ENGINEERED STRUCTURES LIMITED

75 South Street,
Epsom,
KT18 7PY.
Design Calculations & Sketches
Rev 0
December 2021

General Notes:

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1. This is permanent work design only, the contractor is responsible for all temporary work design and the structure stability during the construction work,
2. All timber work New and Existing is assumed to be of grade C24
3. All new Steel members are S335 Grade, unless noted otherwise.
4. The contractor is responsible for timber and steel connections design.
5. All beams and lintels to have 200mm end bearing minimum, unless noted otherwise.
6. Compressive strength of masonry unit assumed to be 5.2 N/mm^2
7. Mortar of class M4 is assumed for all new and old work.
8. Design is carried out to EuroCodes.

8. These calculations and details are to be read in conjunction with all relevant architects and engineers' drawings and specifications.

9. Full building regulation approval should be obtained prior to the commencement of works on site/before any construction materials are ordered .

10. Any works carried out prior to this are undertaken at the clients/contractor's own risk.

11. The works are to be carried out to the approval and satisfaction of the building control officer, to accepted good building practice and with full compliance and in accordance with all relevant British Annexes of Euro Standards and Codes of Practice.

12. All lengths and spans used in these calculations should be verified on site prior to commencement of any construction works. Contractor and/or steel fabricator to take their dimensions on site.

13. Builder/contractor is to check that the structural engineer's proposal (i.e. location of steel beams/trimmers etc) is feasible and necessary before ordering materials.

14. This calculation is property of ESL (Engineered Structures Limited), and intended for the foresaid property only, this pack can not be redistributed without a prior written confirmation from ESL.

Job Description:

Insertion of 2 No. conservation style rooflights in rear roof slope as shown in the Architectural drawing included in this document.

Loading:

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1. Roof Loads:

• Live Loads	0.75 kN/m^2
• Slates, timber battens & felt	0.55 kN/m^2
• Timber rafters & insulation	0.2 kN/m^2
• Ceiling & services	0.15 kN/m^2

❖ Plan Dead Load 0.9/cos45=	**1.30 kN/m^2**
❖ Plan Live Load =	**0.75 kN/m^2**

2. Floor Loads:

• Live loads	1.5 kN/m^2
• Timber Boards	0.15 kN/m^2
• Timber Joists	0.2 kN/m^2
• Ceiling & Services	0.15 kN/m^2

❖ DL =	**0.5 kN/m^2**
❖ LL =	**1.5 kN/m^2**

External Brick Wall

• Brick	2.10 kN/m^2
• 100mm Lightweight Block	1.00 kN/m^2
• Plaster	0.25 kN/m^2

❖ DL=	3.35 **kN/m^2**

Proposed First Floor:

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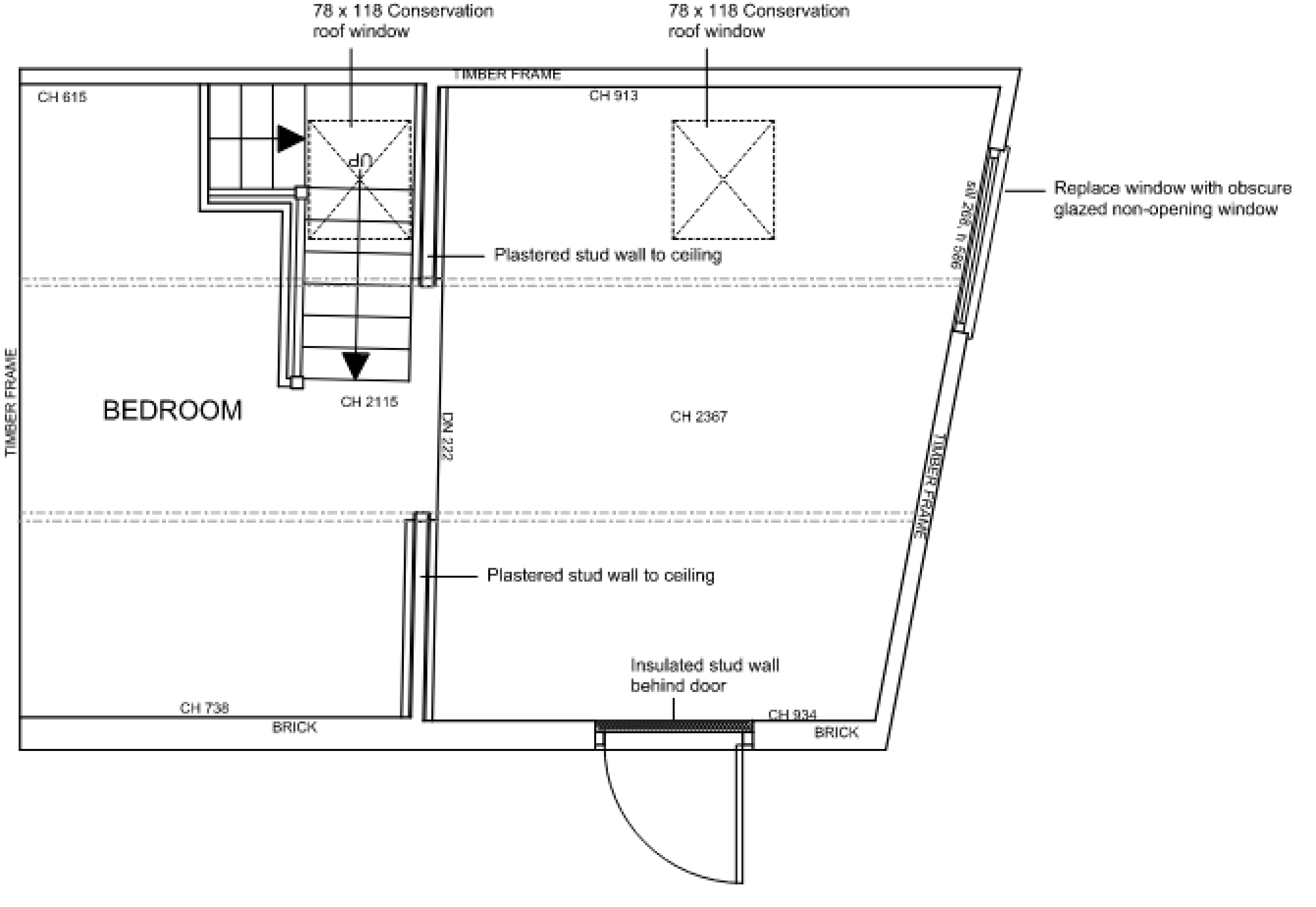

Existing Roof Structural Arrangement:

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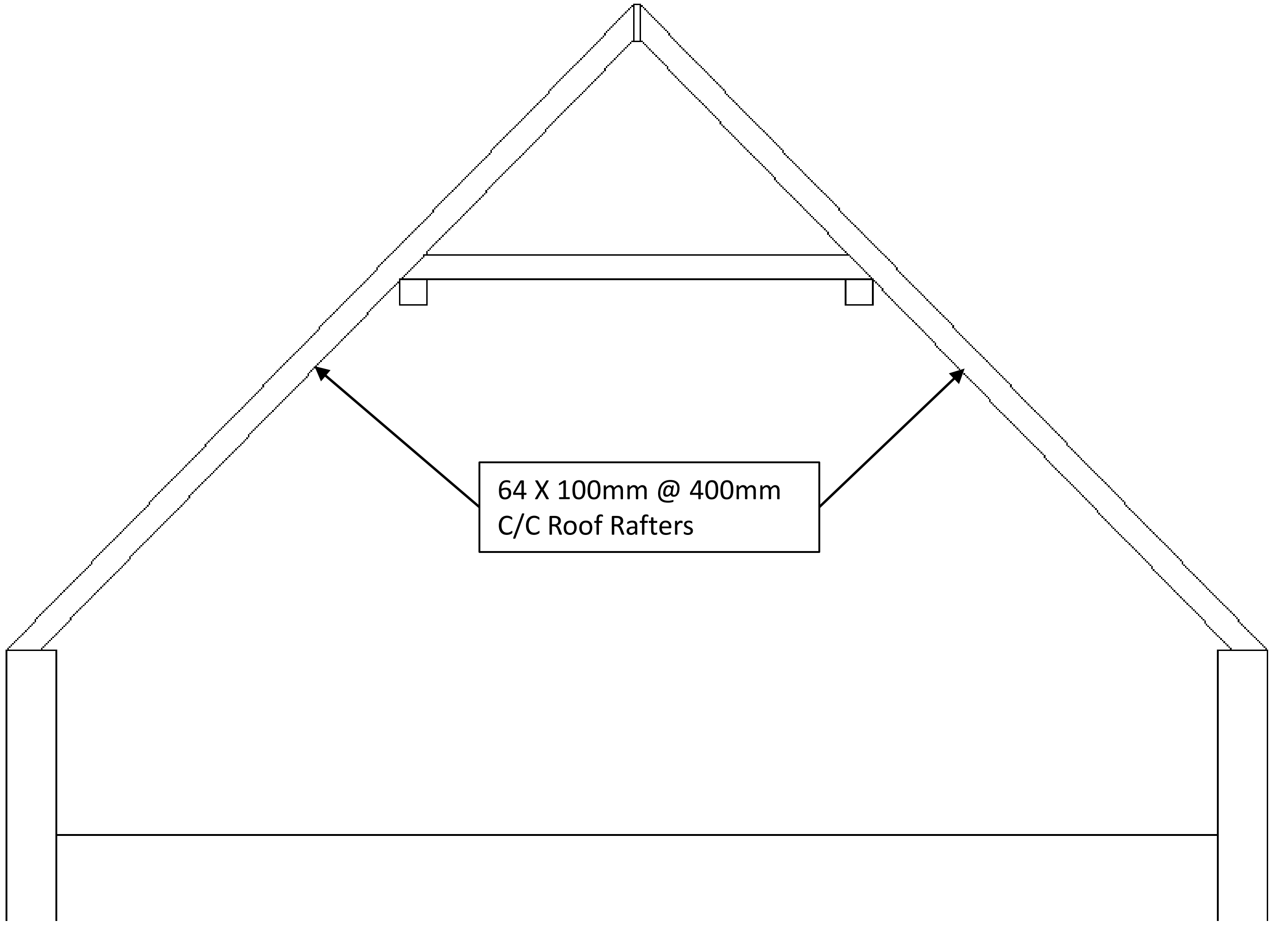


Discussion:

A structural assessment has been carried out on the existing roof rafters to assess the impact of introducing a new roof light in the lower part of the roof.

The existing spacing between the rafters is 400mm, and the new Velux roof window to be introduced is 780mm, meaning a **single** existing roof rafter will need to be cut, to introduce the window.

As such, the existing roof rafters has been analysed for an increase equivalent spacing of 600mm (copy of rafter analysis and design attached to this document appendix) proving that the rafter is still safe and within its design capacity despite the additional loads imposed to it.

Conclusion:

Introducing the proposed Velux window, and trimming a single roof rafter within a distance of 1.6m, would not require structural intervention.

Design Calculations:

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

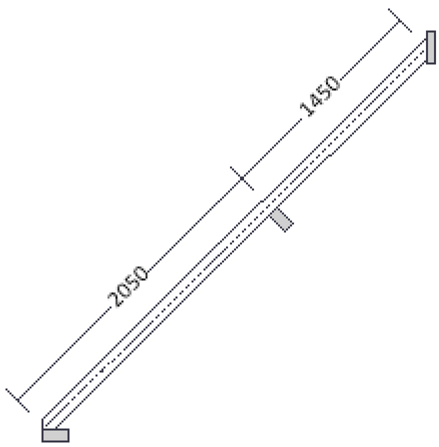
Tedds calculation version 1.0.05

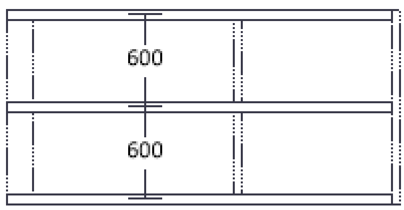
Rafter details

Description — 63 x 100 C24 timber rafters

Rafter spacing — s_{Rafter} = **600** mm

Rafter inclination — θ_{Rafter} = **45** deg





Forces input on Rafter

Permanent load on slope — F_{G_Rafter}= **0.85** kN/m²

Imposed load on plan — F_{Q_Rafter}= **0.75** kN/m²

Snow load on plan — F_{S_Rafter}= **0.60** kN/m²

Rafter loading details

Distributed loads

Permanent load on slope — $p_G = F_{G\_Rafter} \times s_{Rafter}$= **0.51** kN/m

Imposed load on slope — $p_Q = F_{Q\_Rafter} \times s_{Rafter} \times \cos(\theta_{Rafter})$= **0.32** kN/m

Snow load on slope — $p_S = F_{S\_Rafter} \times s_{Rafter} \times \cos(\theta_{Rafter})$= **0.25** kN/m

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
1.35G + 1.50Q + ψ_S1.50S (Strength)	1.35	1.50	0.75	0.00
1.35G + $\psi_0$1.50Q + 1.50S (Strength)	1.35	1.05	1.50	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00
1.00G + 1.00Q + ψ_S1.00S (Service)	1.00	1.00	0.50	0.00
1.00G + $\psi_2$1.00Q (Quasi)	1.00	0.30	0.00	0.00

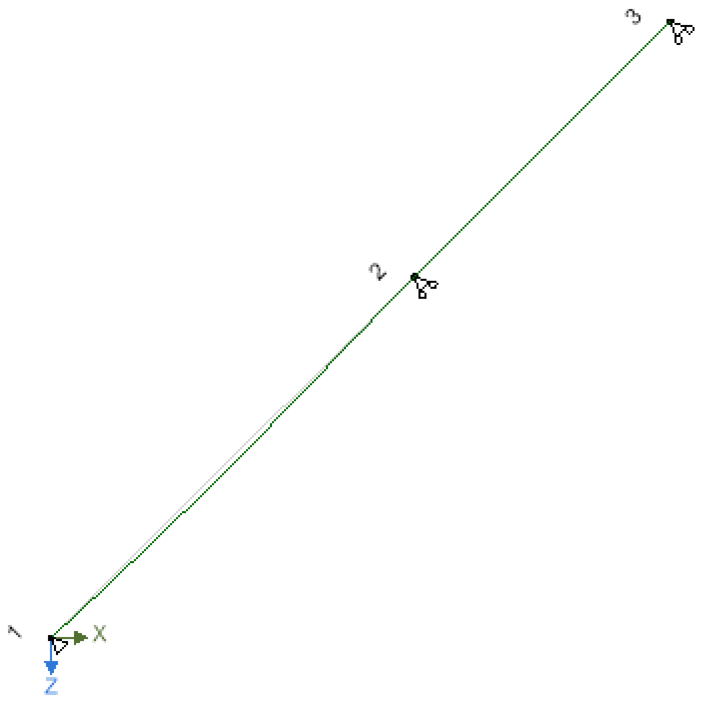
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.51 kN/m at 0 m to 3.5 m
Member	Imposed	UDL	GlobalZ	0.32 kN/m at 0 m to 3.5 m
Member	Snow	UDL	GlobalZ	0.25 kN/m at 0 m to 3.5 m

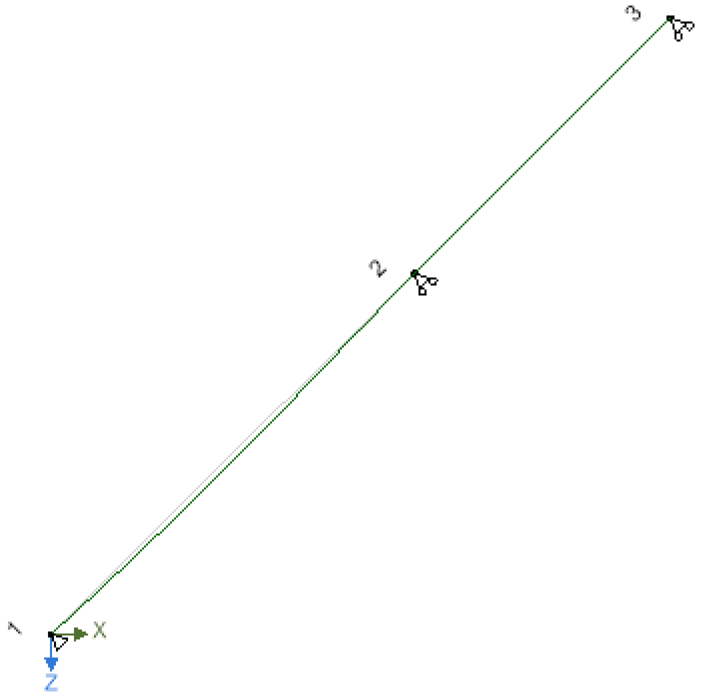
Results

Total deflection

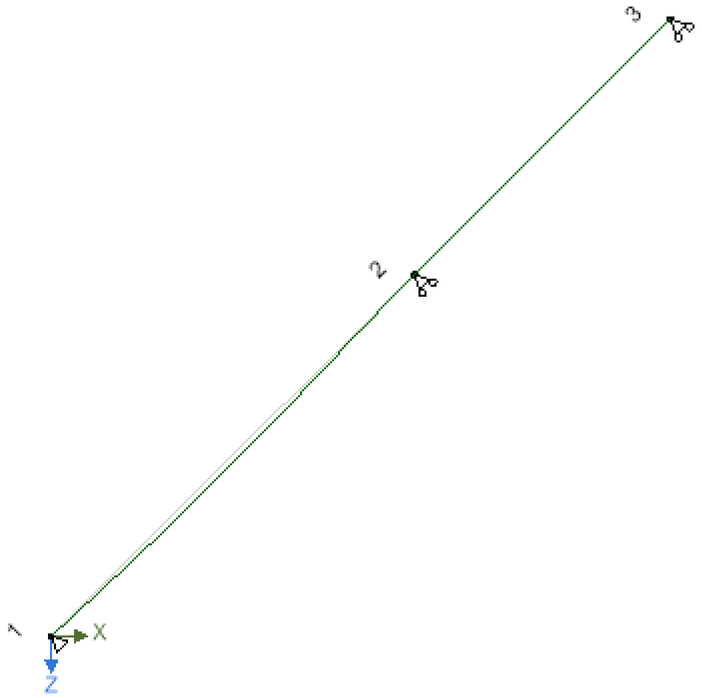
1.35G + 1.50Q (Strength) - Total deflection



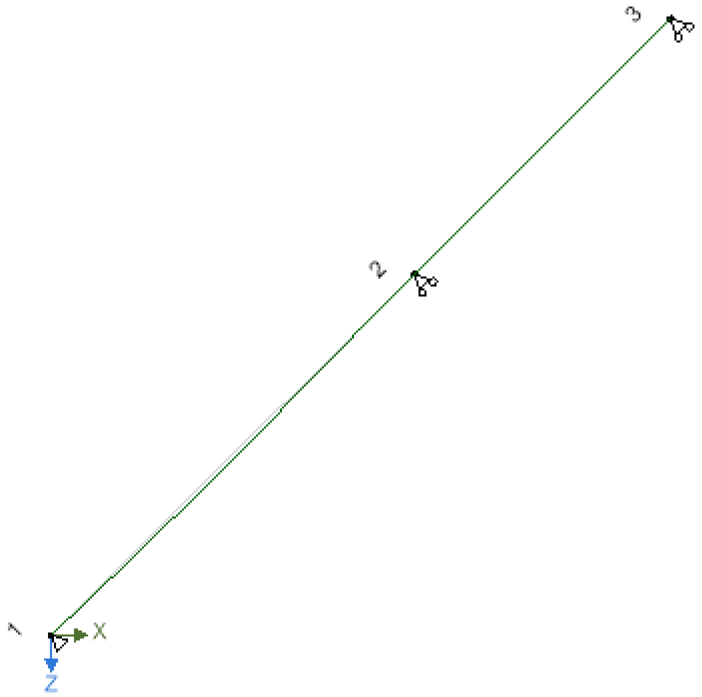
1.35G + 1.50Q + ψ_S1.50S (Strength) - Total deflection



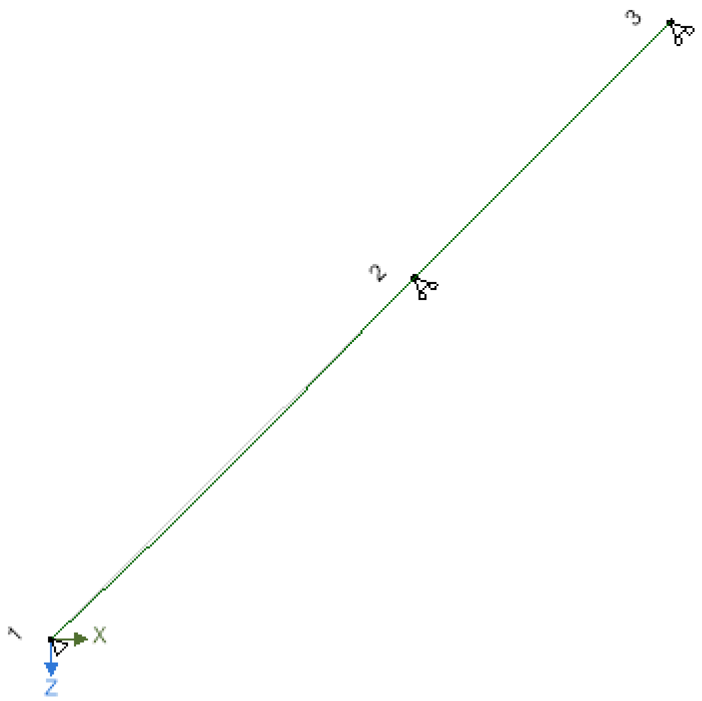
1.35G + $\psi_0$1.50Q + 1.50S (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



1.00G + 1.00Q + ψs1.00S (Service) - Total deflection



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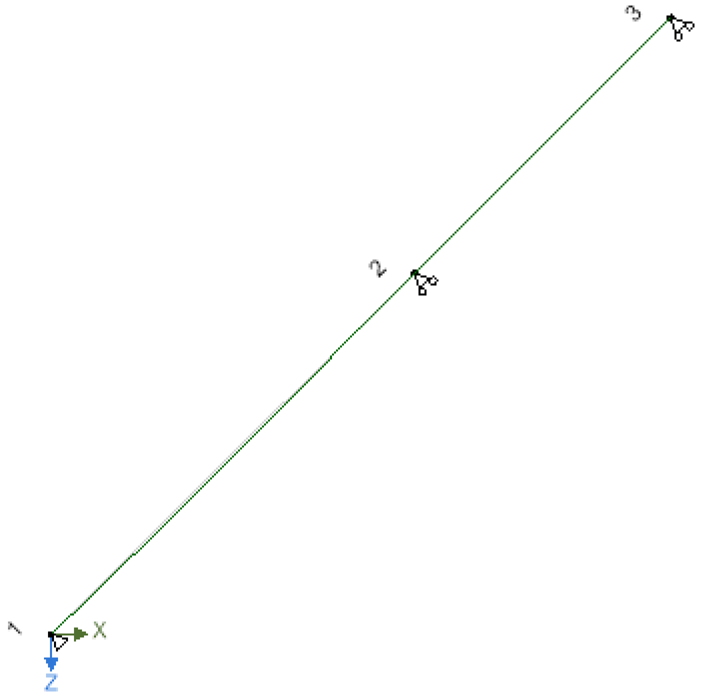
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MKA	07/12/2021				

1.00G + ψ21.00Q (Quasi) - 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	0.18618	Member
2	-0.1	0	-0.06341	Member
3	-0.1	0	-0.02721	Member

Load combination: 1.35G + 1.50Q + ψs1.50S (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.21592	Member
2	-0.1	0	-0.07353	Member
3	-0.1	0	-0.03156	Member

Load combination: 1.35G + ψ01.50Q + 1.50S (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.22336	Member
2	-0.1	0	-0.07607	Member
3	-0.1	0	-0.03265	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.1324	Member
2	0	0	-0.04509	Member
3	-0.1	0	-0.01935	Member

Load combination: 1.00G + 1.00Q + ψ_s1.00S (Service)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.15223	Member
2	-0.1	0	-0.05184	Member
3	-0.1	0	-0.02225	Member

Load combination: 1.00G + $\psi_2$1.00Q (Quasi)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.0977	Member
2	0	0	-0.03327	Member
3	0	0	-0.01428	Member

Total base reactions

Load case/combination	Force FX (kN)	Force FZ (kN)
1.35G + 1.50Q (Strength)	0	4.2
1.35G + 1.50Q + ψ_s1.50S (Strength)	0	4.9
1.35G + $\psi_0$1.50Q + 1.50S (Strength)	0	5
1.00G + 1.00Q (Service)	0	3
1.00G + 1.00Q + ψ_s1.00S (Service)	0	3.4
1.00G + $\psi_2$1.00Q (Quasi)	0	2.2

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	2.05	1	-3	-0.7	0
		2	1.2	-1	-0.3
2	1.45	2	-1.2	-0.9	0.3
		3	0	-0.4	0

Load combination: 1.35G + 1.50Q + ψ_s1.50S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-3.4	-0.8	0
		2	1.4	-1.2	-0.4
2	1.45	2	-1.4	-1	0.4
		3	0	-0.4	0

Load combination: 1.35G + $\psi_0$1.50Q + 1.50S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-3.5	-0.8	0
		2	1.5	-1.2	-0.4
2	1.45	2	-1.5	-1	0.4
		3	0	-0.4	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-2.1	-0.5	0
		2	0.9	-0.7	-0.2
2	1.45	2	-0.9	-0.6	0.2
		3	0	-0.3	0

Load combination: 1.00G + 1.00Q + ψ_S1.00S (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-2.4	-0.6	0
		2	1	-0.8	-0.3
2	1.45	2	-1	-0.7	0.3
		3	0	-0.3	0

Load combination: 1.00G + $\psi_2$1.00Q (Quasi)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-1.6	-0.4	0
		2	0.6	-0.5	-0.2
2	1.45	2	-0.6	-0.4	0.2
		3	0	-0.2	0

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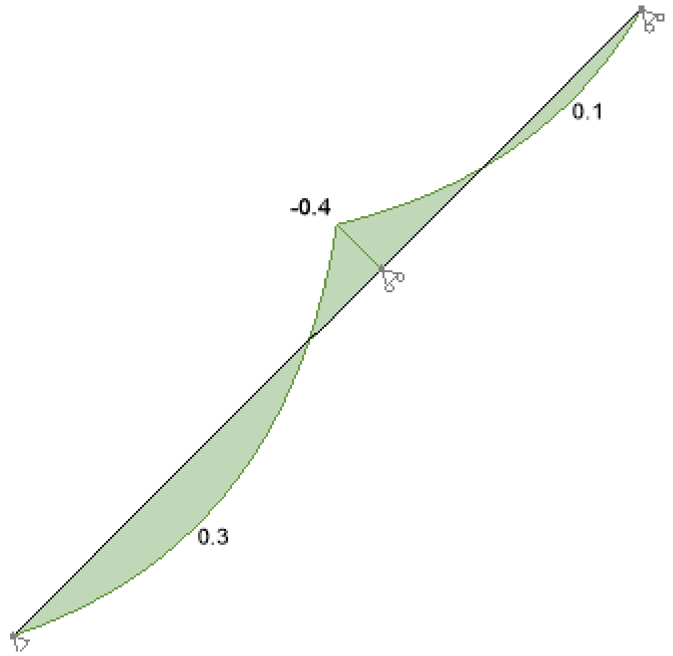
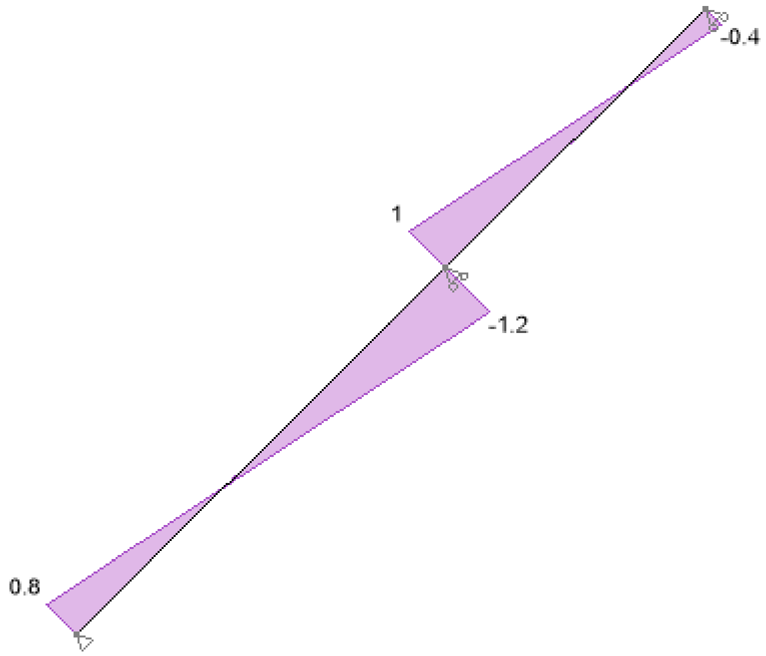
	Calcs by MKA	Calcs date 07/12/2021	Checked by	Checked date	Approved by	Approved date

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	0.825	0		0.3 (max)	0.3
	2.05	1	-1.2 (max abs)	-0.3	-0.4 (min)

Tedds calculation version 2.2.14

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

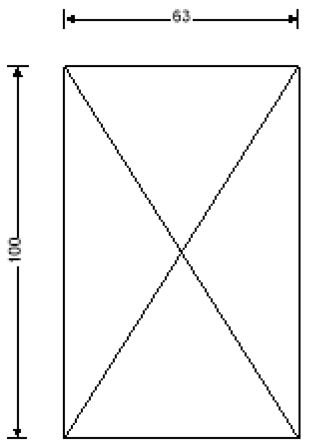
Timber section details

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

Breadth of sections b = **63** mm

Depth of sections h = **100** mm

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



63x100 timber section

Cross-sectional area, A, 6300 mm²

Section modulus, W_y, 105000 mm³

Section modulus, W_z, 66150 mm³

Second moment of area, I_y, 5250000 mm⁴

Second moment of area, I_z, 2083725 mm⁴

Radius of gyration, i_y, 28.9 mm

Radius of gyration, i_z, 18.2 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

Member span 1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm²	14.2	0.6	0.040	PASS
Bending stress	N/mm²	17.6	4.0	0.225	PASS
Shear stress	N/mm²	2.7	0.4	0.163	PASS
Bending and axial force				0.225	PASS
Column stability check				0.251	PASS
Deflection	mm	8.2	4.0	0.493	PASS

Consider Combination 3 - 1.35G + $\psi_0$1.50Q + 1.50S (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**

Depth factor for bending - Major axis - exp.3.1 — $k_{h,m,y} = \min((150 \text{ mm} / h)^{0.2}, 1.3)$ = **1.084**

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 = **3.548** kN

Design compressive stress — $\sigma_{c,0,d} = P_d / A$ = **0.563** N/mm^2

Design compressive strength — $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **14.215** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.040**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at end of span

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **1.242** kN

Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.441** 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

$\tau_{y,d} / f_{v,y,d}$ = **0.163**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment — $M_{y,d}$ = **0.416** kNm

Design bending stress — $\sigma_{m,y,d} = M_{y,d} / W_y$ = **3.958** N/mm^2

Design bending strength — $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **17.618** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.225**

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 — $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.226**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.159**

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 — $L_{e,y} = 0.9 \times 2050$ mm = **1845** mm

Slenderness ratio — $\lambda_y = L_{e,y} / i_y$ = **63.913**

Relative slenderness ratio - exp. 6.21 — $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.084**

Effective length for z-axis bending — $L_{e,z}$ = **0** 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 — β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 — $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.166**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **0.470**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.627**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **1.064**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.288**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.194**

PASS - Column stability is acceptable

Consider Combination 2 - 1.35G + 1.50Q + ψ_S1.50S (Strength)

Check design 923 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **2.2** mm

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

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **4** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **8.2** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.493**

PASS - Allowable deflection exceeds final deflection

Member - Span 2

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

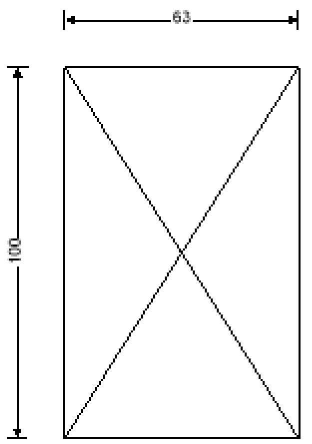
Timber section details

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

Breadth of sections b = **63** mm

Depth of sections h = **100** mm

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



63x100 timber section
Cross-sectional area, A, 6300 mm²
Section modulus, W_y, 105000 mm³
Section modulus, W_z, 66150 mm³
Second moment of area, I_y, 5250000 mm⁴
Second moment of area, I_z, 2083725 mm⁴
Radius of gyration, i_y, 28.9 mm
Radius of gyration, i_z, 18.2 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

Member span 2 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm²	14.2	0.2	0.016	PASS
Bending stress	N/mm²	17.6	4.0	0.225	PASS
Shear stress	N/mm²	2.7	0.4	0.134	PASS

Bending and axial force				0.225	PASS
Column stability check				0.244	PASS
Deflection	mm	5.8	0.3	0.059	PASS

Consider Combination 3 - 1.35G + $\psi_0$1.50Q + 1.50S (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**

Depth factor for bending - Major axis - exp.3.1 — $k_{h,m,y}$ = min((150 mm / h)$^{0.2}$, 1.3) = **1.084**

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 = **1.47** kN

Design compressive stress — $\sigma_{c,0,d} = P_d / A$ = **0.233** N/mm^2

Design compressive strength — $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **14.215** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.016**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **1.022** kN

Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.363** 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

$\tau_{y,d} / f_{v,y,d}$ = **0.134**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment — $M_{y,d}$ = **0.416** kNm

Design bending stress — $\sigma_{m,y,d} = M_{y,d} / W_y$ = **3.958** N/mm^2

Design bending strength — $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **17.618** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.225**

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 — $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.225**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.158**

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 × 1450 mm = **1305** mm

Slenderness ratio — $\lambda_y = L_{e,y} / i_y$ = **45.207**

Relative slenderness ratio - exp. 6.21 — $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.767**

Effective length for z-axis bending — $L_{e,z}$ = **0** 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 — β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **0.840**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **0.470**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.844**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) =$ **1.064**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} =$ **0.244**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} =$ **0.173**

PASS - Column stability is acceptable

Consider Combination 2 - 1.35G + 1.50Q + ψ_S1.50S (Strength)

Check design 976 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y =$ **0.2** mm

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

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def}) =$ **0.3** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s2} / 250 =$ **5.8** mm

$\delta_{y,Final} / \delta_{y,Allowable} =$ **0.059**

PASS - Allowable deflection exceeds final deflection