Variable UDL at top of wall	q_k = **0.50** kN/m
Eccent. of permanent UDL	e_{gu} = **0** mm	Eccent. of variable UDL	e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	λ_{lim} = **27**	Slenderness ratio	λ = **27.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent load	G_{kc1} = **49.60** kN	Variable load	Q_{kc1} = **10.00** kN
Eccentricity of load	e_{c1} = **0** mm	Length of load	L_{c1} = **133** mm
Width of load	w_{c1} = **100** mm	Height of load	h_{c1} = **2700** mm
Distance to right edge	r_{11} = **314** mm	Distance to nearest edge	a_{11} = **314** mm

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Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load	N_{Edc1} = **81.96** kN	Design resistance	N_{Rdc1} = **37.03** kN

Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam

Type of spreader	**Concrete padstone**	Type of load	**Point load**
Length of spreader	L_{sp1} = **440** mm	Width of spreader	w_{sp1} = **100** mm
Height of spreader	h_{sp1} = **215** mm	Eccentricity of load	e_{sp1} = **0** mm
Modulus of elasticity	E_{sp1} = **29962** N/mm^2	Maximum moment	M_{Edsp1} = **4.49** kNm
Maximum shear	V_{Edsp1} = **40.98** kN	Allowable stress	σ_{Rdsp1} = **3.19** N/mm^2
Design stress	σ_{Edsp1} = **1.88** N/mm^2		

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section 6.1.2

Vertical load at mid-height	N_{Ed1} = **88.11** kN/m	Design resistance	N_{Rd1} = **91.32** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

Beam – B3

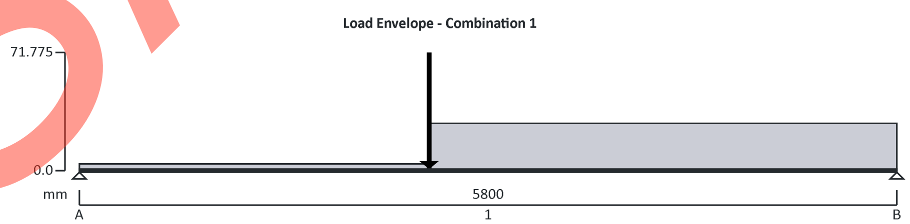
Beam span = 5.80 m

		Area Load	Loaded Width/Height	UDL, unfactored
Roof				
Sloping rafters	Dead	1.48 kN/m^2	2.00 m	2.97 kN/m
	Live	0.85 kN/m^2	2.00 m	1.70 kN/m
1st Floor				
Intermediate floors	Dead	0.55 kN/m^2	2.90 m	1.60 kN/m
	Live	2.00 kN/m^2	2.90 m	5.80 kN/m
Wall				
Int brick wall	Dead	2.80 kN/m^2	2.70 m	7.56 kN/m
	Live	0.00 kN/m^2	2.70 m	0.00 kN/m

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

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

TEDDS calculation version 3.0.14



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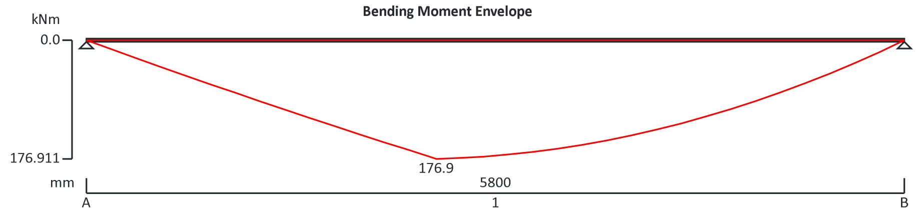
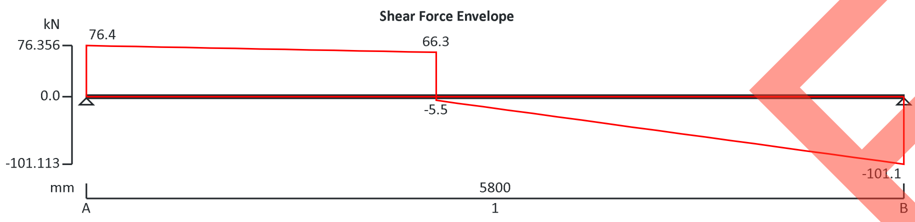




Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent partial UDL 12.12 kN/m from 2480 mm to 5800 mm
	Variable partial UDL 7.5 kN/m from 2480 mm to 5800 mm
	Permanent partial UDL 1 kN/m from 0 mm to 2480 mm
	Variable partial UDL 1 kN/m from 0 mm to 2480 mm
	B1 - Permanent point load 27.7 kN at 2480 mm
	Variable point load 5.5 kN at 2480 mm
	B2 - Permanent point load 15.8 kN at 2480 mm
	Variable point load 3.2 kN at 2480 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} = **176.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **176.9** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **172.1** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **76.4** kN	V_{min} = **-101.1** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **76.4** kN	$V_{s1\_seg1\_min}$ = **-17.6** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-101.1** kN
Deflection segment 3	δ_{max} = **13.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **76.4** kN	R_{A_min} = **76.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **40.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **14.1** kN	

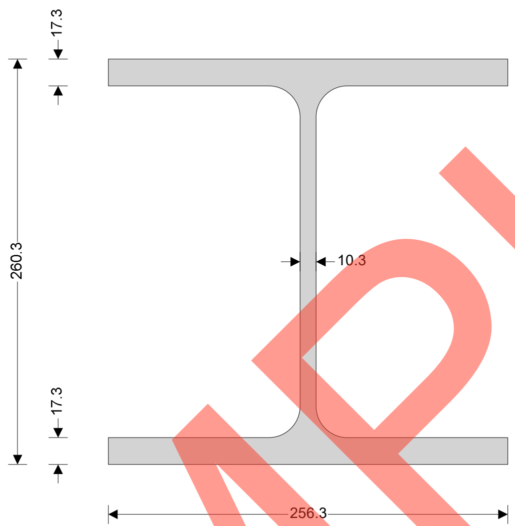
Maximum reaction at support B R_{B_max} = **101.1** kN R_{B_min} = **101.1** kN
Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **50.4** kN
Unfactored variable load reaction at support B $R_{B\_Variable}$ = **22** kN

Section details

Section type **UC 254x254x89 (British Steel Section Range 2022 (BS4-1))**
Steel grade **S355**

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

Nominal thickness of element $t = \max(t_f, t_w)$ = **17.3** mm
Nominal yield strength f_y = **345** N/mm^2
Nominal ultimate tensile strength f_u = **470** 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 midspan

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 $K_{LT.A}$ = **1.200** $+ 2 \times h$
$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

Width of section c = d = **185.7** mm
$c / t_w = 21.8 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **103** mm
$c / t_f = 7.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **225.7** mm

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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})) = **101.1** kN

Shear area - cl 6.2.6(3) — A_v = max(A - 2 × b × t_f + (t_w + 2 × r) × t_f, η × h_w × t_w) = **3538** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **704.8** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — M_{Ed} = max(abs($M_{s1\_seg1\_max}$), abs($M_{s1\_seg1\_min}$)) = **176.9** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **429.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.823**

C_1 = min(1 / k_c^2, 3.5) = **1.478**

Curvature factor — g = $\sqrt{[1 - (I_z / I_y)]}$ = **0.815**

Poissons ratio — ν = **0.3**

Shear modulus — G = E / [2 × (1 + ν)] = **80769** N/mm^2

Unrestrained length — L = (1.2 × L_{s1_seg1} + 2 × h + 1.0 × L_{s1_seg1}) / 2 = **3450** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **2454.2** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.418**

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.569**

LTB reduction factor - eq 6.57 — χ_{LT} = min(1 / [$\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}$], 1, 1 / $\bar{\lambda}_{LT}^2$) = **0.993**

Modification factor — f = min(1 - 0.5 × (1 - k_c)× [1 - 2 × ($\bar{\lambda}_{LT}$ - 0.8)^2], 1) = **0.937**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod}$ = min(χ_{LT} / f, 1) = **1.000**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **429.8** 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 — $\delta_{lim} = L_{s1} / 360$ = **16.1** mm

Maximum deflection span 1 — δ = max(abs(δ_{max}), abs(δ_{min})) = **13.397** mm

PASS - Maximum deflection does not exceed deflection limit

Padstone – P2

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	101.0 kN	119.7 kN	N/A	N/A	0.844	Pass

Masonry panel details

Panel length L = **975** mm
Panel height h = **2700** mm
Load bearing leaf thickness t = **100** mm
Effective height h_{ef} = **2700** mm
Effective thickness t_{ef} = **100** mm

Masonry material details

Unit type **Clay - Group 1**
Mean masonry strength f_b = **51.0** N/mm^2
Specific weight of units γ = **22** kN/m^3
Mortar type **M4 - General Purpose**
Mortar strength f_m = **4.0** N/mm^2
Characteristic strength f_k = **11.88** N/mm^2

Design compressive strength of masonry

Category of manufacturing **Category II**
Class of execution control **Class 2**
Partial factor for material γ_M = **3.00**
Design strength of masonry f_d = **3.93** N/mm^2

Partial safety factors for design loads

Permanent partial factor γ_{fG} = **1.35**
Variable partial factor γ_{fQ} = **1.50**

Superimposed vertical loading details

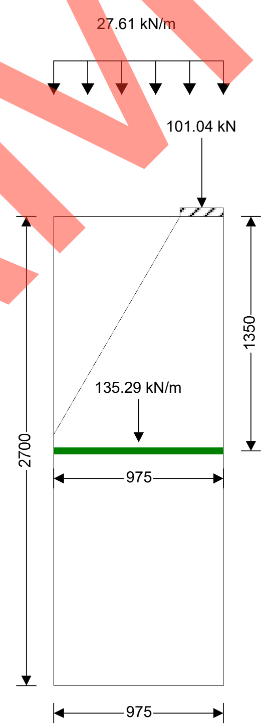
Permanent UDL at top of wall g_k = **12.12** kN/m
Variable UDL at top of wall q_k = **7.50** kN/m
Eccent. of permanent UDL e_{gu} = **0** mm
Eccent. of variable UDL e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit λ_{lim} = **27**
Slenderness ratio λ = **27.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent load G_{kc1} = **50.40** kN
Variable load Q_{kc1} = **22.00** kN
Eccentricity of load e_{c1} = **0** mm
Length of load L_{c1} = **250** mm
Width of load w_{c1} = **100** mm
Height of load h_{c1} = **2700** mm
Distance to right edge r_{11} = **0** mm
Distance to nearest edge a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load N_{Edc1} = **101.04** kN Design resistance N_{Rdc1} = **119.69** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section 6.1.2

Vertical load at mid-height N_{Ed1} = **135.29** kN/m Design resistance N_{Rd1} = **168.67** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

Roof Joists – RJ

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

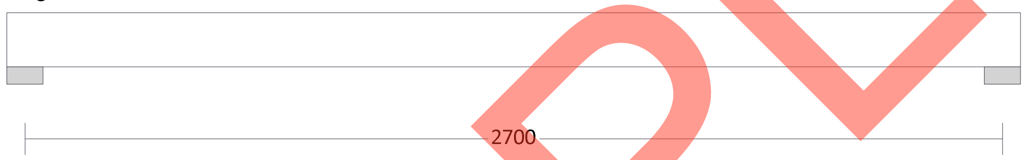
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.07

Joist details

Description; 47 x 150 C16 timber joists

Joist spacing; s_{Joist} = **400** mm



Forces input on Joist

Vertical permanent load on joist; F_{G_Joist}= **0.80** kN/m^2

Vertical imposed load on joist; F_{Q_Joist}= **0.75** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist; $p_G = F_{G\_Joist} \times s_{Joist}$= **0.32** kN/m

Vertical imposed load on joist; $p_Q = F_{Q\_Joist} \times s_{Joist}$= **0.30** kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm^2	1.5	0.2	0.135	PASS
Bending stress	N/mm^2	10.8	4.7	0.435	PASS
Shear stress	N/mm^2	2.2	0.4	0.180	PASS
Deflection	mm	10.8	7.9	0.733	PASS

ANALYSIS

Tedds calculation version 1.0.37

Loading

Self weight included (Permanent x 1)

Load combination factors

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

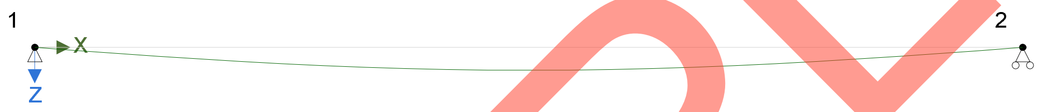
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.32 kN/m at 0 m to 2.7 m
Member	Imposed	UDL	GlobalZ	0.3 kN/m at 0 m to 2.7 m

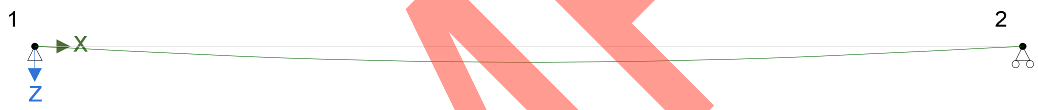
<u>Results</u>

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.40477	
2	0	0	-0.40477	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.28502	
2	0	0	-0.28502	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	2.5
1.00G + 1.00Q (Service)	0	1.7

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

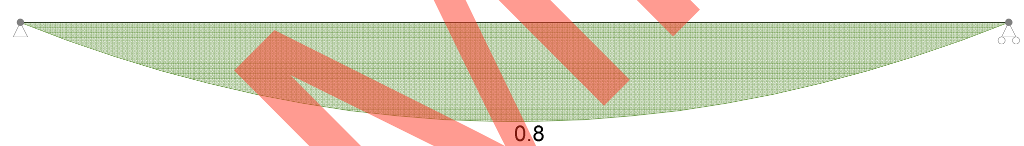
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.7	1	0	-1.2	0
		2	0	-1.2	0

Load combination: 1.00G + 1.00Q (Service)

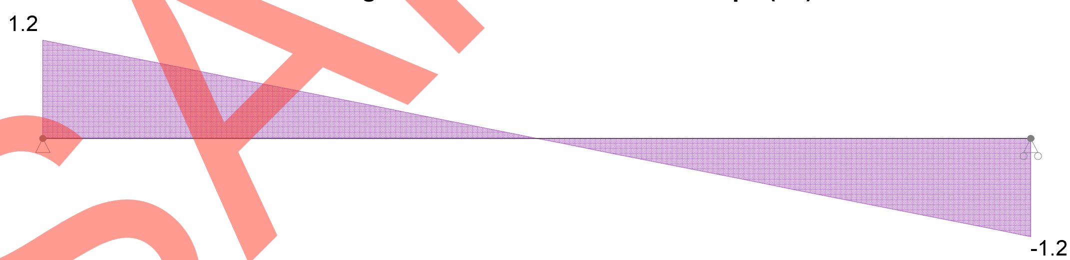
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.7	1	0	-0.9	0
		2	0	-0.9	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	1.2 (max abs)		0 (min)	
	1.35	0		0.8 (max)	
	2.7	-1.2		0 (min)	

;

Tedds calculation version 2.2.20

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2; Medium-term

Service class - cl.2.3.1.3; 2

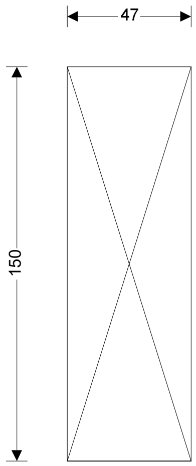
Timber section details

Number of timber sections in member; N = **1**

Breadth of sections; b = **47** mm

Depth of sections; h = **150** mm

Timber strength class - EN 338:2016 Table 1; **C16**



47x150 timber section
Cross-sectional area, A, 7050 mm^2
Section modulus, W_y, 176250 mm^3
Section modulus, W_z, 55225 mm^3
Second moment of area, I_y, 13218750 mm^4
Second moment of area, I_z, 1297787 mm^4
Radius of gyration, i_y, 43.3 mm
Radius of gyration, i_z, 13.6 mm
Timber strength class C16
Characteristic bending strength, $f_{m.k}$, 16 N/mm^2
Characteristic shear strength, $f_{v.k}$, 3.2 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 17 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.2 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 8.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 310 kg/m^3
Mean density, ρ_{mean}, 370 kg/m^3

Span details

Bearing length; L_b = **100** mm

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

Modification factors

Duration of load and moisture content - Table 3.1; k_{mod} = **0.8**

Deformation factor - Table 3.2; k_{def} = **0.8**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.67**

System strength factor - cl.6.6; k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis; $F_{c,y,90,d}$ = **1.23** kN

Effective contact length; $L_{b,ef} = L_b + \min(L_b, 30\text{ mm})$ = **130** mm

Design perpendicular compressive stress - exp.6.4; $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.201** N/mm^2

Design perpendicular compressive strength; $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.489** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.135**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force; $F_{y,d}$ = **1.23** kN

Design shear stress - exp.6.60; $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.391** N/mm^2

Design shear strength; $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.166** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.180**

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Integrity | Knowledge | Teamwork

PASS - Design shear strength exceeds design shear stress

Check design 1350 mm along span

Check bending moment - Section 6.1.6

Design bending moment; $M_{y,d}$ = **0.83** kNm

Design bending stress; $\sigma_{m,y,d} = M_{y,d} / W_y$ = **4.71** N/mm^2

Design bending strength; $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **10.831** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.435**

PASS - Design bending strength exceeds design bending stress

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

Check design 1350 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection; δ_y = **4.4** mm

Quasi-permanent variable load factor; ψ_2 = **0.3**

Final deflection with creep; $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **7.9** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **10.8** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.733**

PASS - Allowable deflection exceeds final deflection

Floor Joists - FJ

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

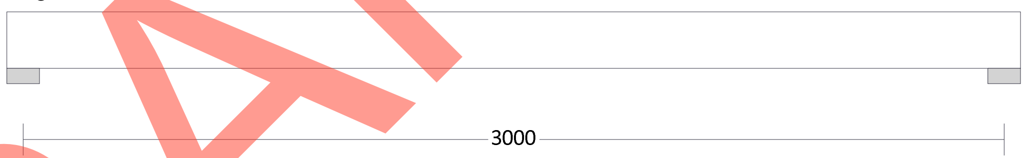
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.07

Joist details

Description; 47 x 175 C24 timber joists

Joist spacing; s_{Joist} = **400** mm



Forces input on Joist

Vertical permanent load on joist; F_{G_Joist}= **0.60** kN/m^2

Vertical imposed load on joist; F_{Q_Joist}= **2.00** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist; $p_G = F_{G\_Joist} \times s_{Joist}$= **0.24** kN/m

Vertical imposed load on joist; $p_Q = F_{Q\_Joist} \times s_{Joist}$= **0.80** kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm^2	1.7	0.4	0.228	PASS
Bending stress	N/mm^2	16.2	7.4	0.453	PASS

Shear stress	N/mm^2	2.7	0.6	0.237	PASS
Deflection	mm	12	9.3	0.774	PASS

ANALYSIS

Tedds calculation version 1.0.37

Loading

Self weight included (Permanent x 1)

Load combination factors

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

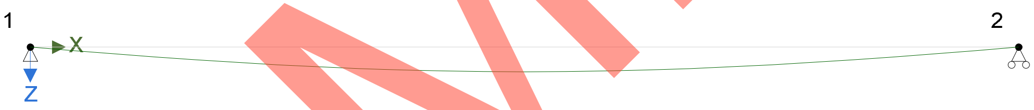
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.24 kN/m at 0 m to 3 m
Member	Imposed	UDL	GlobalZ	0.8 kN/m at 0 m to 3 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection

Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.43821	
2	0	0	-0.43821	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.29978	
2	0	0	-0.29978	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.7
1.00G + 1.00Q (Service)	0	3.2

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

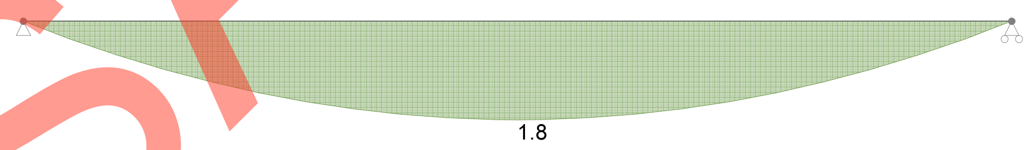
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	0	-2.4	0
		2	0	-2.4	0

Load combination: 1.00G + 1.00Q (Service)

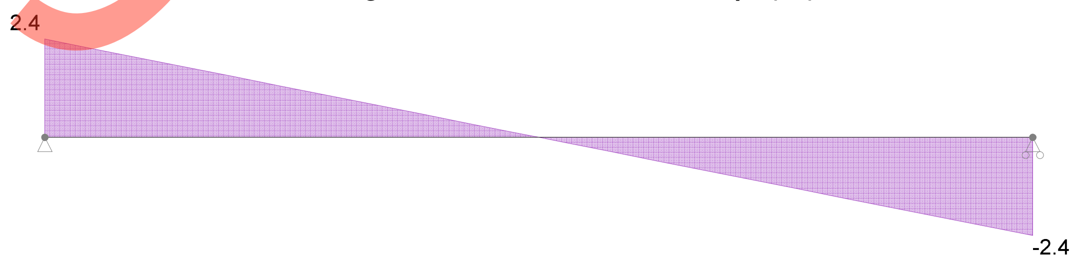
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	0	-1.6	0
		2	0	-1.6	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	2.4 (max abs)		0 (min)	
	1.5	0		1.8 (max)	
	3	-2.4		0 (min)	

;

Tedds calculation version 2.2.20

<u>Member - Span 1</u>

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2; Medium-term

Service class - cl.2.3.1.3; 2

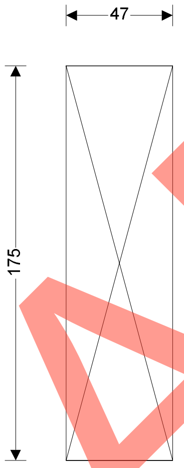
Timber section details

Number of timber sections in member; N = **1**

Breadth of sections; b = **47** mm

Depth of sections; h = **175** mm

Timber strength class - EN 338:2016 Table 1; **C24**



47x175 timber section

Cross-sectional area, A, 8225 mm^2

Section modulus, W_y, 239895.8 mm^3

Section modulus, W_z, 64429 mm^3

Second moment of area, I_y, 20990885 mm^4

Second moment of area, I_z, 1514085 mm^4

Radius of gyration, i_y, 50.5 mm

Radius of gyration, i_z, 13.6 mm

Timber strength class C24

Characteristic bending strength, $f_{m.k}$, 24 N/mm^2

Characteristic shear strength, $f_{v.k}$, 4 N/mm^2

Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2

Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2

Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2

Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2

Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2

Shear modulus of elasticity, G_{mean}, 690 N/mm^2

Characteristic density, ρ_k, 350 kg/m^3

Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length; L_b = **100** mm

<u>Consider Combination 1 - 1.35G + 1.50Q (Strength)</u>

Modification factors

Duration of load and moisture content - Table 3.1; k_{mod} = **0.8**

Deformation factor - Table 3.2; k_{def} = **0.8**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.67**

System strength factor - cl.6.6; k_{sys} = **1.1**

<u>Check design at start of span</u>

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis; $F_{c,y,90,d}$ = **2.355** kN

Effective contact length; $L_{b,ef} = L_b + \min(L_b, 30\ mm) =$ **130** mm

Design perpendicular compressive stress - exp.6.4; $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) =$ **0.385** N/mm^2

Design perpendicular compressive strength; $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M =$ **1.692** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) =$ **0.228**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force; $F_{y,d} =$ **2.355** kN

Design shear stress - exp.6.60; $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) =$ **0.641** N/mm^2

Design shear strength; $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M =$ **2.708** N/mm^2

$\tau_{y,d} / f_{v,y,d} =$ **0.237**

PASS - Design shear strength exceeds design shear stress

<u>Check design 1500 mm along span</u>

Check bending moment - Section 6.1.6

Design bending moment; $M_{y,d} =$ **1.766** kNm

Design bending stress; $\sigma_{m,y,d} = M_{y,d} / W_y =$ **7.361** N/mm^2

Design bending strength; $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M =$ **16.246** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d} =$ **0.453**

PASS - Design bending strength exceeds design bending stress

<u>Consider Combination 2 - 1.00G + 1.00Q (Service)</u>

<u>Check design 1500 mm along span</u>

Check y-y axis deflection - Section 7.2

Instantaneous deflection; $\delta_y =$ **5.2** mm

Quasi-permanent variable load factor; $\psi_2 =$ **0.3**

Final deflection with creep; $\delta_{y,Final} = \delta_y \times (1 + k_{def}) =$ **9.3** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1\_s1} / 250 =$ **12** mm

$\delta_{y,Final} / \delta_{y,Allowable} =$ **0.774**

PASS - Allowable deflection exceeds final deflection