

STRUCTURAL CALCULATIONS

3 Jubilee Garden, High Peak

Reference	1300-XX-CA-S-0630-S2-P01
Date	31/10/2025
Revision	P01
Author	CW

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APPENDICES

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1.0 INTRODUCTION

1.1 Instruction

i have been instructed to design structural support for a new loft conversion in an existing property.

1.2 Objectives

The objective of this set of calculations is to provide structural support requirements for the super structures.

1.3 Scope of Work

This report covers the calculations for assessing the requirements for the new steelwork.

1.4 Site Location

The property is located at 3 Jubilee Gardens.

1.5 Limitations and Exclusions

This report covers the calculations for the referenced items only.

2.0 OBSERVATIONS & CALCULATIONS

General loading information is provided below;

Dead loads

Floor Joists – 0.75kN/m^2

Roof Construction – 1.25kN/m^2

Imposed – 2.0kN/m^2

Roof Imposed – 0.6kN/m^2

Steelwork design

B1

Max Span – 5200mm

Dead = 6.75kN/m

Imposed = 2.7kN/m

Use 254x102x25UB (S355) With 300mm LNG x 215DP x 100 WD padstones.

Timber design

New Rafters

Max Span – 4600mm

Dead Roof = 1.25kN/m^2

Imposed Roof = 0.6kN/m^2

Use 2 No. 50wdx150dp C24 timbers @ 400 c/c fixed together with M8 bolts @ 600 c/c with square washers and dogtooth connectors.

New Joists

Max Span – 2950mm

Dead Roof = 0.65kN/m^2

Imposed Roof = 2.0kN/m^2

Use 50wdx150dp C24 Timbers @ 400 c/c

New Dormer Rafters

Max Span – 3100mm

Dead Roof = 1.25kN/m^2

Imposed Roof = 0.6kN/m^2

Use 50wdx150dp C24 Timbers @ 400 c/c

New Dormer Studs

Max Span – 2250mm

Dead Roof = 1.25kN/m^2

Imposed Roof = 0.6kN/m^2

Use 50wdx150dp C24 Timbers @ 400 c/c

Joists to Dormer Stud Support.

Max Span – 3100mm

Dead Roof = 1.95kN/m

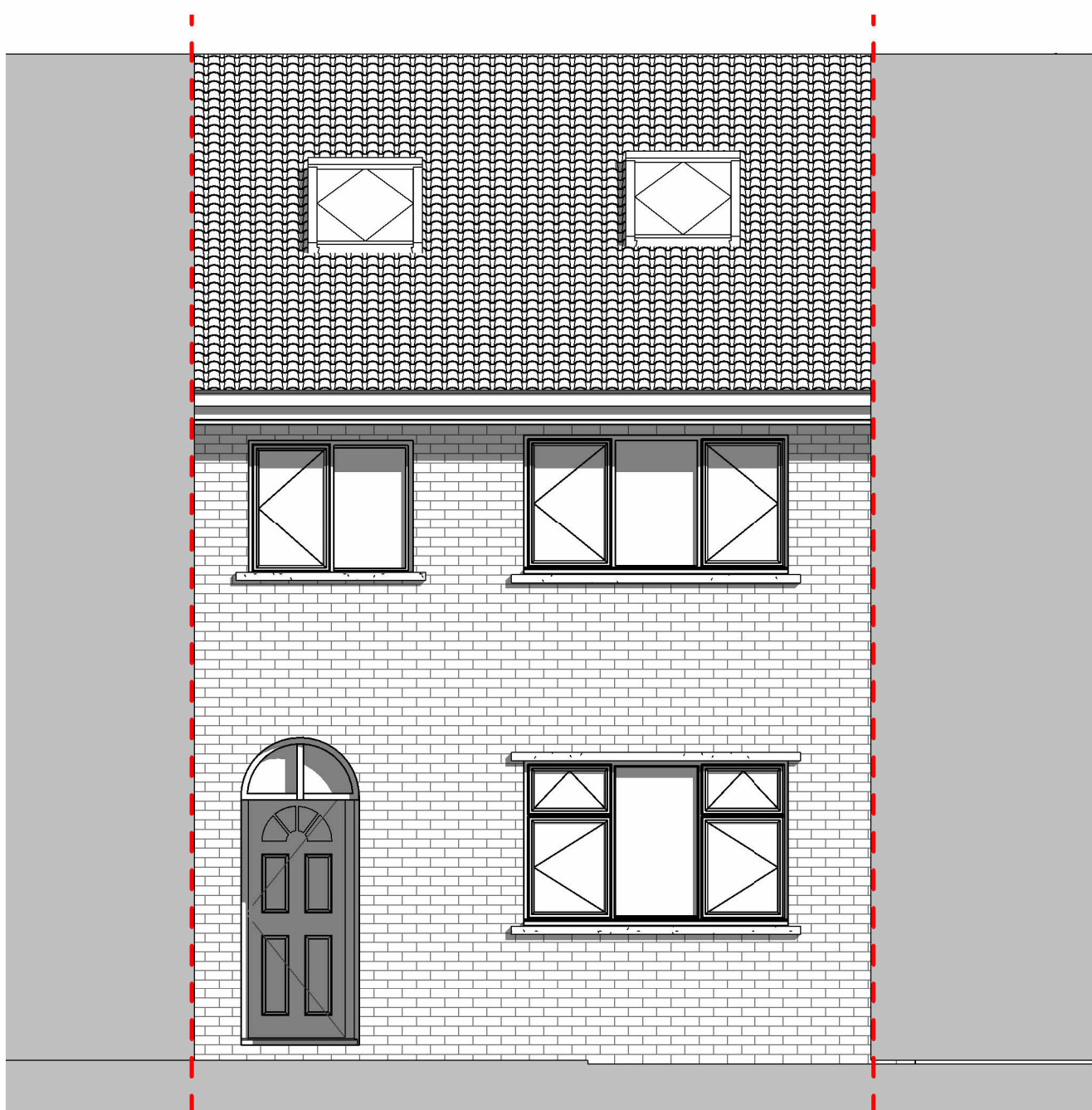
Imposed Roof = 1kN/m

Use 3 No. 50wdx150dp C24 timbers fixed together with M8 bolts @ 600 c/c with square washers and dogtooth connectors.

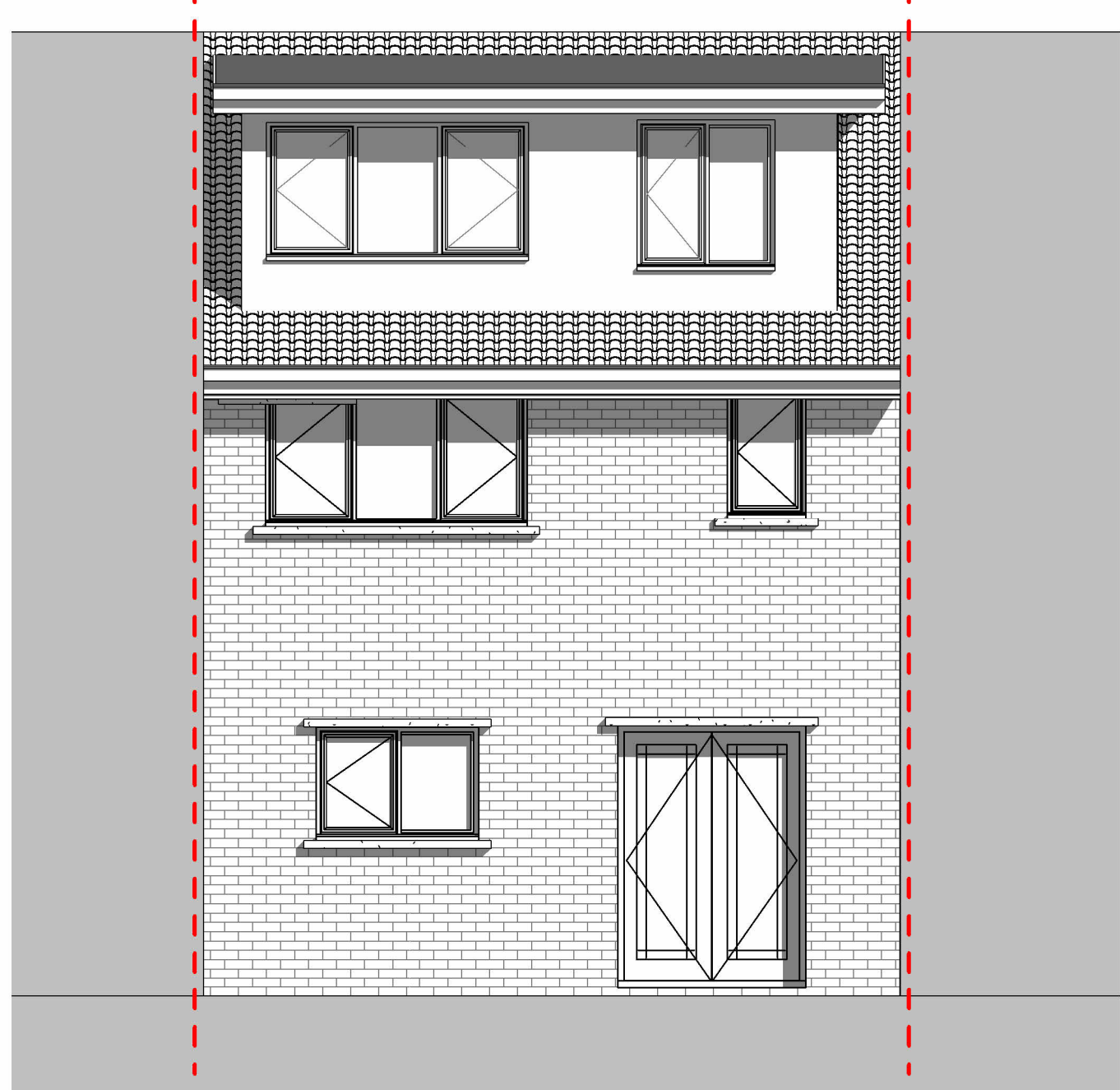
C Wright
MEng (Hons)

October 2025

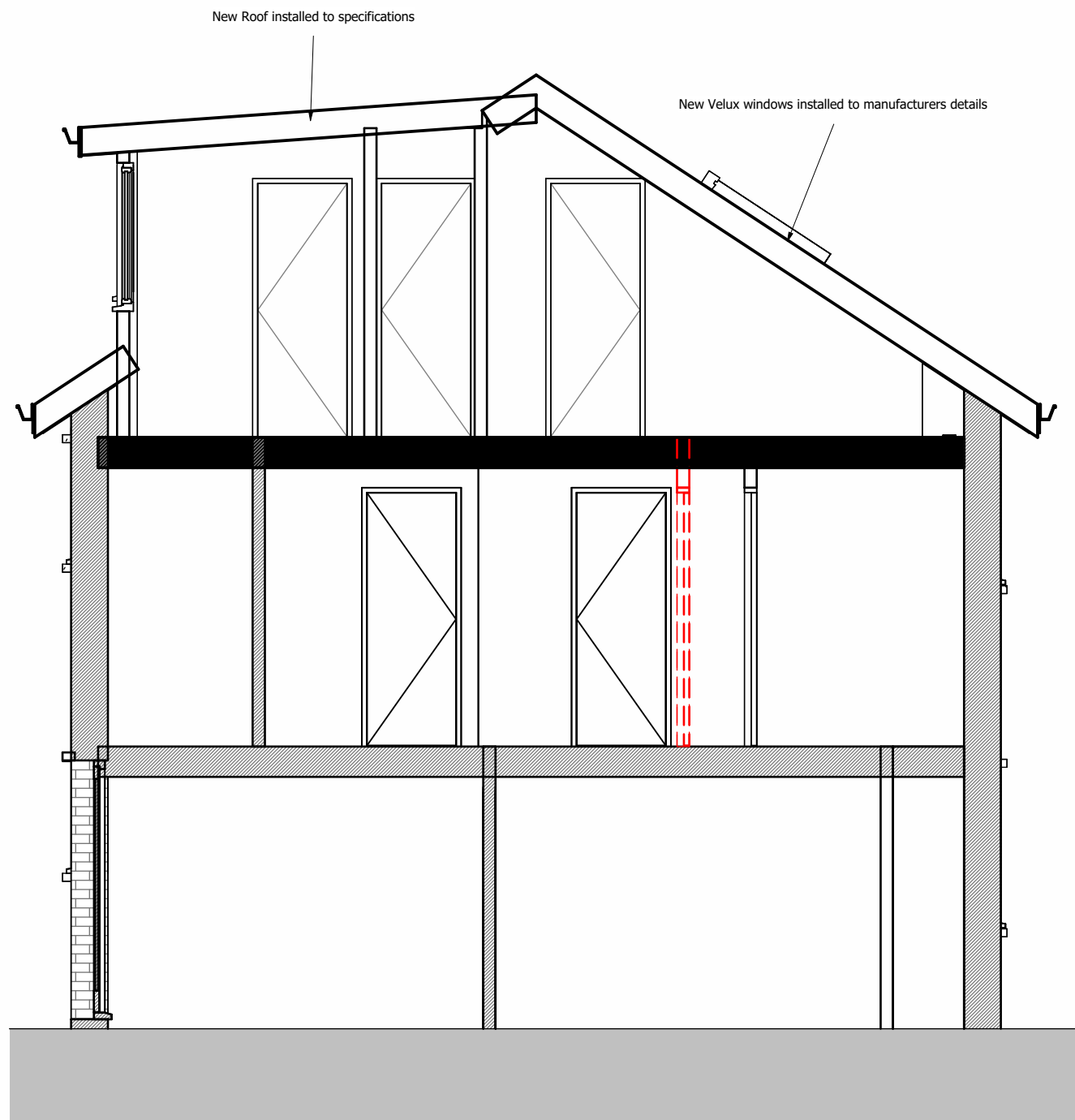
Appendix A



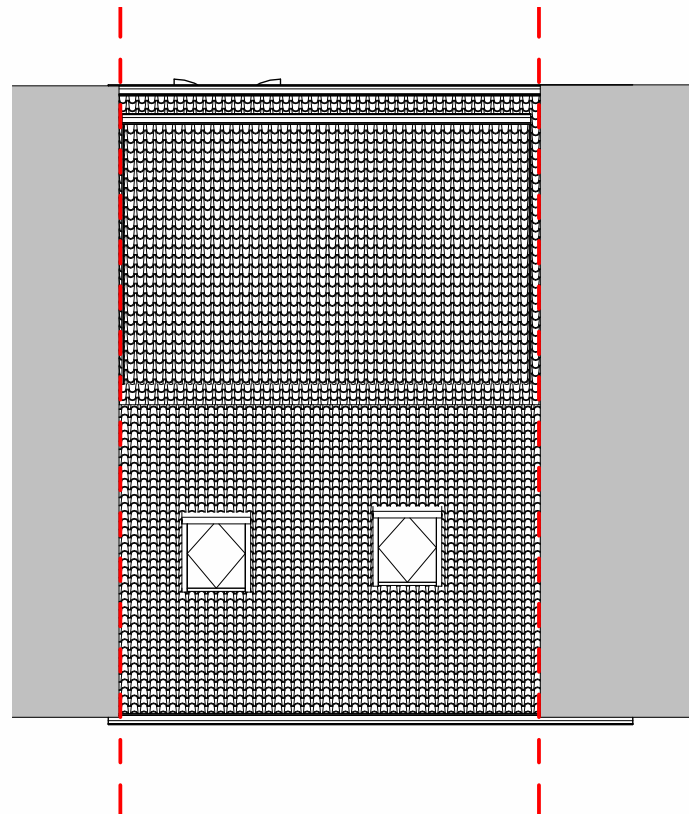
Front Elevation
1 : 50



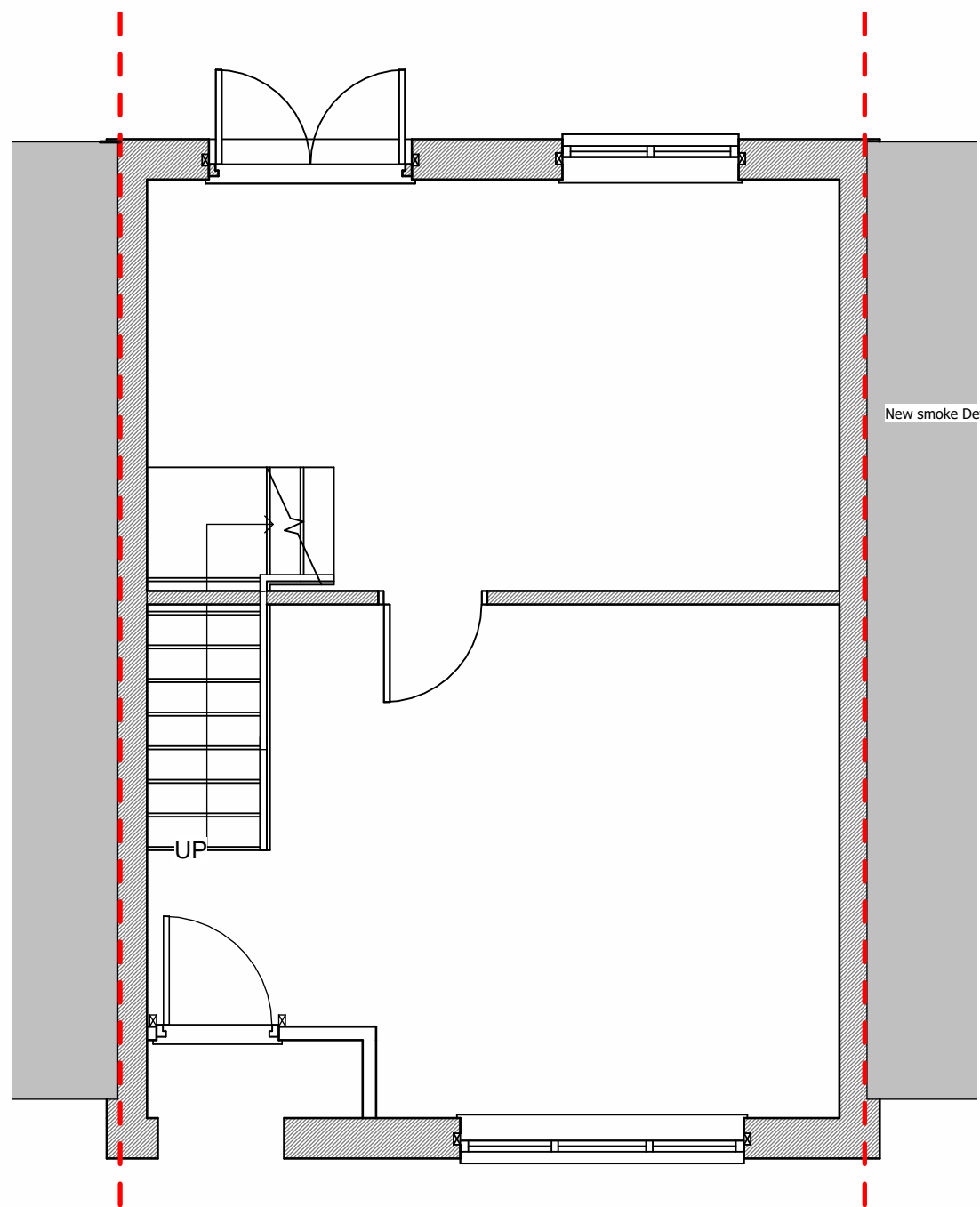
Rear Elevation
1 : 50



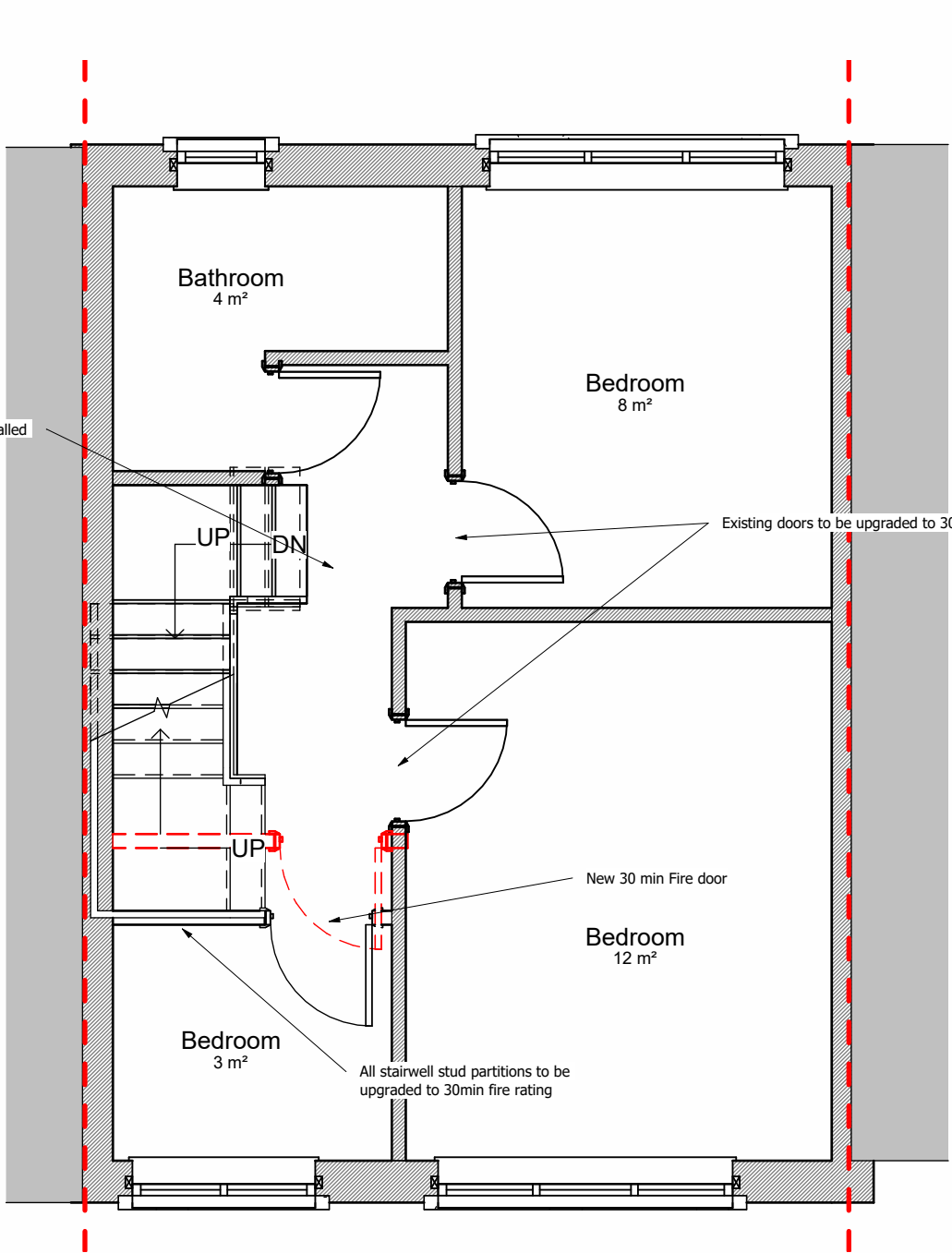
Section
1 : 50



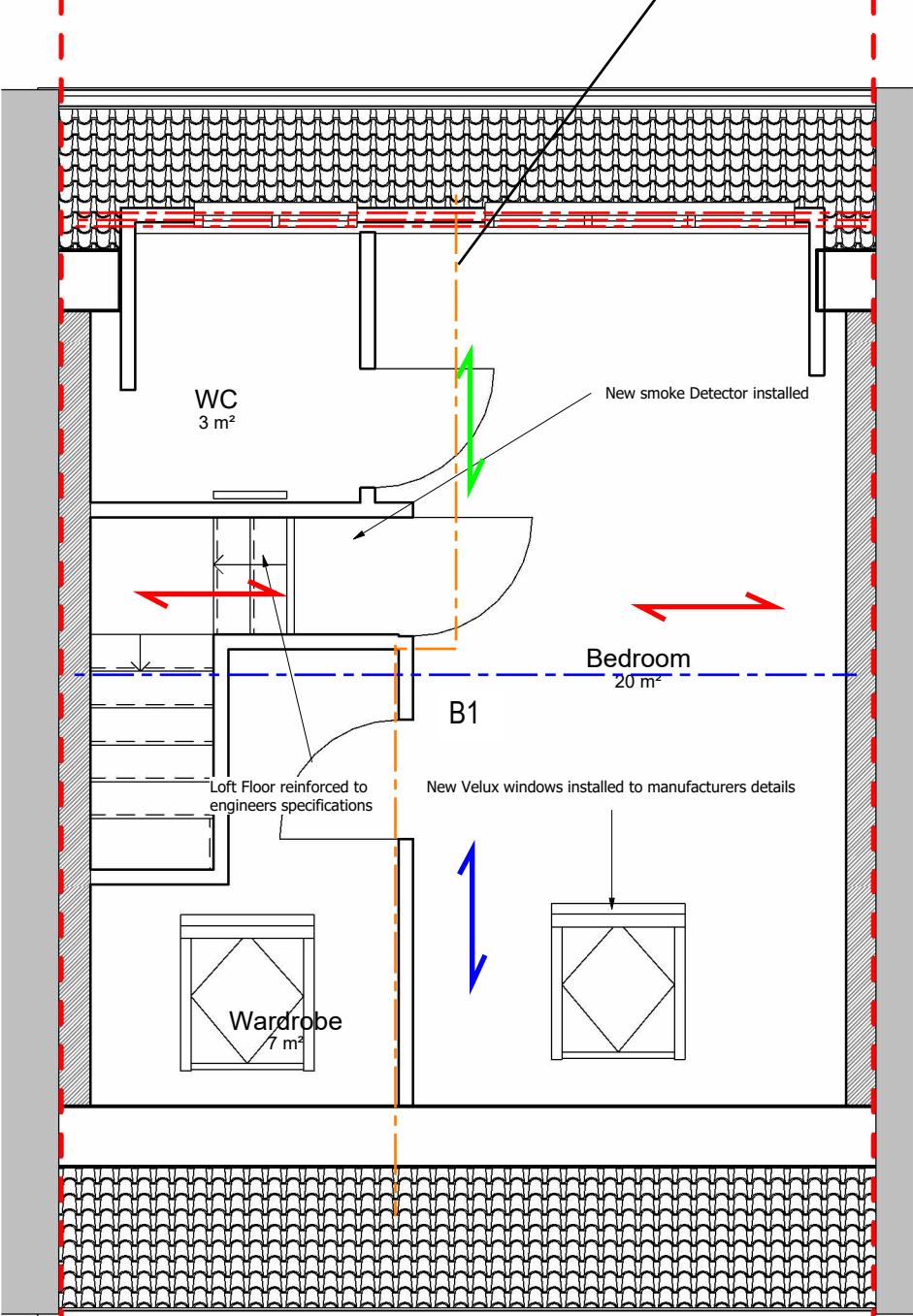
Roof Plan
1 : 100



Ground Floor Plan
1 : 50



First Floor Plan
1 : 50



Loft Floor Plan
1 : 50

THERMAL BRIDGING
Care shall be taken to limit the occurrence of thermal bridging in the insulation layers caused by gaps within the thermal element, (i.e. around windows and door openings).

MATERIALS AND WORKMANSHIP
All works are to be carried out in a workmanlike manner. All materials and workmanship must comply with Regulation 7 of the Building Regulations, all relevant British Standards, European Standards, Agreement Certificates, Product Certification of Scheme (like Marks) etc. Products conforming to a European technical standard or harmonised European product should have a CE marking. The latest edition of the British Standard (including any amendments) applies to any undated references within these specifications.

EXISTING STRUCTURE
Existing structure includes foundations, floor, beams, walls, roof and lintels are to be exposed and checked for adequacy prior to commencement of work and as required by the Building Control Officer.

LEGEND

Steelwork

B1
Use 254x102x25UB (S355) With 300mm LNG x 215DP x 100 WD padstones.

Timber

New Rafters

Use 2 No. 50wdx150dp C24 timbers @ 400 c/c fixed together with M8 bolts @ 600 c/c with square washers and dogtooth connectors.

New Joists

Use 50wdx150dp C24 Timbers @ 400 c/c

New Dormer Rafters

Use 50wdx150dp C24 Timbers @ 400 c/c

New Dormer Studs

Use 50wdx150dp C24 Timbers @ 400 c/c

Joists to Dormer Stud Support.

Use 3 No. 50wdx150dp C24 timbers fixed together with M8 bolts @ 600 c/c with square washers and dogtooth connectors.

r load bearing elements before works commence on site. New steel beams to be rt to provide 1/2 hour fire resistance, as agreed with Building Control. All fire

l using 1200mm x 30mm x 5mm galvanised mild steel horizontal straps or other s. Provide solid nogging between rafters at strap positions. All wall plates to be 100 x +A1.2016) at maximum 2m centres.

x 30mm x 5mm galvanised mild steel straps or other approved, in compliance with oggins between joists at strap positions.

maximum 2.0m centres, straps to be fixed to internal wall faces.

atters to be provided to ensure vented and drained cavity (if required) fixed to eathing (or other approved). Ply fixed to treated timber frame studs constructed and calculations.
board over. Finish with 3mm skim coat of finishing plaster.
r walls built off existing masonry walls to have galvanised mild steel straps placed at ard internally to achieve 1/2 hour fire resistance from both sides.

current BBA or other approved accreditation. Roof covering to be laid in compliance by manufacturer) over 150mm Kingspan Thermaflex insulation.
3mm C24 flat roof joists at 400mm c/c's, max span 5.08m or as Structural Engineer's to 100 x 50mm wall plates and anchored to wall.

ONSIBILITY TO ASSESS YOUR DESIGN TO ASCERTAIN WHETHER ENGINEER'S INS IN FLOORS, CEILINGS AND ROOFS FOR DWELLINGS OR ASK YOUR BUILDING

Engineer's details and calculations. Insulation between and over studs; Insulation with 3mm skim coat of finishing plaster. All junctions to have water tight

Provide min 20mm 1 and g chipboard or timber board flooring. In areas such as must be laid upper most to allow easy identification. To upgrade to half hour fire between joists and extend to eaves. Chicken wire to be fixed to the joists with nails ver 2.5m to be strutted at mid span, use 38 x 38mm herringbone strutting or 38mm als with 1200mm x 30mm x 5mm galvanised mild steel straps or other approved in Provide 38mm wide x 4% depth solid nogging between joists at strap

compliance with BS EN 845-1 (+A1:2016) , at max 2.0m centres, straps to be taken across minimum 3 no. joists. Straps to be built into walls. Provide 38mm wide x 4% depth solid nogging between joists at strap positions.

INTERNAL STUD PARTITIONS
100mm x 50mm softwood treated timbers studs at 400mm ctrs with 50 x 100mm head and sole plates and solid intermediate horizontal nogging at 1/3 height or 450mm c/c's. Provide min 10kg/m³ density acoustic soundproof quilt tightly packed (e.g. 100mm Rockwool or Isovol mineral fibre sound insulation) in all voids the full depth of the stud. Partitions to be built off doubled up joists where partitions run parallel or provide nogging where at right angles. Walls fixed throughout with 12.5mm plasterboard with skim plaster finish. Plasterboard to be taped and jointed complete with loads and stops.

STAIRS
Dimensions to be checked and measured on site prior to fabrication of stairs. Timber stairs to comply with BS585 and with Part K of the Building Regulations. Max rise 220mm, min going 220mm. Two risers plus one going should be between 550 and 700mm. Tapered treads to have going in centre of tread at least the same as the going on the going on the straight. Min 50mm going of tapered treads measured at narrow end. Pitch not to exceed 42 degrees. The width and length of every landing should be at least as great as the smallest width of the flight. Doors which swing across a landing at the bottom of a flight should leave a clear space of at least 400mm across the full width of the flight. Cupboard doors may open across the top landing where the swing is a minimum of 400mm from the tread. Min 2.0m headroom measured vertically above pitch line of stairs and landings. Handrail on staircase to be 900mm above the pitchline, handrail to be at least one side if stairs are less than 1m wide and on both sides if they are wider. Ensure a clear width between handrails of minimum 600mm. Balustrade designed to be unclimbable and should contain no space through which a 100mm sphere could pass. Allow for all structure as designed by a Structural Engineer.

ELECTRICAL
All electrical work required to meet the requirements of Part P (electrical safety) must be designed, installed, inspected and tested by a Competent Person registered under a Competent Person Self Certification Scheme such as IRE certification Ltd, BS, NICEIC Certification Services or Zurich Ltd. An appropriate BS7671 Electrical Installation Certificate is to be issued for the work by a person competent to do so. A copy of a certificate will be given to Building Control on completion.

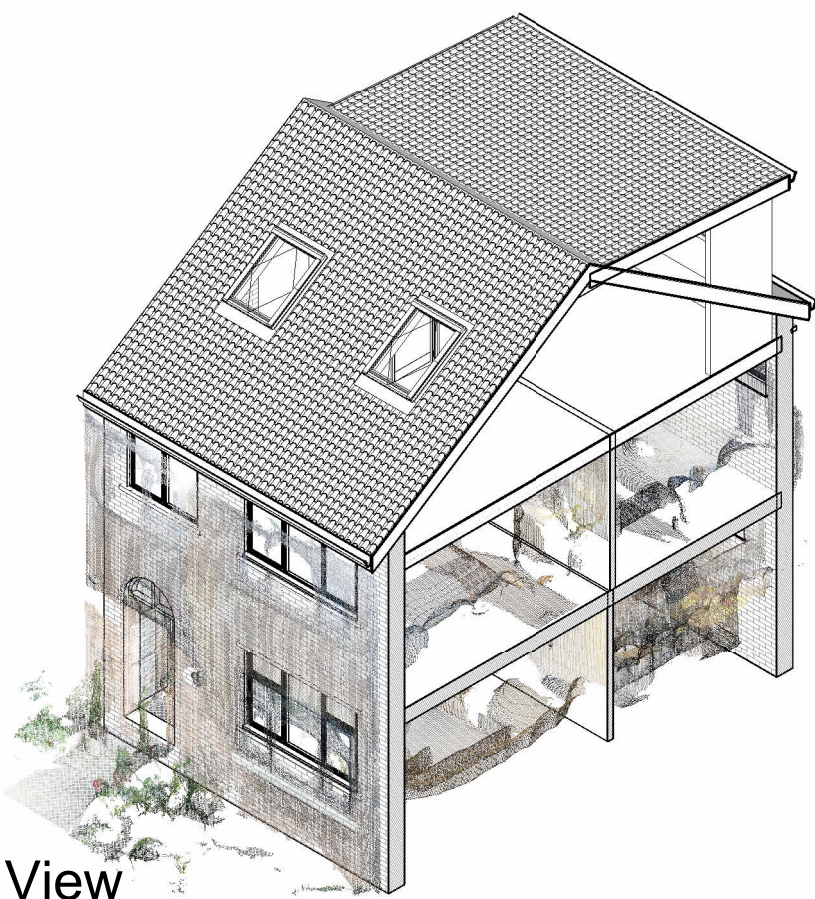
INTERNAL LIGHTING
Install low energy light fittings that only take lamps having a luminous efficacy better than 80 lumens per circuit watt. All fixed to have lighting capacity (lm) 185 x total floor area, to comply with Part L of the current Building Regulations and the Domestic Building Services Compliance Guide.

HEATING
Extend all heating and hot water services from existing and provide new TRVs to radiators. Heating system to be designed, installed, tested and fully certified by a GAS SAFE registered specialist. All work to be in accordance with the Local Water Authorities bye laws, the Gas Safety (Installation and Use) Regulations 1998 and IEE Regulations. The energy performance of the new components to be assessed. The results should be recorded and given to the building owner. All accessible pipes to be insulated to the standards in Table 4.4 Approved Document L.

MEANS OF ESCAPE - Fire doors
Form a protected escape stairway by providing half hour fire resistance to all partitions. Ceiling to be upgraded to provide half hour fire resistance or modified half hour fire resistance as appropriate. Stairway to be protected at all levels - from the loft room/rooms then leading directly to an external door at ground level (no inner rooms allowed). All doors on to the stairway must be FD20 rated fire doors to BS 476 or the European equivalent BS EN 1634 (fitted with intumescent strips related around sides to top of door or frame if required by BCO). Where applicable, any glazing in fire doors to be half hour fire resisting and glazing in the walls forming the escape route enclosure to have 30 minutes fire resistance to at least 1.1m above the floor level or stair pitch line.

SMOKE DETECTION
Provide a linked smoke alarm detection system to BS EN 14604 and BS 5839-6:2019 to at least a Grade D2 (Grade A for large house) category LD2 standard. System to be mains powered with battery back up. Smoke detectors to be provided to:
- Each hallway and landing
- Every principal living room
An interlinked heat detector to be provided in the kitchen.
In hallways exceeding 7.5m in length, no point within the hallway should exceed 7.5m from the nearest detector and no bedroom door should be further than 3m from the nearest smoke alarm. If ceiling mounted detectors to be 300mm from the walls and light fittings.

SMOKE DETECTION
Provide a linked smoke alarm detection system to BS EN 14604 and BS 5839-6:2019 to at least a Grade D2 category LD3 standard (Grade A for large house, Grade A LD2 for large house with 3 stories or more). System to be mains powered with battery back up. At least one smoke detector to be provided in each hallway and landing. In hallways exceeding 7.5m in length, no point within the hallway should exceed 7.5m from the nearest detector and no bedroom door should be further than 3m from the nearest smoke alarm. If ceiling mounted they should be 300mm from the walls and light fittings.
Mains-wired, interlinked heat detector to be provided to the kitchen and smoke detectors to principal living rooms, if required by Building Control.



3D View

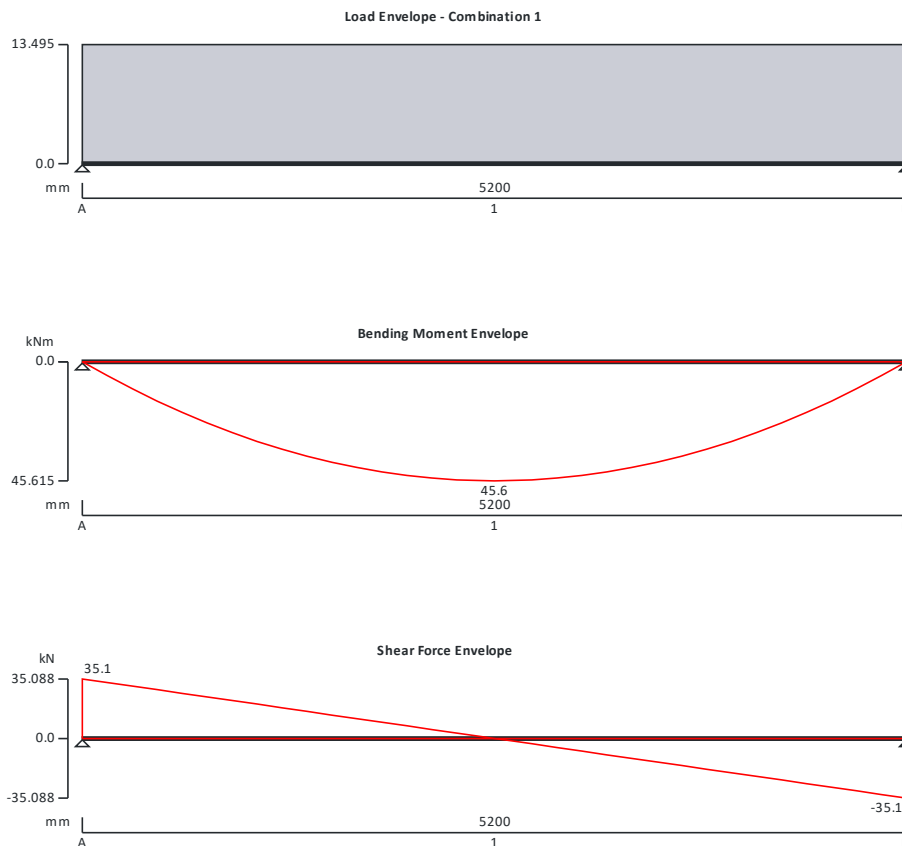
Revision Description	
Location	
Mr&Mrs 3 Jubilee Gardens High Peak SK22 4PL	
Title	
Proposed Plans and Elevations	
Scale	Drawing No.
As indicated @A1	25001 P04
Revision	

Project 3 Jubilee Gardens				Job no. 1305	
Calcs for Beam B1				Start page no./Revision 1	
Calcs by CW	Calcs date 02/11/2025	Checked by	Checked date	Approved by	Approved date

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 3.0.16



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 6.75 kN/m

Variable full UDL 2.7 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$

Project 3 Jubilee Gardens				Job no. 1305	
Calcs for Beam B1				Start page no./Revision 2	
Calcs by CW	Calcs date 02/11/2025	Checked by	Checked date	Approved by	Approved date

Analysis results

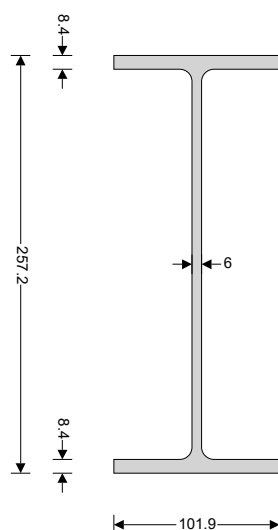
Maximum moment	$M_{\max} = 45.6 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 35.1 \text{ kN}$	$V_{\min} = -35.1 \text{ kN}$
Deflection	$\delta_{\max} = 12.9 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 35.1 \text{ kN}$	$R_{A_{\min}} = 35.1 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 18.2 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 35.1 \text{ kN}$	$R_{B_{\min}} = 35.1 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 18.2 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 7 \text{ kN}$	

Section details

Section type	UB 254x102x25 (British Steel Section Range 2022 (BS4-1))
Steel grade	S355

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

Nominal thickness of element	$t = \max(t_f, t_w) = 8.4 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

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

Lateral restraint

Span 1 has 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	$K_{LTA} = 1.000$
	$K_{LTB} = 1.000$

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Calcs for Beam B1				Start page no./Revision 3	
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Classification of cross sections - Section 5.5

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

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

Width of section $c = d = \mathbf{225.2 \text{ mm}}$

$$c / t_w = 46.1 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{40.3 \text{ mm}}$

$$c / t_f = 5.9 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{240.4 \text{ mm}}$

Shear area factor $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{35.1 \text{ kN}}$

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1670 \text{ mm}^2}$

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{342.3 \text{ kN}}$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{45.6 \text{ kNm}}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{108.5 \text{ kNm}}$

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 $\delta_{lim} = L_{s1} / 360 = \mathbf{14.4 \text{ mm}}$

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{12.874 \text{ mm}}$

PASS - Maximum deflection does not exceed deflection limit

Project 3 Jubilee Gardens				Job no. 1300	
Calcs for Timber Rafters to Dormer Roof				Start page no./Revision 1	
Calcs by CW	Calcs date 02/11/2025	Checked by	Checked date	Approved by	Approved date

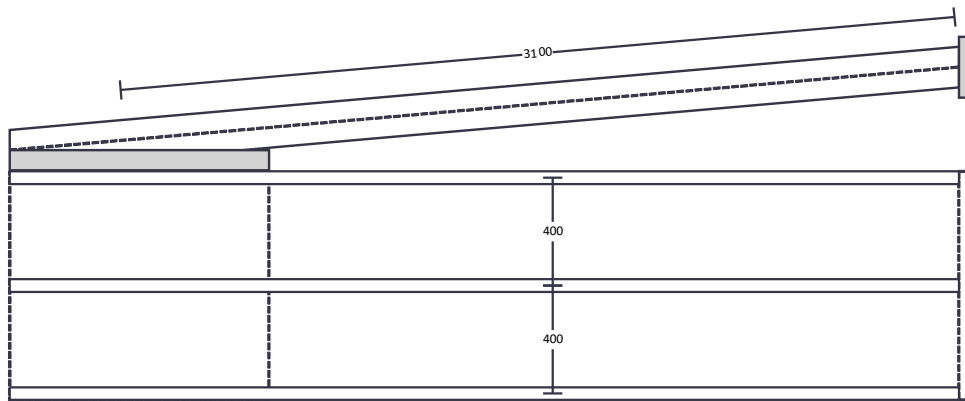
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.09

Rafter details

Description 47 x 150 C24 timber rafters
Rafter spacing $S_{\text{Rafter}} = 400 \text{ mm}$
Rafter inclination $\theta_{\text{Rafter}} = 5 \text{ deg}$



Forces input on Rafter

Permanent load on slope $F_{G_Rafter} = 1.25 \text{ kN/m}^2$
Imposed load on plan $F_{Q_Rafter} = 0.60 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope $p_G = F_{G_Rafter} \times S_{\text{Rafter}} = 0.50 \text{ kN/m}$
Imposed load on slope $p_Q = F_{Q_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.24 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	12.438	0.041	0.003	PASS
Bearing stress	N/mm ²	2.221	0.273	0.123	PASS
Bending stress	N/mm ²	14.215	7.24	0.509	PASS
Shear stress	N/mm ²	2.369	0.523	0.221	PASS
Bending and axial force				0.509	PASS
Column stability check				0.515	PASS
Deflection	mm	12.4	10.422	0.840	PASS

ANALYSIS

Tedds calculation version 1.0.38

Loading

Self weight included (Permanent x 1)

Project 3 Jubilee Gardens				Job no. 1300	
Calcs for Timber Rafters to Dormer Roof				Start page no./Revision 2	
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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

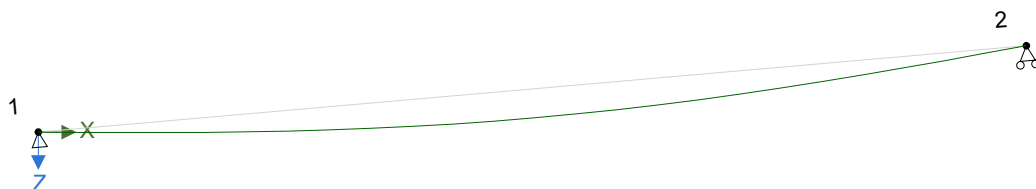
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.5 kN/m at 0 m to 3.1 m
Member	Imposed	UDL	GlobalZ	0.24 kN/m at 0 m to 3.1 m

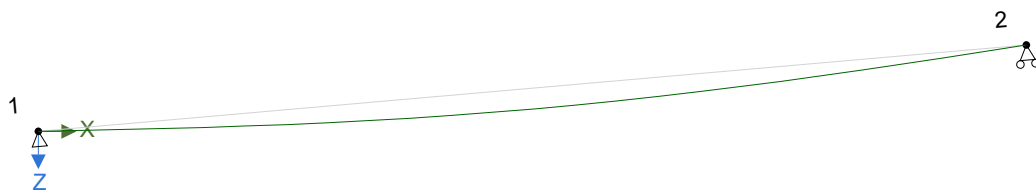
Results

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		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.51956	Member
2	0	0	-0.51956	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.37192	Member
2	0	0	-0.37192	Member

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Calcs for Timber Rafters to Dormer Roof				Start page no./Revision 3	
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Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	3.3
1.00G + 1.00Q (Service)	0	2.4

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

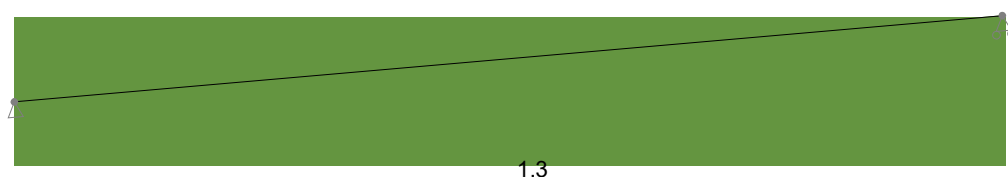
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.1	1	-0.3	-1.6	0
		2	0	-1.6	0

Load combination: 1.00G + 1.00Q (Service)

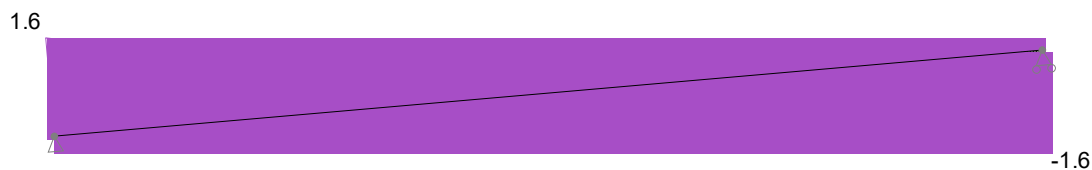
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.1	1	-0.2	-1.2	0
		2	0	-1.2	0

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	1.6		0 (min)	
	1.55	0		1.3 (max)	
	3.1	-1.6 (max abs)		0 (min)	

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Calcs for Timber Rafters to Dormer Roof				Start page no./Revision 4	
Calcs by CW	Calcs date 02/11/2025	Checked by	Checked date	Approved by	Approved date

Member - Span 1

Partial factor for material properties and resistances

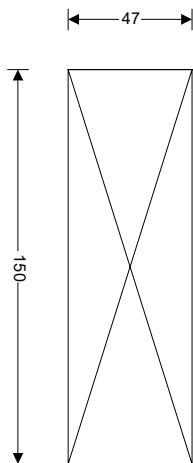
Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Long-term
Service class - cl.2.3.1.3 1

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 **C24**



47x150 timber section

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

Span details

Bearing length $L_b = 100$ mm

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

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$
Deformation factor - Table 3.2 $k_{def} = 0.6$
Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
Load configuration factor - cl.6.1.5(4) $k_{c,90} = 1.5$
System strength factor - cl.6.6 $k_{sys} = 1.1$

Check compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 0.288$ kN
Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.041$ N/mm²
Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 12.438$ N/mm²
 $\sigma_{c,0,d} / f_{c,0,d} = 0.003$

PASS - Design parallel compression strength exceeds design parallel compression stress

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Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 1.665 \text{ kN}$
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130 \text{ mm}$
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.273 \text{ N/mm}^2$
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.481 \text{ N/mm}^2$
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.123$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.646 \text{ kN}$
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.523 \text{ N/mm}^2$
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.369 \text{ N/mm}^2$
 $\tau_{y,d} / f_{v,y,d} = 0.221$

PASS - Design shear strength exceeds design shear stress

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

Effective length for y-axis bending $L_{e,y} = 0.9 \times 3100 \text{ mm} = 2790 \text{ mm}$
 Slenderness ratio $\lambda_{y} = L_{e,y} / i_y = 64.432$
 Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.093$
 Effective length for z-axis bending $L_{e,z} = 0 \text{ mm}$
 Slenderness ratio $\lambda_z = L_{e,z} / i_z = 0$
 Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor $\beta_c = 0.2$
 Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.176$
 $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
 $k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.621$
 $k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
 Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.005$
 $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.003$

PASS - Column stability is acceptable

Check design 1550 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 1.276 \text{ kNm}$
 Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 7.24 \text{ N/mm}^2$
 Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.215 \text{ N/mm}^2$
 $\sigma_{m,y,d} / f_{m,y,d} = 0.509$

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = 0.509$
 $(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.357$

PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis bending $L_{e,y} = 0.9 \times 3100 \text{ mm} = 2790 \text{ mm}$

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Slenderness ratio	$\lambda_y = L_{e,y} / i_y = \mathbf{64.432}$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{1.093}$
Effective length for z-axis bending	$L_{e,z} = \mathbf{0} \text{ mm}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = \mathbf{0}$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{0}$
	$\lambda_{rel,y} > 0.3$ column stability check is required
Straightness factor	$\beta_c = \mathbf{0.2}$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{1.176}$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{0.470}$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.621}$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{1.064}$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.515}$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.360}$
	PASS - Column stability is acceptable

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

Check design 1550 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	$\delta_y = \mathbf{6.5} \text{ mm}$
Quasi-permanent variable load factor	$\psi_2 = \mathbf{0}$
Final deflection with creep	$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = \mathbf{10.4} \text{ mm}$
Allowable deflection	$\delta_{y,Allowable} = L_{m1_s1} / 250 = \mathbf{12.4} \text{ mm}$
	$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.84}$
	PASS - Allowable deflection exceeds final deflection

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

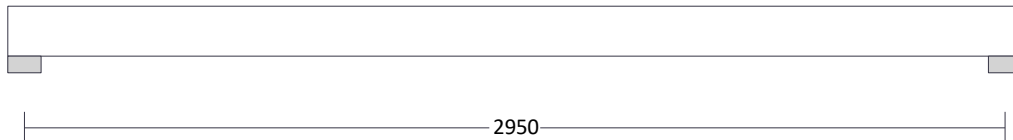
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Joist details

Description 50 x 150 C24 timber joists

Joist spacing $S_{Joist} = 400 \text{ mm}$



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.65 \text{ kN/m}^2$

Vertical imposed load on joist $F_{Q_Joist} = 2.00 \text{ kN/m}^2$

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.26 \text{ kN/m}$

Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 0.80 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	2.221	0.361	0.163	PASS
Bending stress	N/mm ²	14.215	9.24	0.650	PASS
Shear stress	N/mm ²	2.369	0.701	0.296	PASS
Deflection	mm	11.8	11.567	0.980	PASS

ANALYSIS

Tedds calculation version 1.0.38

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	0.26 kN/m at 0 m to 2.95 m
Member	Imposed	UDL	GlobalZ	0.8 kN/m at 0 m to 2.95 m

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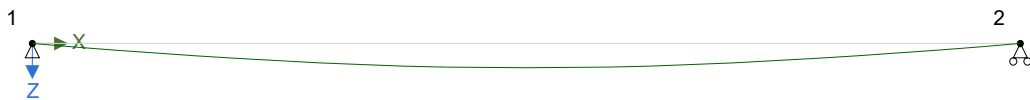
Results

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		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.63104	
2	0	0	-0.63104	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.43222	
2	0	0	-0.43222	

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	2.95	1	0	-2.3	0
		2	0	-2.3	0

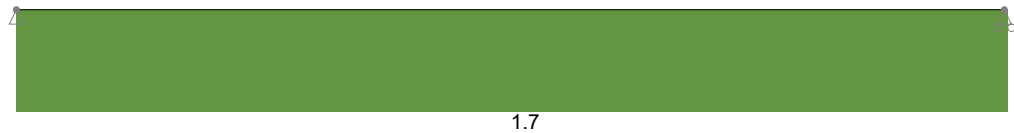
Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.95	1	0	-1.6	0
		2	0	-1.6	0

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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	2.3 (max abs)		0 (min)	
	1.475	0		1.7 (max)	
	2.95	-2.3		0 (min)	

Tedds calculation version 2.2.25

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

Timber section details

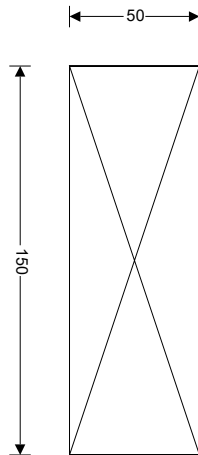
Number of timber sections in member N = 1

Breadth of sections b = 50 mm

Depth of sections h = 150 mm

Timber strength class - EN 338:2016 Table 1 C24

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50x150 timber section

Cross-sectional area, A , 7500 mm²
 Section modulus, W_y , 187500 mm³
 Section modulus, W_z , 62500 mm³
 Second moment of area, I_y , 14062500 mm⁴
 Second moment of area, I_z , 1562500 mm⁴
 Radius of gyration, i_y , 43.3 mm
 Radius of gyration, i_z , 14.4 mm
Timber strength class C24
 Characteristic bending strength, $f_{m,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0,05}$, 7400 N/mm²
 Shear modulus of elasticity, G_{mean} , 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

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

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$
 Deformation factor - Table 3.2 $k_{def} = 0.6$
 Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
 Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
 Load configuration factor - cl.6.1.5(4) $k_{c,90} = 1.5$
 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 $F_{c,y,90,d} = 2.349$ kN
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.361$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.481$ N/mm²
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.163$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 2.349$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.701$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.369$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.296$

PASS - Design shear strength exceeds design shear stress

Check design 1475 mm along span
Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 1.733$ kNm
 Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 9.24$ N/mm²
 Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.215$ N/mm²

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$$\sigma_{m,y,d} / f_{m,y,d} = 0.65$$

PASS - Design bending strength exceeds design bending stress

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

Check design 1475 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 7.2 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 11.6 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1,s1} / 250 = 11.8 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.98$$

PASS - Allowable deflection exceeds final deflection

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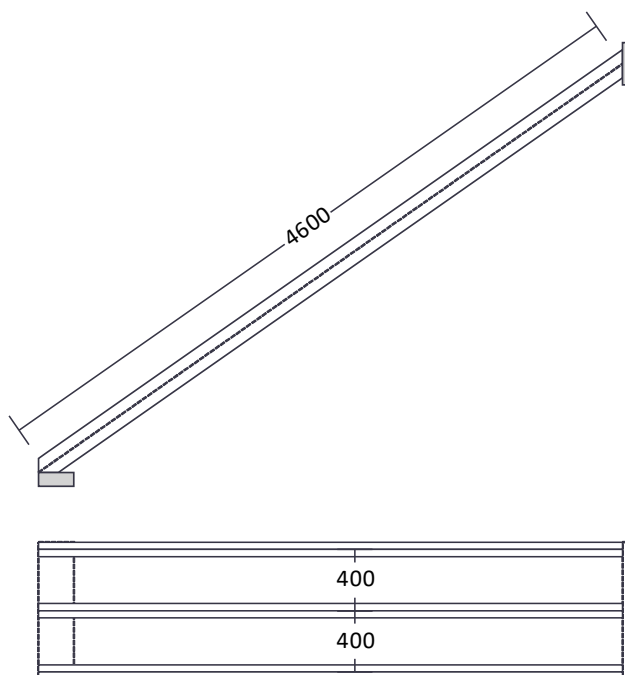
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.09

Rafter details

Description	2/47 x 150 C24 timber rafters
Rafter spacing	$S_{\text{Rafter}} = 400 \text{ mm}$
Rafter inclination	$\theta_{\text{Rafter}} = 35 \text{ deg}$



Forces input on Rafter

Permanent load on slope	$F_{G_Rafter} = 1.25 \text{ kN/m}^2$
Imposed load on plan	$F_{Q_Rafter} = 0.60 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope	$p_G = F_{G_Rafter} \times S_{\text{Rafter}} = 0.50 \text{ kN/m}$
Imposed load on slope	$p_Q = F_{Q_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.20 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	12.438	0.194	0.016	PASS
Bearing stress	N/mm ²	2.221	0.265	0.119	PASS
Bending stress	N/mm ²	14.215	6.363	0.448	PASS
Shear stress	N/mm ²	2.369	0.31	0.131	PASS
Bending and axial force				0.448	PASS
Column stability check				0.495	PASS
Deflection	mm	18.4	12.433	0.676	PASS

ANALYSIS

Tedds calculation version 1.0.38

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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

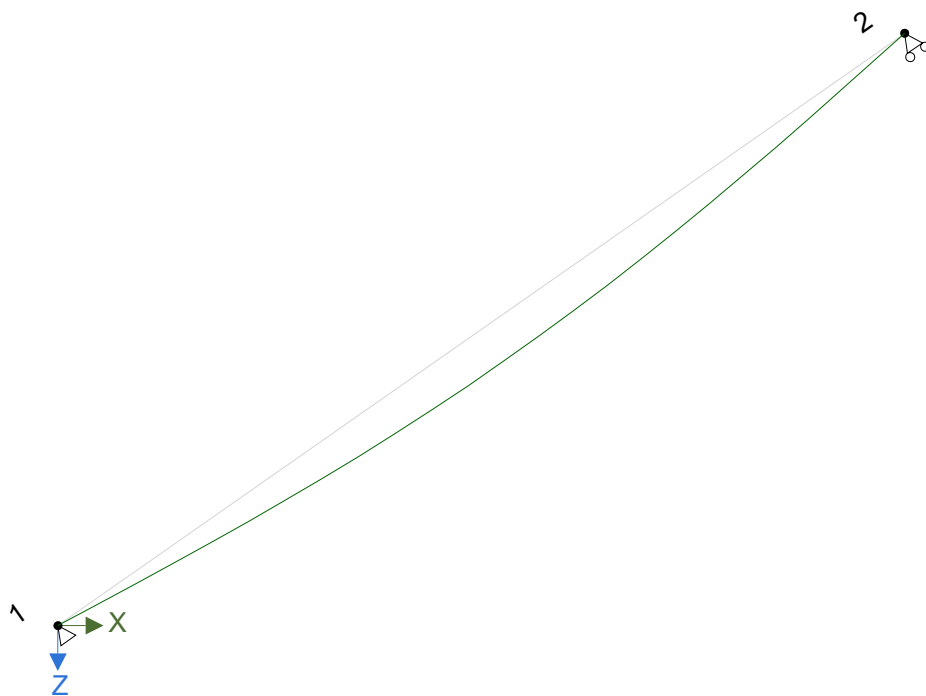
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.5 kN/m at 0 m to 4.6 m
Member	Imposed	UDL	GlobalZ	0.2 kN/m at 0 m to 4.6 m

Results

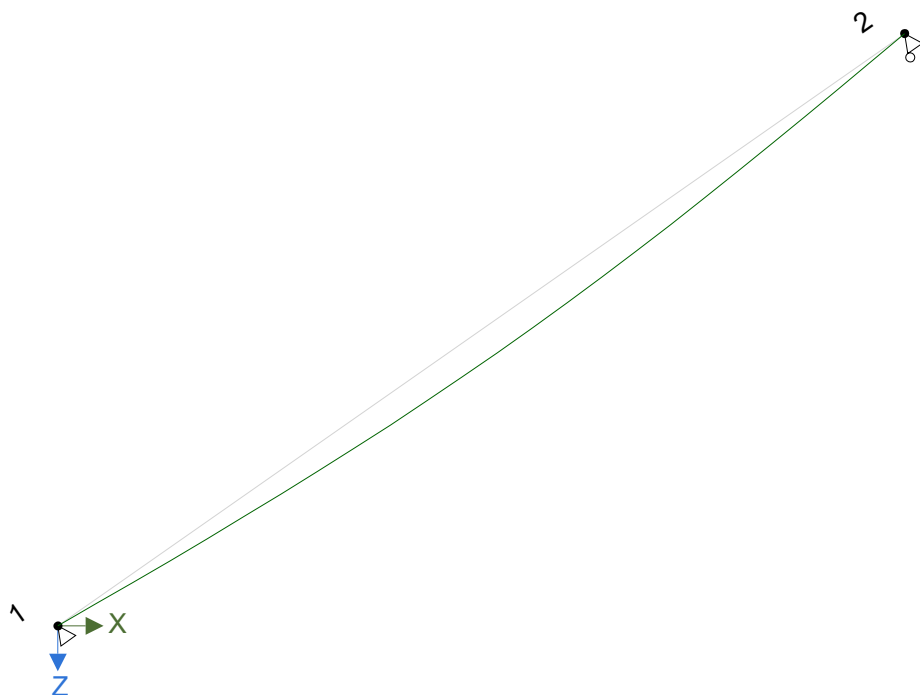
Total deflection

1.35G + 1.50Q (Strength) - Total deflection



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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	0.6776	Member
2	0	0	-0.6776	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.48763	Member
2	0	0	-0.48763	Member

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.8
1.00G + 1.00Q (Service)	0	3.4

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Element end forces

Load combination: 1.35G + 1.50Q (Strength)

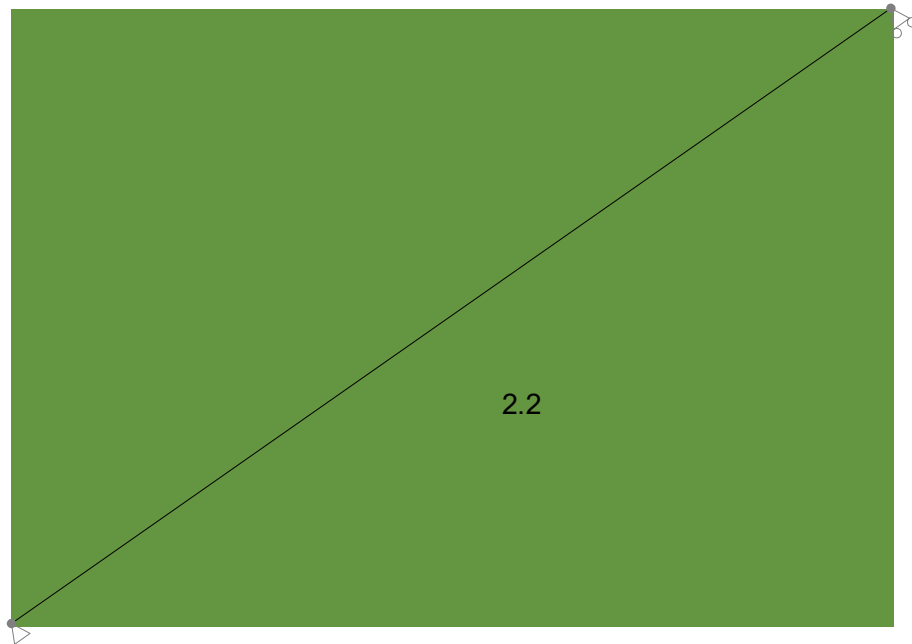
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.6	1	-2.7	-2	0
		2	0	-2	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.6	1	-2	-1.4	0
		2	0	-1.4	0

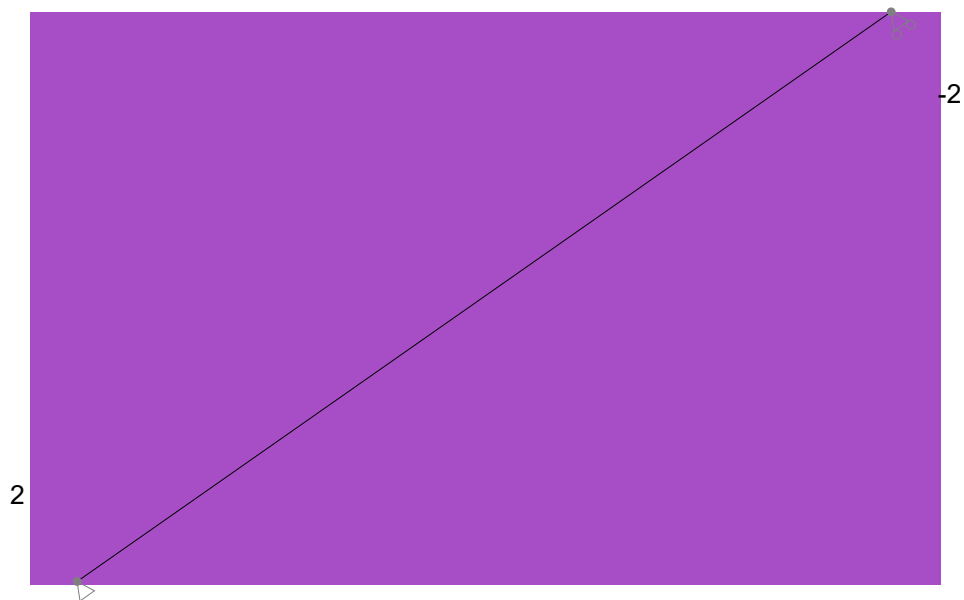
Forces

Strength combinations - Moment envelope (kNm)



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Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	2 (max abs)		0 (min)	
	2.3	0		2.2 (max)	
	4.6	-2		0 (min)	

Tedds calculation version 2.2.25

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

Timber section details

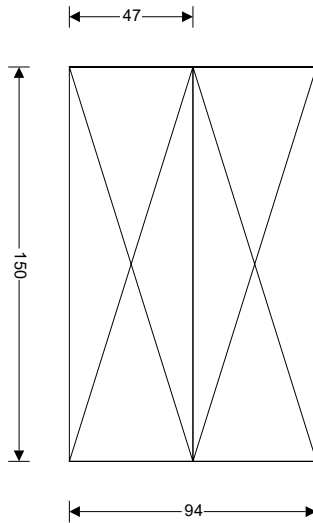
Number of timber sections in member $N = 2$

Breadth of sections $b = 47$ mm

Depth of sections $h = 150$ mm

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

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2/47x150 timber sections

Cross-sectional area, A , 14100 mm²
 Section modulus, W_y , 352500 mm³
 Section modulus, W_z , 110450 mm³
 Second moment of area, I_y , 26437500 mm⁴
 Second moment of area, I_z , 2595575 mm⁴
 Radius of gyration, i_y , 43.3 mm
 Radius of gyration, i_z , 13.6 mm
Timber strength class C24
 Characteristic bending strength, $f_{m,k}$, 24 N/mm²
 Characteristic shear strength, $f_{v,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0,05}$, 7400 N/mm²
 Shear modulus of elasticity, G_{mean} , 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

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

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$
 Deformation factor - Table 3.2 $k_{def} = 0.6$
 Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
 Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
 Load configuration factor - cl.6.1.5(4) $k_{c,90} = 1.5$
 System strength factor - cl.6.6 $k_{sys} = 1.1$

Check compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 2.731$ kN
 Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.194$ N/mm²
 Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 12.438$ N/mm²
 $\sigma_{c,0,d} / f_{c,0,d} = 0.016$

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 3.234$ kN
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (N \times b \times L_{b,ef}) = 0.265$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.481$ N/mm²
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.119$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.95$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times N \times b \times h) = 0.310$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.369$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.131$

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PASS - Design shear strength exceeds design shear stress

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

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 4600 \text{ mm} = 4140 \text{ mm}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = 95.609$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.621$
Effective length for z-axis bending	$L_{e,z} = 0 \text{ mm}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$
	$\lambda_{rel,y} > 0.3$ column stability check is required
Straightness factor	$\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.946$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.331$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.047$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.015$

PASS - Column stability is acceptable

Check design 2300 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d} = 2.243 \text{ kNm}$
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y = 6.363 \text{ N/mm}^2$
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.215 \text{ N/mm}^2$
	$\sigma_{m,y,d} / f_{m,y,d} = 0.448$

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20	$(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = 0.448$
	$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.314$

PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 4600 \text{ mm} = 4140 \text{ mm}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = 95.609$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.621$
Effective length for z-axis bending	$L_{e,z} = 0 \text{ mm}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$
	$\lambda_{rel,y} > 0.3$ column stability check is required
Straightness factor	$\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.946$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.331$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = 0.495$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.328$

PASS - Column stability is acceptable

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

Check design 2300 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 12.4 \text{ mm}$$

Allowable deflection

$$\delta_{y, \text{Allowable}} = L_{m1_s1} / 250 = 18.4 \text{ mm}$$

$$\delta_y / \delta_{y, \text{Allowable}} = 0.676$$

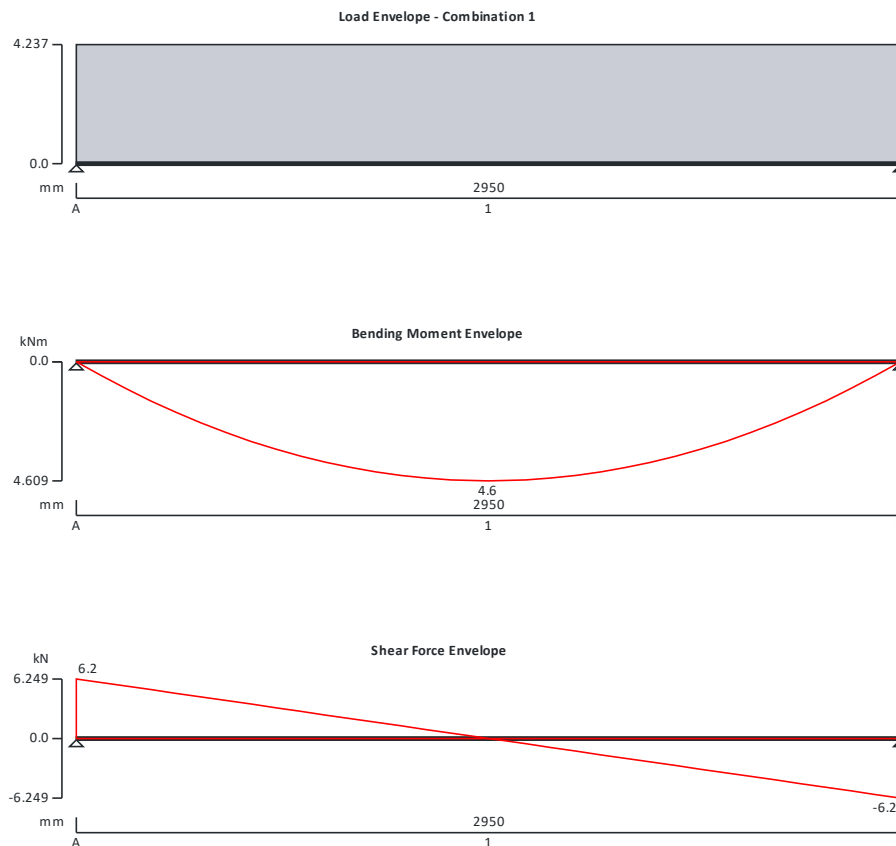
PASS - Allowable deflection exceeds instantaneous deflection

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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 $\times 1$
 Permanent full UDL 1.950 kN/m
 Variable full UDL 1.000 kN/m

Load combinations

Load combination 1

Support A	Permanent $\times 1.35$ Variable $\times 1.50$
Span 1	Permanent $\times 1.35$ Variable $\times 1.50$
Support B	Permanent $\times 1.35$ Variable $\times 1.50$

Analysis results

Maximum moment

$M_{\max} = 4.609$ kNm

$M_{\min} = 0.000$ kNm

Design moment

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 4.609$ kNm

Maximum shear

$F_{\max} = 6.249$ kN

$F_{\min} = -6.249$ kN

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Design shear

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = \mathbf{6.249 \text{ kN}}$$

Total load on beam

$$W_{\text{tot}} = \mathbf{12.498 \text{ kN}}$$

Reactions at support A

$$R_{A_{\max}} = \mathbf{6.249 \text{ kN}}$$

$$R_{A_{\min}} = \mathbf{6.249 \text{ kN}}$$

Unfactored permanent load reaction at support A

$$R_{A_{\text{Permanent}}} = \mathbf{2.990 \text{ kN}}$$

Unfactored variable load reaction at support A

$$R_{A_{\text{Variable}}} = \mathbf{1.475 \text{ kN}}$$

Reactions at support B

$$R_{B_{\max}} = \mathbf{6.249 \text{ kN}}$$

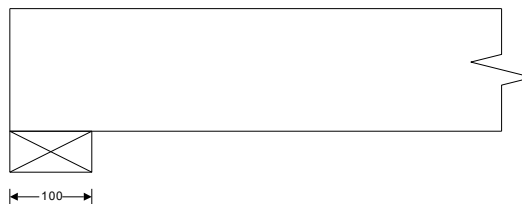
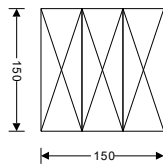
$$R_{B_{\min}} = \mathbf{6.249 \text{ kN}}$$

Unfactored permanent load reaction at support B

$$R_{B_{\text{Permanent}}} = \mathbf{2.990 \text{ kN}}$$

Unfactored variable load reaction at support B

$$R_{B_{\text{Variable}}} = \mathbf{1.475 \text{ kN}}$$



Timber section details

Breadth of timber sections

$$b = \mathbf{50 \text{ mm}}$$

Depth of timber sections

$$h = \mathbf{150 \text{ mm}}$$

Number of timber sections in member

$$N = \mathbf{3}$$

Overall breadth of timber member

$$b_b = N \times b = \mathbf{150 \text{ mm}}$$

Timber strength class - EN 338:2016 Table 1

$$\mathbf{C24}$$

Member details

Load duration - cl.2.3.1.2

$$\mathbf{\text{Long-term}}$$

Service class of timber - cl.2.3.1.3

$$\mathbf{1}$$

Length of span

$$L_{s1} = \mathbf{2950 \text{ mm}}$$

Length of bearing

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

Section properties

Cross sectional area of member

$$A = N \times b \times h = \mathbf{22500 \text{ mm}^2}$$

Section modulus

$$W_y = N \times b \times h^2 / 6 = \mathbf{562500 \text{ mm}^3}$$

$$W_z = h \times (N \times b)^2 / 6 = \mathbf{562500 \text{ mm}^3}$$

Second moment of area

$$I_y = N \times b \times h^3 / 12 = \mathbf{42187500 \text{ mm}^4}$$

$$I_z = h \times (N \times b)^3 / 12 = \mathbf{42187500 \text{ mm}^4}$$

Radius of gyration

$$r_y = \sqrt{I_y / A} = \mathbf{43.3 \text{ mm}}$$

$$r_z = \sqrt{I_z / A} = \mathbf{43.3 \text{ mm}}$$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3

$$\gamma_M = \mathbf{1.300}$$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$$k_{\text{mod}} = \mathbf{0.700}$$

Deformation factor for service classes - Table 3.2

$$k_{\text{def}} = \mathbf{0.600}$$

Depth factor for bending - exp.3.1

$$k_{h,m} = \mathbf{1.000}$$

Depth factor for tension - exp.3.1

$$k_{h,t} = \mathbf{1.000}$$

Bending stress re-distribution factor - cl.6.1.6(2)

$$k_m = \mathbf{0.700}$$

Crack factor for shear resistance - cl.6.1.7(2)

$$k_{cr} = \mathbf{0.670}$$

Load configuration factor - exp.6.4

$$k_{c,90} = \mathbf{1.000}$$

System strength factor - cl.6.6

$$k_{\text{sys}} = \mathbf{1.000}$$

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Lateral buckling factor - cl.6.3.3(5)

$$k_{crit} = 1.000$$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress

$$\sigma_{c,90,d} = R_{A,max} / (N \times b \times L_b) = 0.417 \text{ N/mm}^2$$

Design compressive strength

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.346 \text{ N/mm}^2$$

$$\sigma_{c,90,d} / f_{c,90,d} = 0.309$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress

$$\sigma_{m,d} = M / W_y = 8.193 \text{ N/mm}^2$$

Design bending strength

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 12.923 \text{ N/mm}^2$$

$$\sigma_{m,d} / f_{m,d} = 0.634$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.622 \text{ N/mm}^2$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.154 \text{ N/mm}^2$$

$$\tau_d / f_{v,d} = 0.289$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = 11.800 \text{ mm}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = 4.478 \text{ mm}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 7.165 \text{ mm}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = 2.209 \text{ mm}$$

Factor for quasi-permanent variable action

$$\psi_2 = 0.3$$

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 2.607 \text{ mm}$$

Total final deflection

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 9.772 \text{ mm}$$

$$\delta_{fin} / \delta_{lim} = 0.828$$

PASS - Total final deflection is less than the deflection limit

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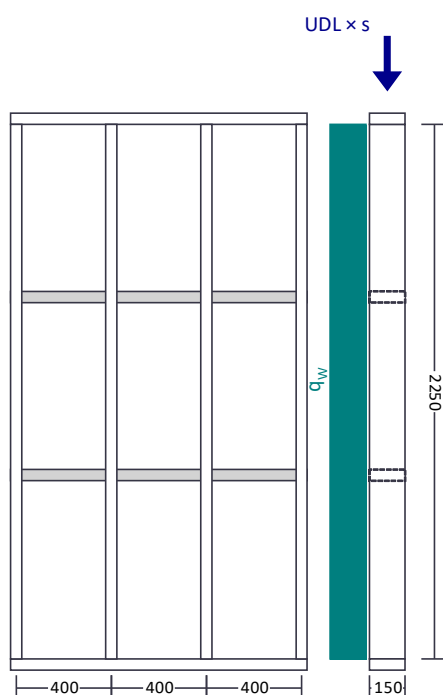
TIMBER STUD ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.09

Stud details

Description	47 x 150 C24 timber studs
Restraint in plane of panel	Dwangs (x 2)
Stud spacing	$s_{\text{Stud}} = 400 \text{ mm}$
Stud height	$l_{\text{Stud}} = 2250 \text{ mm}$
Panel height	$l_{\text{Panel}} = l_{\text{Stud}} + 2 \times b = 2344 \text{ mm}$



Forces input on Stud

Lateral wind load	$H_{W_Z0_Stud} = 1.50 \text{ kN/m}^2$
Permanent distributed load on top rail	$L_{G_Stud} = 2.50 \text{ kN/m}$
Imposed distributed load on top rail	$L_{Q_Stud} = 1.20 \text{ kN/m}$

Stud loading details

Distributed loads

Lateral wind load	$p_{W,h0} = H_{W_Z0_Stud} \times s_{\text{Stud}} = 0.60 \text{ kN/m}$
-------------------	---

Point loads

Total vertical permanent point load (2250 mm)	$P_{G_1} = L_{G_Stud} \times s_{\text{Stud}} = 1.00 \text{ kN}$
Total vertical imposed point load (2250 mm)	$P_{Q_1} = L_{Q_Stud} \times s_{\text{Stud}} = 0.48 \text{ kN}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	15.992	0.304	0.019	PASS
Bearing stress	N/mm ²	1.904	0.166	0.087	PASS
Bending stress	N/mm ²	18.277	3.231	0.177	PASS

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Shear stress	N/mm ²	3.046	0.322	0.106	PASS
Bending and axial force				0.177	PASS
Column stability check				0.197	PASS
Beam stability check				0.055	PASS
Deflection	mm	9	2.353	0.261	PASS

ANALYSIS

Tedds calculation version 1.0.38

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
1.35G + 1.50Q + ψ_s 1.50S (Strength)	1.35	1.50	0.75	0.00
1.35G + ψ_0 1.50Q + 1.50S (Strength)	1.35	1.05	1.50	0.00
1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength)	1.35	1.50	0.75	0.75
1.35G + ψ_0 1.50Q + 1.50S + ψ_w 1.50W (Strength)	1.35	1.05	1.50	0.75
1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)	1.35	1.05	0.75	1.50
1.0G + 1.50W (Strength)	1.00	0.00	0.00	1.50
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00
1.00G + 1.00Q + ψ_s 1.00S (Service)	1.00	1.00	0.50	0.00
1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service)	1.00	1.00	0.50	0.50
1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service)	1.00	0.70	1.00	0.50
1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service)	1.00	0.70	0.50	1.00
1.0G + 1.00W (Service)	1.00	0.00	0.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	Point load	GlobalZ	1 kN at 2.25 m
Member	Imposed	Point load	GlobalZ	0.48 kN at 2.25 m
Member	Wind	UDL	GlobalX	0.6 kN/m at 0 m to 2.25 m

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Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.35G + 1.50Q + ψ_s 1.50S (Strength) - Total deflection





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1.35G + ψ_0 1.50Q + 1.50S (Strength) - Total deflection



1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength) - Total deflection



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$1.35G + \psi_0 1.50Q + 1.50S + \psi_w 1.50W$ (Strength) - Total deflection



$1.35G + \psi_0 1.50Q + \psi_s 1.50S + 1.50W$ (Strength) - Total deflection



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1.0G + 1.50W (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection





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1.00G + 1.00Q + ψ_s 1.00S (Service) - Total deflection



1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service) - Total deflection





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1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service) - Total deflection

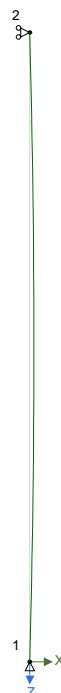


1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service) - Total deflection



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1.0G + 1.00W (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	
2	0	0.1	0	

Load combination: 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0	
2	0	0.1	0	

Load combination: 1.35G + ψ_0 1.50Q + 1.50S (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0	
2	0	0.1	0	

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Load combination: 1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.08416	
2	0	0.1	-0.08416	

Load combination: 1.35G + ψ_0 1.50Q + 1.50S + ψ_w 1.50W (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.08416	
2	0	0.1	-0.08416	

Load combination: 1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.16831	
2	0	0.1	-0.16831	

Load combination: 1.0G + 1.50W (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.16831	
2	0	0	-0.16831	

Load combination: 1.00G + 1.00Q (Service)

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

Load combination: 1.00G + 1.00Q + ψ_s 1.00S (Service)

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

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Load combination: 1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.0561	
2	0	0	-0.0561	

Load combination: 1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.0561	
2	0	0	-0.0561	

Load combination: 1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.11221	
2	0	0	-0.11221	

Load combination: 1.0G + 1.00W (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.11221	
2	0	0	-0.11221	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	2.1
1.35G + 1.50Q + ψ_s 1.50S (Strength)	0	2.1
1.35G + ψ_0 1.50Q + 1.50S (Strength)	0	1.9
1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength)	-1	2.1
1.35G + ψ_0 1.50Q + 1.50S + ψ_w 1.50W (Strength)	-1	1.9
1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)	-2	1.9
1.0G + 1.50W (Strength)	-2	1.1
1.00G + 1.00Q (Service)	0	1.5
1.00G + 1.00Q + ψ_s 1.00S (Service)	0	1.5

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Load case/combination	Force	
	FX (kN)	FZ (kN)
1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service)	-0.7	1.5
1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service)	-0.7	1.4
1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service)	-1.3	1.4
1.0G + 1.00W (Service)	-1.3	1.1

Reactions

Load case: Permanent

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1.1	0
2	0	0	0

Load case: Imposed

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.5	0
2	0	0	0

Load case: Snow

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0	0

Load case: Wind

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.7	0	0
2	-0.7	0	0

Load combination: 1.35G + 1.50Q (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	2.1	0
2	0	0	0

Load combination: 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	2.1	0
2	0	0	0

Load combination: 1.35G + ψ_0 1.50Q + 1.50S (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1.9	0
2	0	0	0

Load combination: 1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.5	2.1	0
2	-0.5	0	0

Load combination: 1.35G + ψ_0 1.50Q + 1.50S + ψ_w 1.50W (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.5	1.9	0
2	-0.5	0	0

Load combination: 1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-1	1.9	0
2	-1	0	0

Load combination: 1.0G + 1.50W (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-1	1.1	0
2	-1	0	0

Load combination: 1.00G + 1.00Q (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1.5	0
2	0	0	0

Load combination: 1.00G + 1.00Q + ψ_s 1.00S (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1.5	0
2	0	0	0

Load combination: 1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.3	1.5	0
2	-0.3	0	0

Load combination: 1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.3	1.4	0
2	-0.3	0	0

Load combination: 1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.7	1.4	0
2	-0.7	0	0

Load combination: 1.0G + 1.00W (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-0.7	1.1	0
2	-0.7	0	0

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	2.25	1	-2.1	0	0
		2	0	0	0

Load combination: 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-2.1	0	0
		2	0	0	0

Load combination: 1.35G + ψ_0 1.50Q + 1.50S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.9	0	0
		2	0	0	0

Load combination: 1.35G + 1.50Q + ψ_s 1.50S + ψ_w 1.50W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-2.1	-0.5	0
		2	0	-0.5	0

Load combination: 1.35G + ψ_0 1.50Q + 1.50S + ψ_w 1.50W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.9	-0.5	0
		2	0	-0.5	0

Load combination: 1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.9	-1	0
		2	0	-1	0

Load combination: 1.0G + 1.50W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.1	-1	0
		2	0	-1	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.5	0	0
		2	0	0	0

Load combination: 1.00G + 1.00Q + ψ_s 1.00S (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.5	0	0
		2	0	0	0

Load combination: 1.00G + 1.00Q + ψ_s 1.00S + ψ_w 1.00W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.5	-0.3	0
		2	0	-0.3	0

Load combination: 1.00G + ψ_0 1.00Q + 1.00S + ψ_w 1.00W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.4	-0.3	0
		2	0	-0.3	0

Load combination: 1.00G + ψ_0 1.00Q + ψ_s 1.00S + 1.00W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.4	-0.7	0
		2	0	-0.7	0

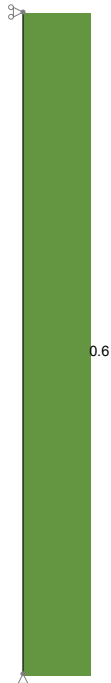
Load combination: 1.0G + 1.00W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.25	1	-1.1	-0.7	0
		2	0	-0.7	0

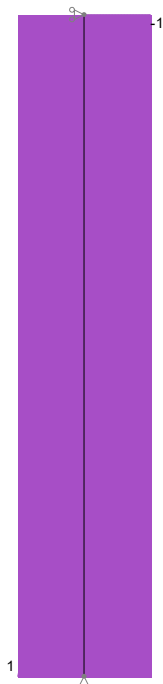
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Forces

Strength combinations - Moment envelope (kNm)

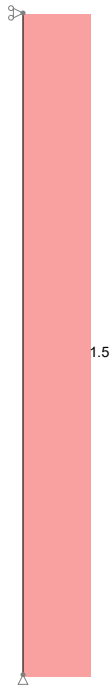


Strength combinations - Shear envelope (kN)



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Service combinations - Deflection envelope (mm)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	1 (max abs)	0	0	
	1.125	0		0.6 (max)	0
	2.25	0	-1	0	

Tedds calculation version 2.2.25

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

Timber section details

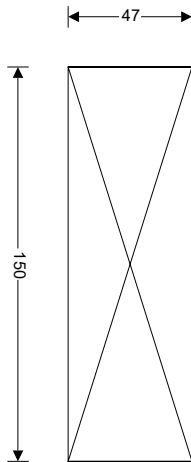
Number of timber sections in member N = 1

Breadth of sections b = 47 mm

Depth of sections h = 150 mm

Timber strength class - EN 338:2016 Table 1 C24

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47x150 timber section

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

Span details

Bearing length

$L_b = 100$ mm

Consider Combination 6 - 1.35G + ψ_0 1.50Q + ψ_s 1.50S + 1.50W (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.9$
 Deformation factor - Table 3.2 $k_{def} = 0.6$
 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 compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 1.928$ kN
 Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.273$ N/mm²
 Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 15.992$ N/mm²
 $\sigma_{c,0,d} / f_{c,0,d} = 0.017$

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 1.013$ kN
 Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.166$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.904$ N/mm²
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.087$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.013$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.322$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 3.046$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.106$

PASS - Design shear strength exceeds design shear stress

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

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 2250 \text{ mm} = \mathbf{2025 \text{ mm}}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = \mathbf{46.765}$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{0.793}$
Effective length for z-axis bending	$L_{e,z} = \mathbf{750 \text{ mm}}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = \mathbf{55.278}$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{0.937}$
	Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is required
Straightness factor	$\beta_c = \mathbf{0.2}$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{0.864}$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{1.003}$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.829}$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{0.735}$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = \mathbf{0.021}$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.023}$
	PASS - Column stability is acceptable

Check design 1125 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d} = \mathbf{0.57 \text{ kNm}}$
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y = \mathbf{3.231 \text{ N/mm}^2}$
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = \mathbf{18.277 \text{ N/mm}^2}$
	$\sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.177}$
	PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20	$(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.177}$
	$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.124}$
	PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis bending	$L_{e,y} = 0.9 \times 2250 \text{ mm} = \mathbf{2025 \text{ mm}}$
Slenderness ratio	$\lambda_y = L_{e,y} / i_y = \mathbf{46.765}$
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{0.793}$
Effective length for z-axis bending	$L_{e,z} = \mathbf{750 \text{ mm}}$
Slenderness ratio	$\lambda_z = L_{e,z} / i_z = \mathbf{55.278}$
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = \mathbf{0.937}$
	Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is required
Straightness factor	$\beta_c = \mathbf{0.2}$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{0.864}$
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{1.003}$
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.829}$
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{0.735}$
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.197}$
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.147}$
	PASS - Column stability is acceptable

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

Effective length - Table 6.1	$L_{ef} = 750 \text{ mm} + 2 \times h = \mathbf{1050 \text{ mm}}$
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef}) = \mathbf{80.955 \text{ N/mm}^2}$
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m,k} / \sigma_{m,crit})} = \mathbf{0.544}$
Lateral buckling factor - exp.6.34	$k_{crit} = \mathbf{1.000}$
Beam stability check - exp.6.35	$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.055}$
PASS - Beam stability is acceptable	

Consider Combination 13 - 1.0G + 1.00W (Service)

Check design 1125 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	$\delta_y = \mathbf{1.5 \text{ mm}}$
Quasi-permanent variable load factor	$\psi_2 = \mathbf{0.3}$
Final deflection with creep	$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = \mathbf{2.4 \text{ mm}}$
Allowable deflection	$\delta_{y,Allowable} = L_{m1_s1} / 250 = \mathbf{9 \text{ mm}}$
	$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.261}$
PASS - Allowable deflection exceeds final deflection	

