Structural Calculation Document

Moss Farm Cottages
Lower Moss Farm
Lower Lane
Longridge

Roof & Floor Timbers

March 2023

Description

The following calculations are for the new roof and floor structure at Moss Farm Cottages, Lower Moss Farm, Lower Lane, Longridge.

This document relates to the primary timber members of the floor plate and roof of the proposed cottages. All other structural items such as foundations, walls, lintels etc are to be confirmed by others.

All dimensions and details are indicative and should be checked to be suitable by on site measurement.

For all ancillary items such as wall plates, straps, joist etc, refer to the Building Regulations.

Moss Farm Cottages
Lower Moss Farm
Lower Lane
Longridge

29/03/2023

Roof Floor Timbers

LOADING SCHEDULE

ROOF - PITCHED

DEAD

	Slate on Batons		0.65	
	Rafters		0.15	
	Purlins		0.10	
	Ceiling Joists		0.15	
	Insul. & P/Board		0.15	
Slope =	***30 degrees***	Total (plan)	1.39	kN\m^2

IMPOSED

	Roof (Snow)		0.75	
		Total	0.75	kN\m^2

FIRST FLOOR

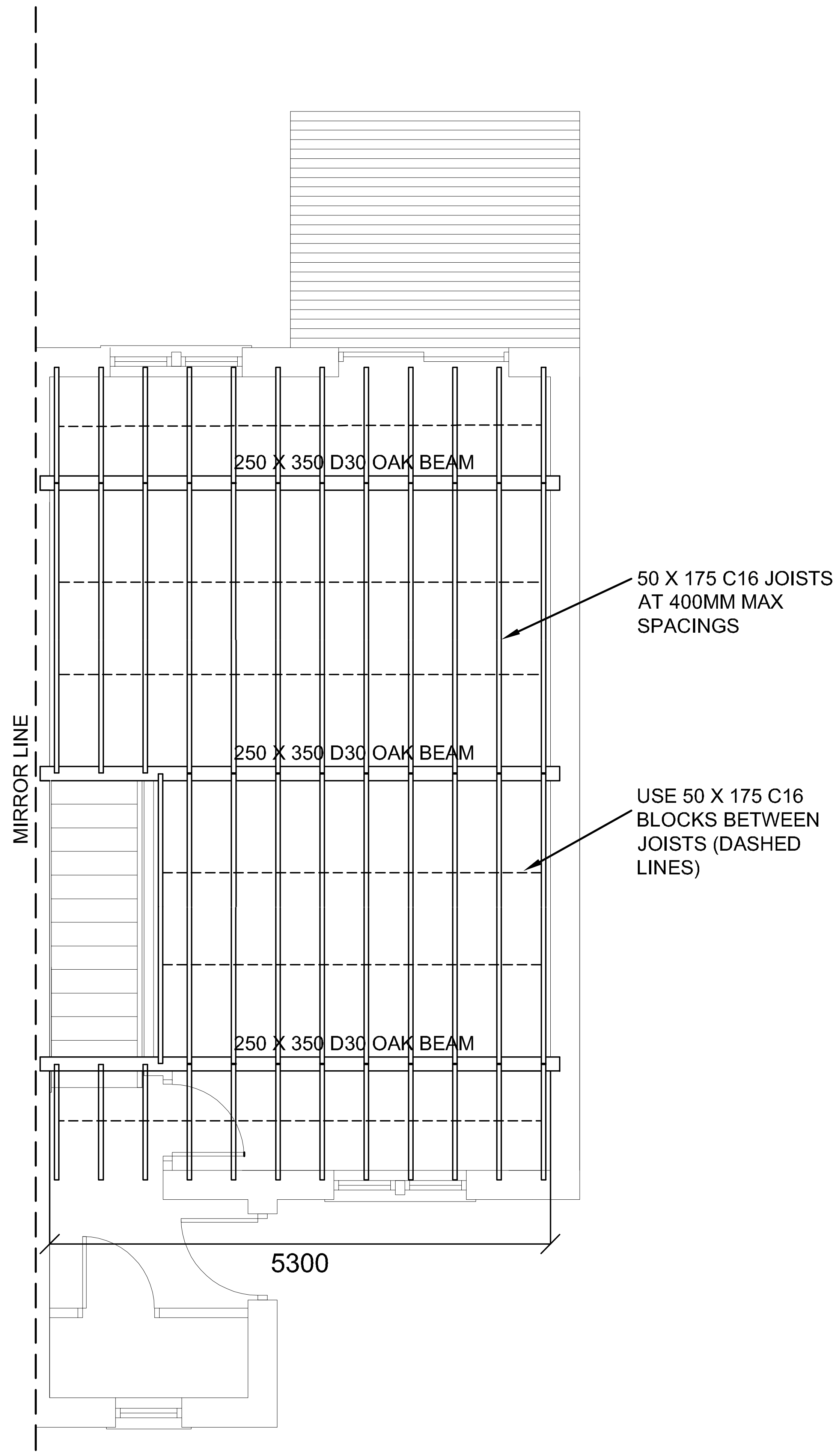
DEAD

	Finishes		0.10	
	Chipboard		0.15	
	Joists		0.20	
	Insul. & P/Board		0.15	
		Total	0.60	kN\m^2

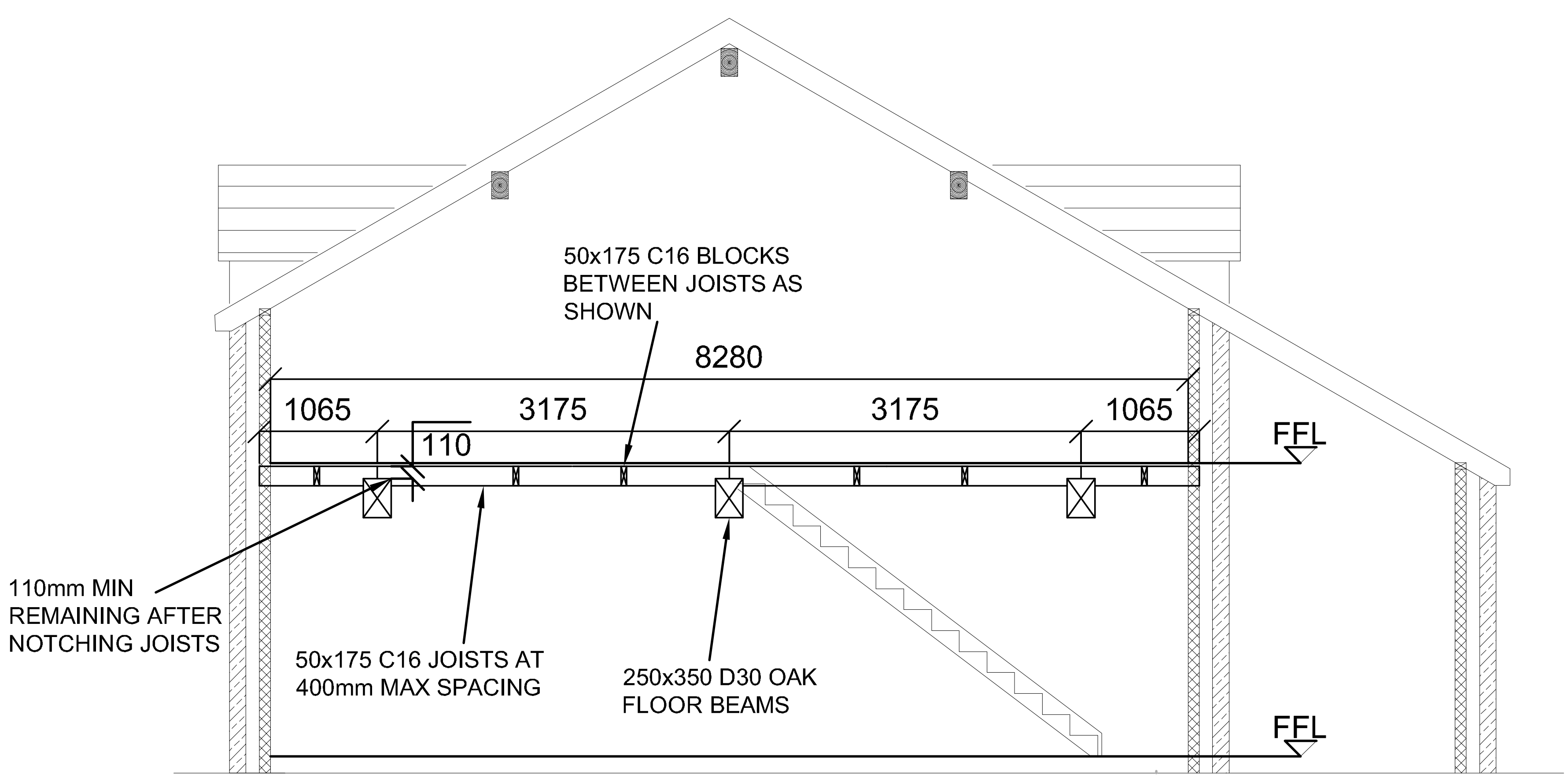
IMPOSED

	Domestic		1.50	
	Partitions		0.50	
		Total	2.00	kN\m^2

1ST FLOOR DESIGN



1ST FLOOR PLAN



SECTION

	Project				Job no.	
	Moss Farm Cottages					
	Calcs for				Start page no./Revision	
	First Floor Joists				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	MC	29/03/2023				

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

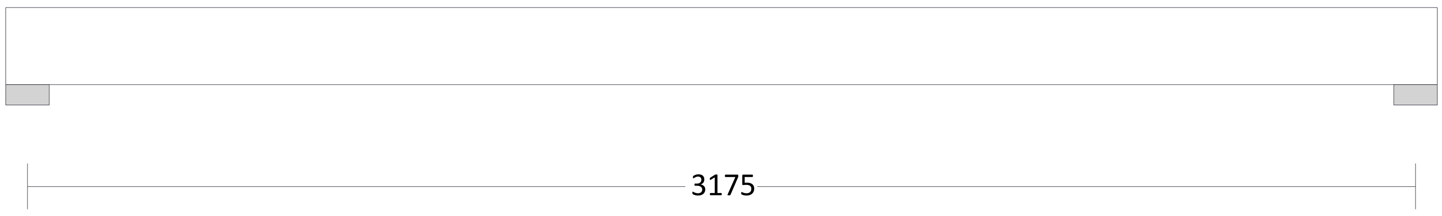
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the recommended values

Tedds calculation version 1.0.04

Joist details

Description: 47 x 175 C16 timber joists

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



Forces input on Joist

Vertical permanent load on joist: F_{G_Joist}= **0.40** kN/m^2

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

Joist loading details

Distributed loads

Vertical permanent load on joist: $p_G = F_{G\_Joist} * s_{Joist}$= **0.16** kN/m

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

ANALYSIS

Tedds calculation version 1.0.35

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

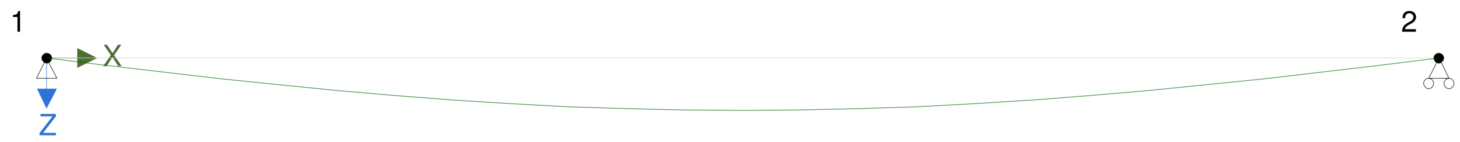
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.16 kN/m at 0 m to 3.175 m
Member	Imposed	UDL	GlobalZ	0.8 kN/m at 0 m to 3.175 m

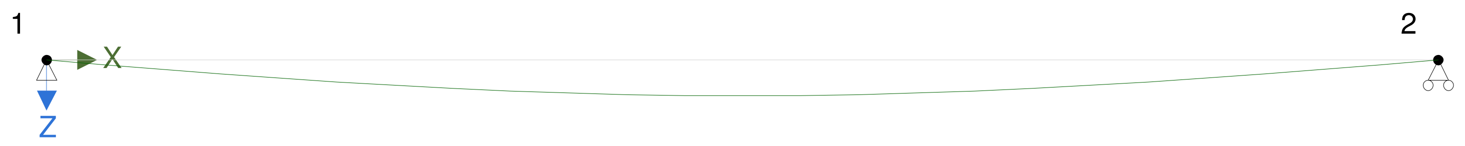
Results

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	0.65965	
2	0	0	-0.65965	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.44819	
2	0	0	-0.44819	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.6
1.00G + 1.00Q (Service)	0	3.1

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

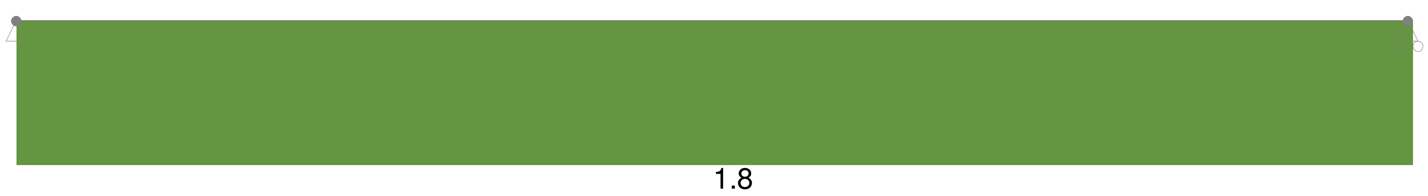
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.175	1	0	-2.3	0
		2	0	-2.3	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.175	1	0	-1.6	0
		2	0	-1.6	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	2.3 (max abs)		0 (min)	
	1.588	0		1.8 (max)	
	3.175	-2.3		0 (min)	

Tedds calculation version 2.2.07

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 1

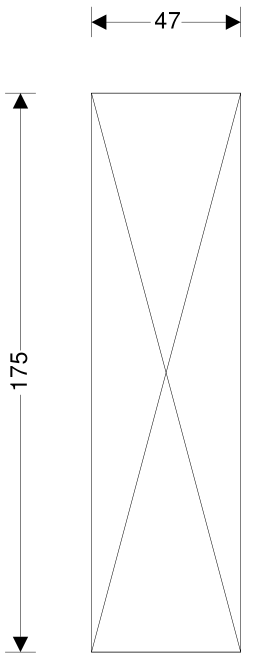
Timber section details

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

Breadth of sections b = **47** mm

Depth of sections h = **175** mm

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



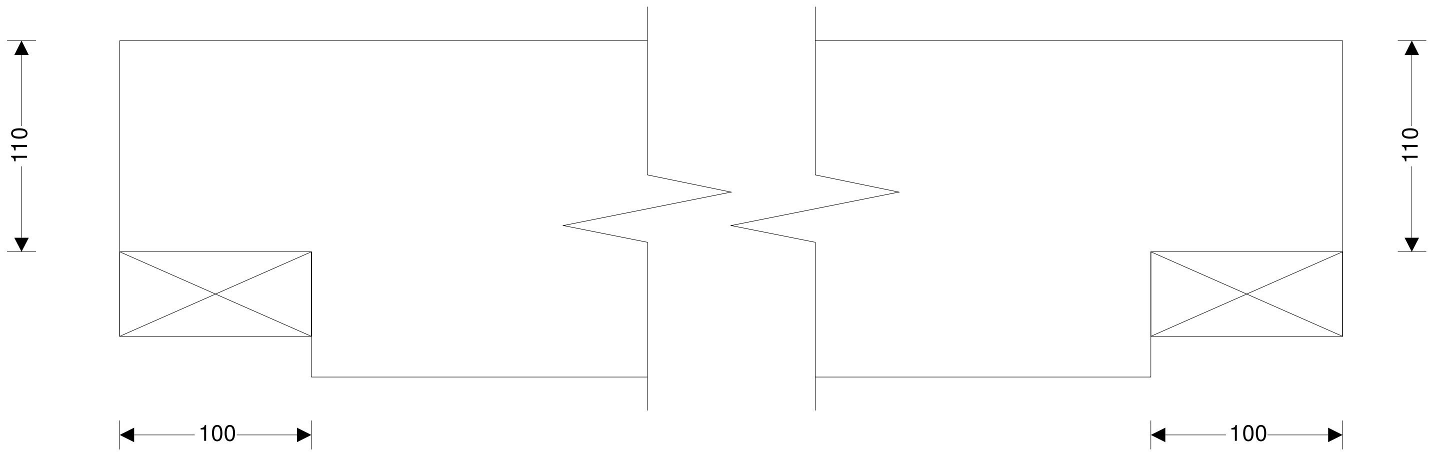
47x175 timber section
Cross-sectional area, A, 8225 mm^2
Section modulus, W_y, 239895.8 mm^3
Section modulus, W_z, 64429 mm^3
Second moment of area, I_y, 20990885 mm^4
Second moment of area, I_z, 1514085 mm^4
Radius of gyration, i_y, 50.5 mm
Radius of gyration, i_z, 13.6 mm
Timber strength class C16
Characteristic bending strength, $f_{m.k}$, 16 N/mm^2
Characteristic shear strength, $f_{v.k}$, 3.2 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 17 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.2 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 8.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 310 kg/m^3
Mean density, ρ_{mean}, 370 kg/m^3

Project				Job no.	
Moss Farm Cottages					
Calcs for First Floor Joists				Start page no./Revision 4	
Calcs by MC	Calcs date 29/03/2023	Checked by	Checked date	Approved by	Approved date

Span details

Bearing length L_b = **100** mm

Depth at notched end h_{ef} = **110** mm



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

Modification factors

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

Deformation factor - Table 3.2 k_{def} = **0.6**

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**

Notch inclination i_n = **0**

Distance from support reaction to corner of notch $x_n = L_b / 2 + L_x$ = **50** mm

Material faction - exp.6.63 k_n = **5**

Member end depth ratio $\alpha_n = h_{ef} / h$ = **0.629**

Notch reduction factor - exp.6.62

$k_v = \min(1, k_n * (1 + 1.1 * i_n^{1.5} / \sqrt{(h / 1 mm)}) / (\sqrt{(h / 1 mm)} * (\sqrt{(\alpha_n * (1 - \alpha_n))} + 0.8 * x_n / h * \sqrt{(1 / \alpha_n - \alpha_n^2)}))) = \mathbf{0.516}$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **2.301** kN

Effective contact length $L_{b,ef} = L_b$ = **100** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b * L_{b,ef})$ = **0.490** N/mm^2

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

$\sigma_{c,y,90,d} / (k_{c,90} * f_{c,y,90,d})$ = **0.329**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **2.301** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 * F_{y,d} / (k_{cr} * b * h_{ef})$ = **0.997** N/mm^2

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

$\tau_{y,d} / (k_v * f_{v,y,d})$ = **0.892**

PASS - Design shear strength exceeds design shear stress

Check design 1588 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **1.827** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **7.615** N/mm^2

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

$\sigma_{m,y,d} / f_{m,y,d} =$ **0.703**

PASS - Design bending strength exceeds design bending stress

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} =$ **2.301** kN

Effective contact length $L_{b,ef} = L_b =$ **100** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b * L_{b,ef}) =$ **0.490** N/mm^2

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

$\sigma_{c,y,90,d} / (k_{c,90} * f_{c,y,90,d}) =$ **0.329**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} =$ **2.301** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 * F_{y,d} / (k_{cr} * b * h_{ef}) =$ **0.997** N/mm^2

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

$\tau_{y,d} / (k_v * f_{v,y,d}) =$ **0.892**

PASS - Design shear strength exceeds design shear stress

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

Check design 1587 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y =$ **8.1** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250 =$ **12.7** mm

$\delta_y / \delta_{y,Allowable} =$ **0.64**

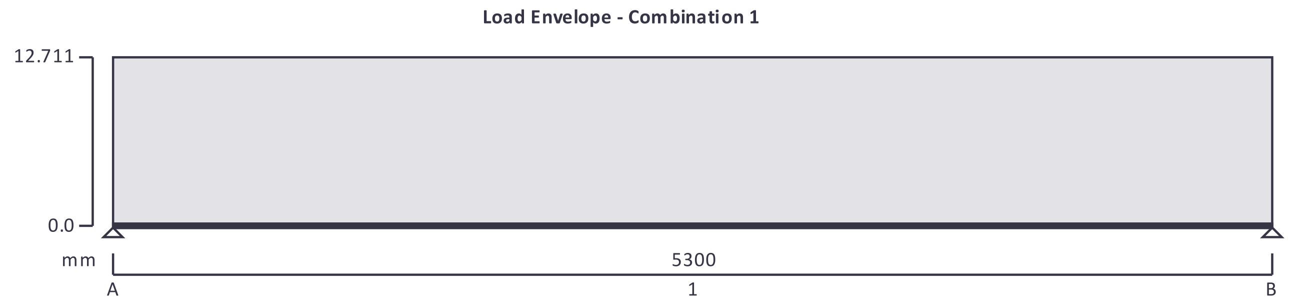
PASS - Allowable deflection exceeds final deflection

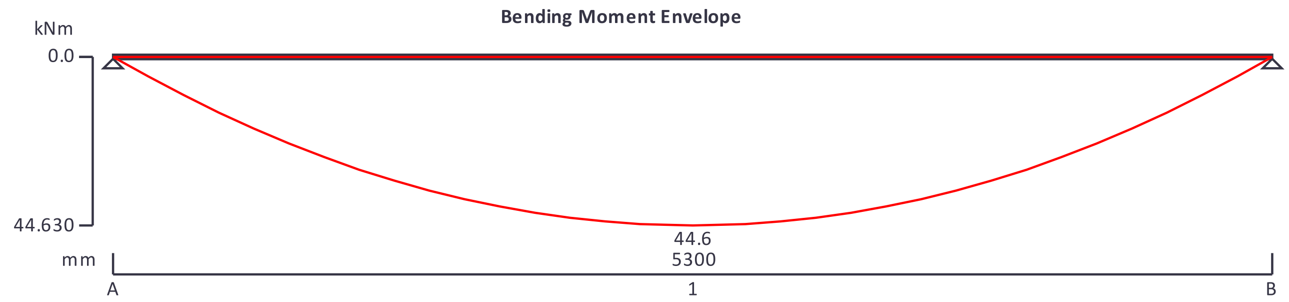
	Project				Job no.	
	Moss Farm Cottages					
	Calcs for				Start page no./Revision	
	Oak Floor Beams				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	MC	29/03/2023				

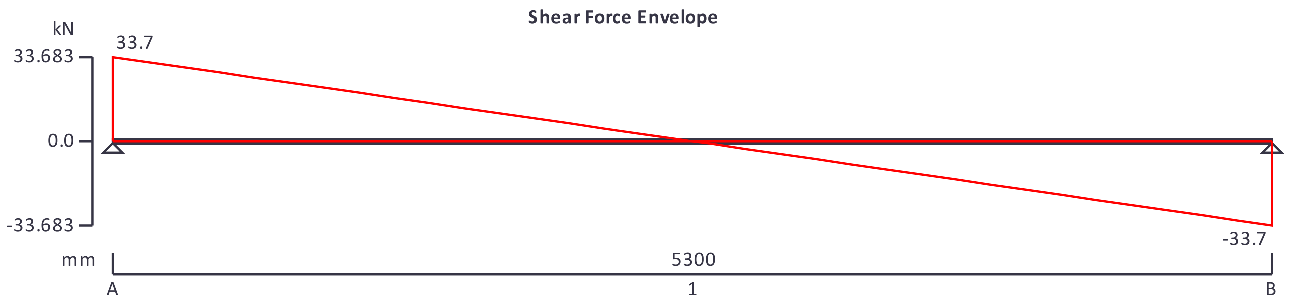
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

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 1.7.04







Applied loading

Beam loads

	Permanent self weight of beam * 1
Dead	Permanent full UDL 1.905 kN/m
Imposed	Variable full UDL 6.350 kN/m

Load combinations

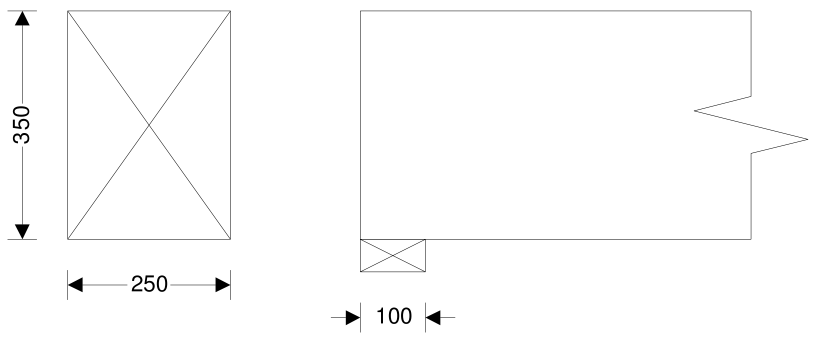
Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
	Span 1	Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

Maximum moment	M_{max} = **44.630** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **44.630** kNm	
Maximum shear	F_{max} = **33.683** kN	F_{min} = **-33.683** kN

Project				Job no.	
Moss Farm Cottages					
Calcs for				Start page no./Revision	
Oak Floor Beams				2	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MC	29/03/2023				

Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **33.683** kN	
Total load on beam	W_{tot} = **67.367** kN	
Reactions at support A	R_{A_max} = **33.683** kN	R_{A_min} = **33.683** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **6.253** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **16.828** kN	
Reactions at support B	R_{B_max} = **33.683** kN	R_{B_min} = **33.683** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **6.253** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **16.828** kN	



Timber section details

Breadth of timber sections — b = **250** mm

Depth of timber sections — h = **350** mm

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

Overall breadth of timber member — $b_b = N \times b$ = **250** mm

Timber strength class - EN 338:2016 Table 3 — **D30**

Member details

Load duration - cl.2.3.1.2 — **Medium-term**

Service class of timber - cl.2.3.1.3 — **1**

Length of span — L_{s1} = **5300** mm

Length of bearing — L_b = **100** mm

Section properties

Cross sectional area of member — $A = N \times b \times h$ = **87500** mm^2

Section modulus — $W_y = N \times b \times h^2 / 6$ = **5104167** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **3645833** mm^3

Second moment of area — $I_y = N \times b \times h^3 / 12$ = **893229167** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **455729167** mm^4

Radius of gyration — $r_y = \sqrt{(I_y / A)}$ = **101.0** mm

$r_z = \sqrt{(I_z / A)}$ = **72.2** mm

Partial factor for material properties and resistances

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

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.800**

Deformation factor for service classes - Table 3.2 — k_{def} = **0.600**

Depth factor for bending - exp.3.1 — $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.1 — $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.670**

Load configuration factor - exp.6.4 — $k_{c.90}$ = **1.000**

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

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

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress — $\sigma_{c.90.d} = R_{A\_max} / (N * b * L_b)$ = **1.347** N/mm^2

Design compressive strength — $f_{c.90.d} = k_{mod} * k_{sys} * k_{c.90} * f_{c.90.k} / \gamma_M$ = **3.262** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.413**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress — $\sigma_{m.d} = M / W_y$ = **8.744** N/mm^2

Design bending strength — $f_{m.d} = k_{h.m} * k_{mod} * k_{sys} * k_{crit} * f_{m.k} / \gamma_M$ = **18.462** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **0.474**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress — $\tau_d = 3 * F / (2 * k_{cr} * A)$ = **0.862** N/mm^2

Permissible shear stress — $f_{v.d} = k_{mod} * k_{sys} * f_{v.k} / \gamma_M$ = **2.400** N/mm^2

$\tau_d / f_{v.d}$ = **0.359**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit — δ_{lim} = min(14 mm, 0.004 * L_{s1}) = **14.000** mm

Instantaneous deflection due to permanent load — δ_{instG} = **2.632** mm

Final deflection due to permanent load — $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **4.212** mm

Instantaneous deflection due to variable load — δ_{instQ} = **7.083** mm

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

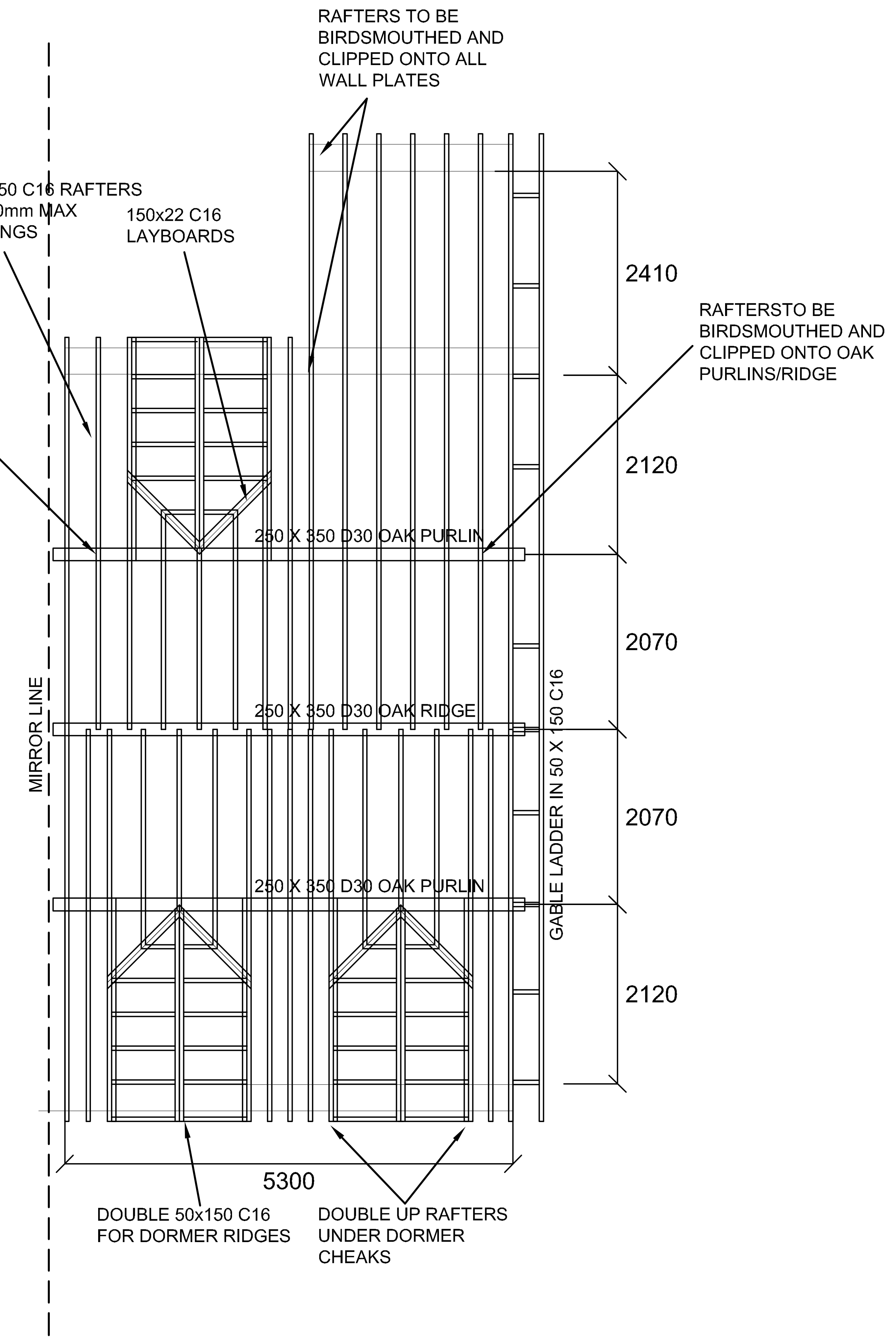
Final deflection due to variable load — $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **8.358** mm

Total final deflection — $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **12.570** mm

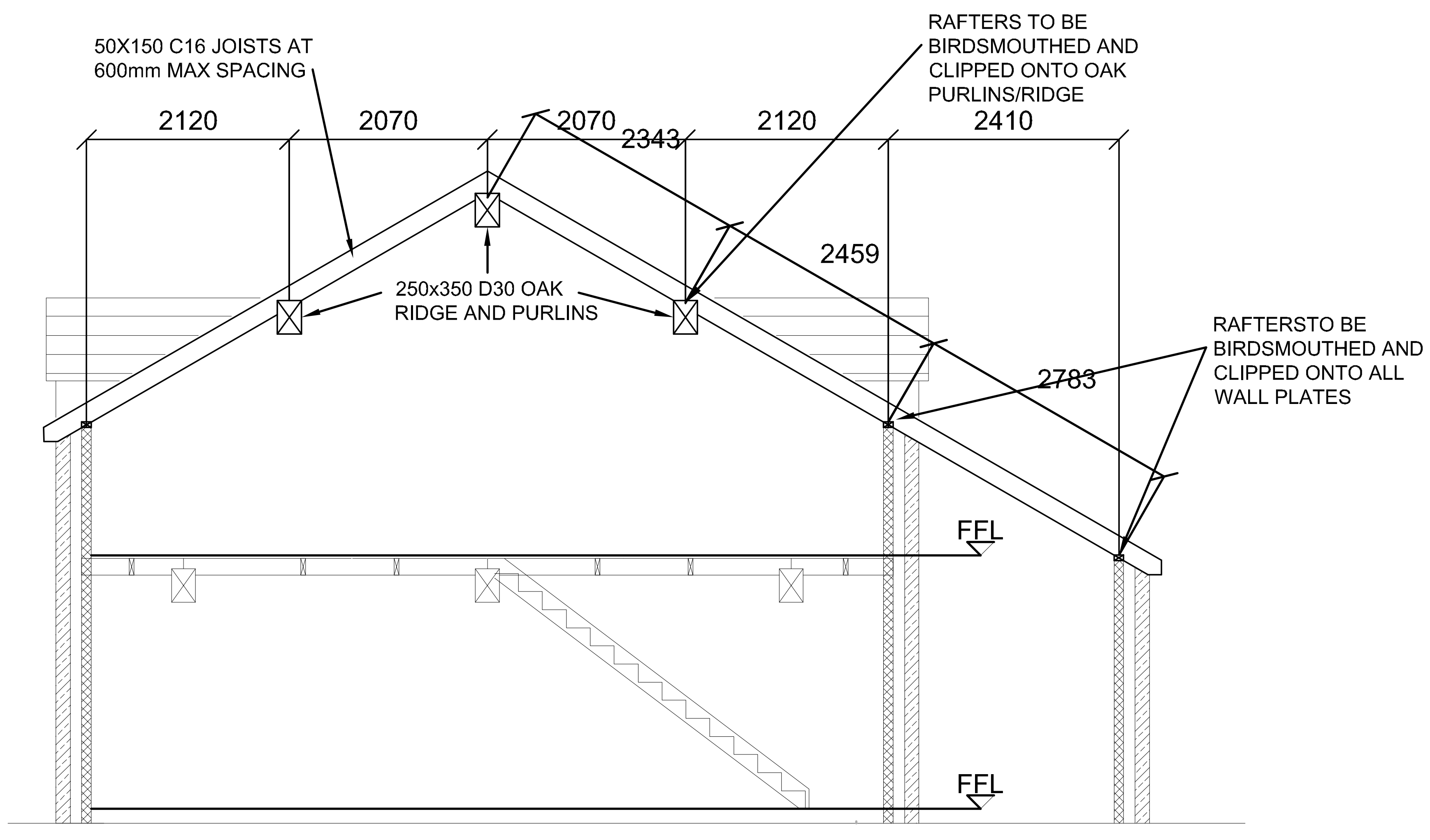
$\delta_{fin} / \delta_{lim}$ = **0.898**

PASS - Total final deflection is less than the deflection limit

ROOF DESIGN



ROOF PLAN

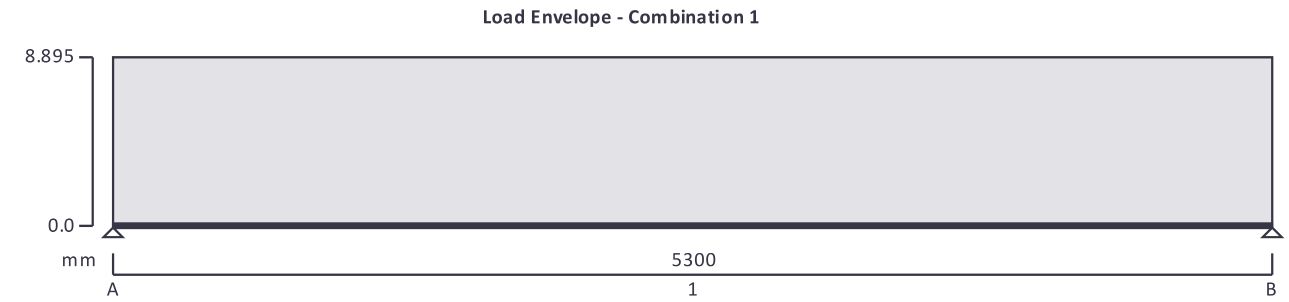


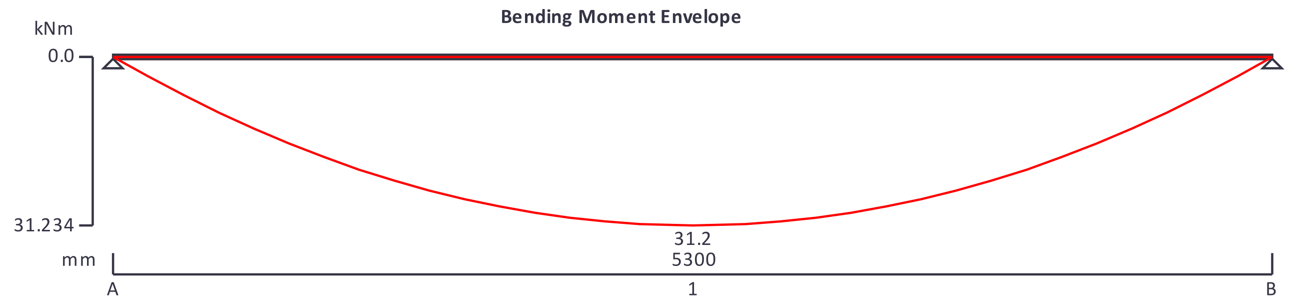
SECTION

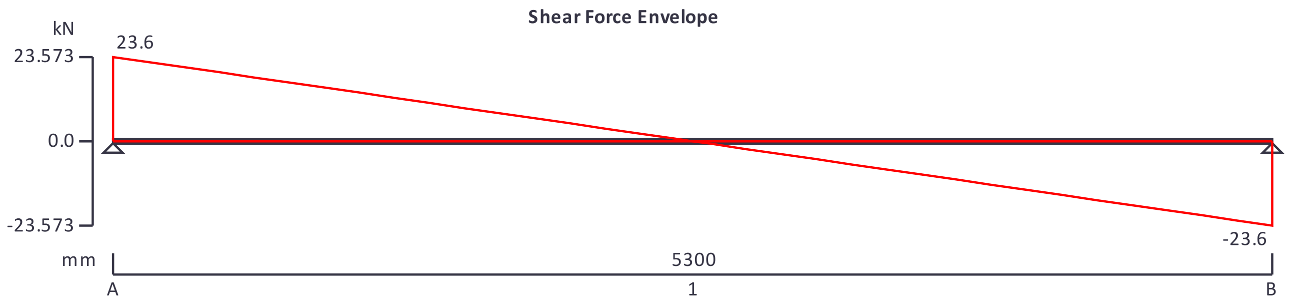
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

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 1.7.04







Applied loading

Beam loads

	Permanent self weight of beam * 1
Dead	Permanent full UDL 4.390 kN/m
Imposed	Variable full UDL 1.570 kN/m

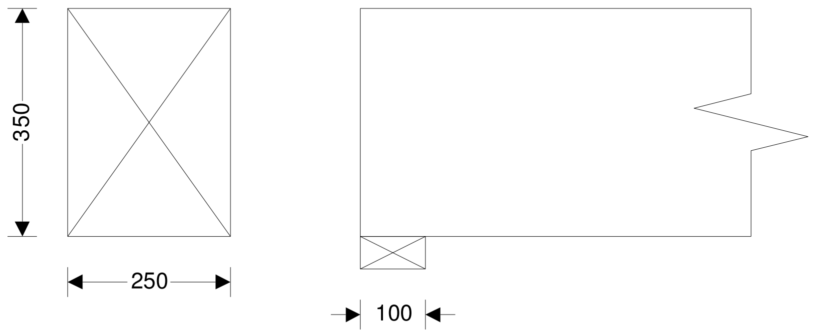
Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
	Span 1	Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

Maximum moment	M_{max} = **31.234** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **31.234** kNm	
Maximum shear	F_{max} = **23.573** kN	F_{min} = **-23.573** kN

Design shear	$F = \max(\mathrm{abs}(F_{max}),\mathrm{abs}(F_{min}))$ = **23.573** kN	
Total load on beam	W_{tot} = **47.146** kN	
Reactions at support A	R_{A_max} = **23.573** kN	R_{A_min} = **23.573** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **12.839** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **4.161** kN	
Reactions at support B	R_{B_max} = **23.573** kN	R_{B_min} = **23.573** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **12.839** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **4.161** kN	



Timber section details

Breadth of timber sections: b = **250** mm

Depth of timber sections: h = **350** mm

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

Overall breadth of timber member: $b_b = N \times b$ = **250** mm

Timber strength class - EN 338:2016 Table 3: **D30**

Member details

Load duration - cl.2.3.1.2: **Medium-term**

Service class of timber - cl.2.3.1.3: **1**

Length of span: L_{s1} = **5300** mm

Length of bearing: L_b = **100** mm

Section properties

Cross sectional area of member: $A = N \times b \times h$ = **87500** mm^2

Section modulus: $W_y = N \times b \times h^2 / 6$ = **5104167** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **3645833** mm^3

Second moment of area: $I_y = N \times b \times h^3 / 12$ = **893229167** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **455729167** mm^4

Radius of gyration: $r_y = \sqrt{(I_y / A)}$ = **101.0** mm

$r_z = \sqrt{(I_z / A)}$ = **72.2** mm

Partial factor for material properties and resistances

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

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.800**

Deformation factor for service classes - Table 3.2: k_{def} = **0.600**

Depth factor for bending - exp.3.1: $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.1: $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2): k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2): k_{cr} = **0.670**

Load configuration factor - exp.6.4: $k_{c.90}$ = **1.000**

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

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

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{A\_max} / (N * b * L_b) =$ **0.943** N/mm^2

Design compressive strength $f_{c.90.d} = k_{mod} * k_{sys} * k_{c.90} * f_{c.90.k} / \gamma_M =$ **3.262** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d} =$ **0.289**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = M / W_y =$ **6.119** N/mm^2

Design bending strength $f_{m.d} = k_{h.m} * k_{mod} * k_{sys} * k_{crit} * f_{m.k} / \gamma_M =$ **18.462** N/mm^2

$\sigma_{m.d} / f_{m.d} =$ **0.331**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 * F / (2 * k_{cr} * A) =$ **0.603** N/mm^2

Permissible shear stress $f_{v.d} = k_{mod} * k_{sys} * f_{v.k} / \gamma_M =$ **2.400** N/mm^2

$\tau_d / f_{v.d} =$ **0.251**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(14\ mm, 0.004 * L_{s1}) =$ **14.000** mm

Instantaneous deflection due to permanent load $\delta_{instG} =$ **5.404** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def}) =$ **8.647** mm

Instantaneous deflection due to variable load $\delta_{instQ} =$ **1.751** mm

Factor for quasi-permanent variable action $\psi_2 =$ **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) =$ **2.066** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ} =$ **10.713** mm

$\delta_{fin} / \delta_{lim} =$ **0.765**

PASS - Total final deflection is less than the deflection limit

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

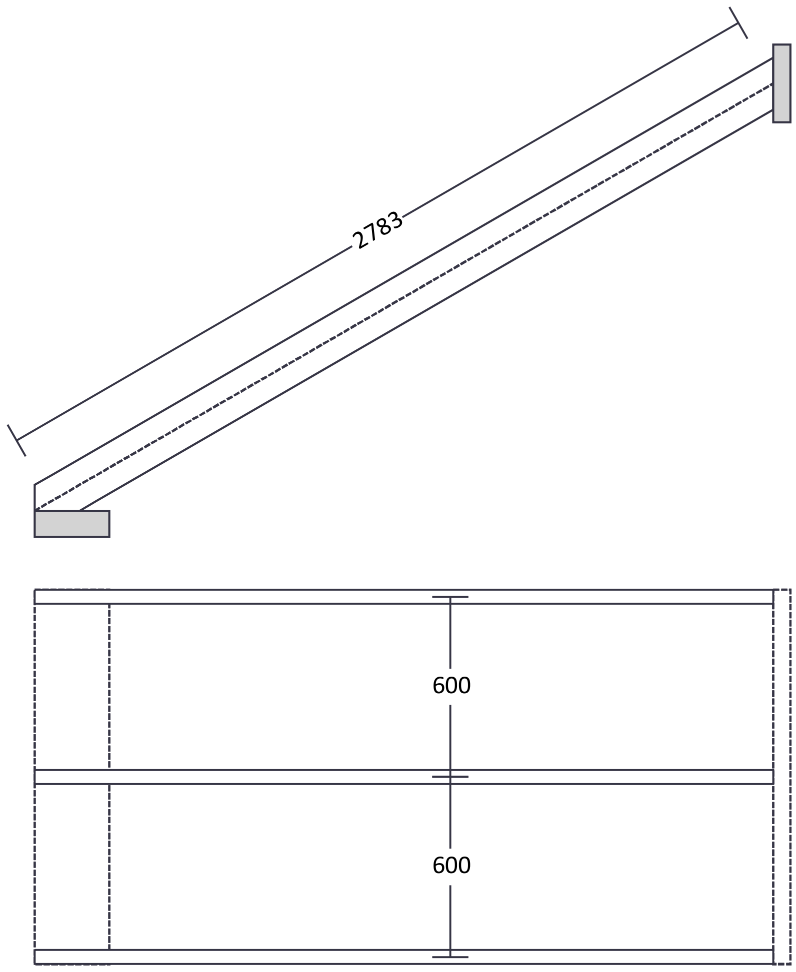
Tedds calculation version 1.0.04

Rafter details

Description 47 x 150 C16 timber rafters

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

Rafter inclination θ_{Rafter} = **30** deg



Forces input on Rafter

Permanent load on slope F_{G_Rafter}= **1.10** kN/m^2

Snow load on plan F_{S_Rafter}= **0.75** kN/m^2

Rafter loading details

Distributed loads

Permanent load on slope $p_G = F_{G\_Rafter} * s_{Rafter}$= **0.66** kN/m

Snow load on slope $p_S = F_{S\_Rafter} * s_{Rafter} * \cos(\theta_{Rafter})$= **0.39** kN/m

ANALYSIS

Tedds calculation version 1.0.35

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

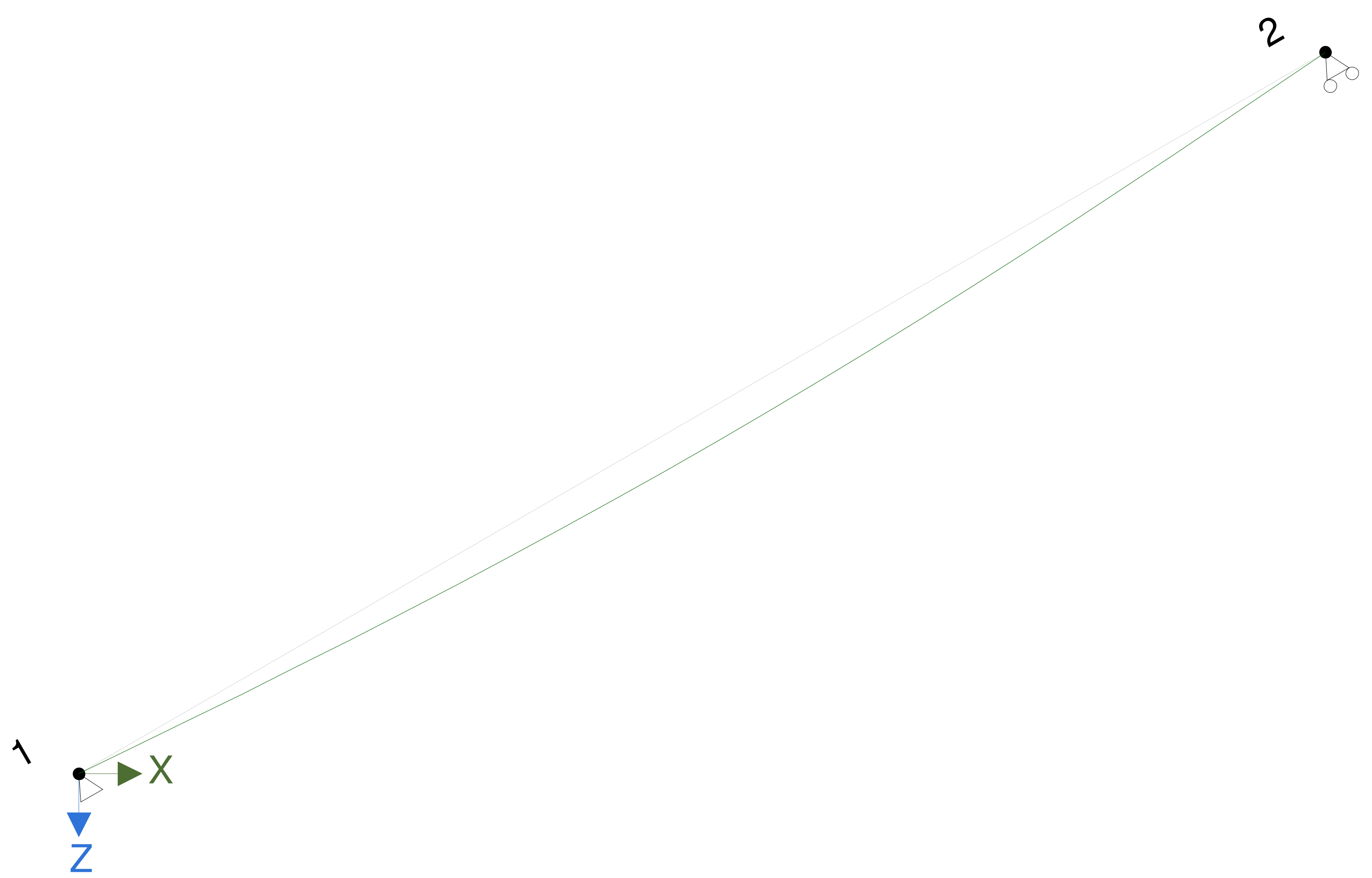
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.66 kN/m at 0 m to 2.783 m
Member	Snow	UDL	GlobalZ	0.39 kN/m at 0 m to 2.783 m

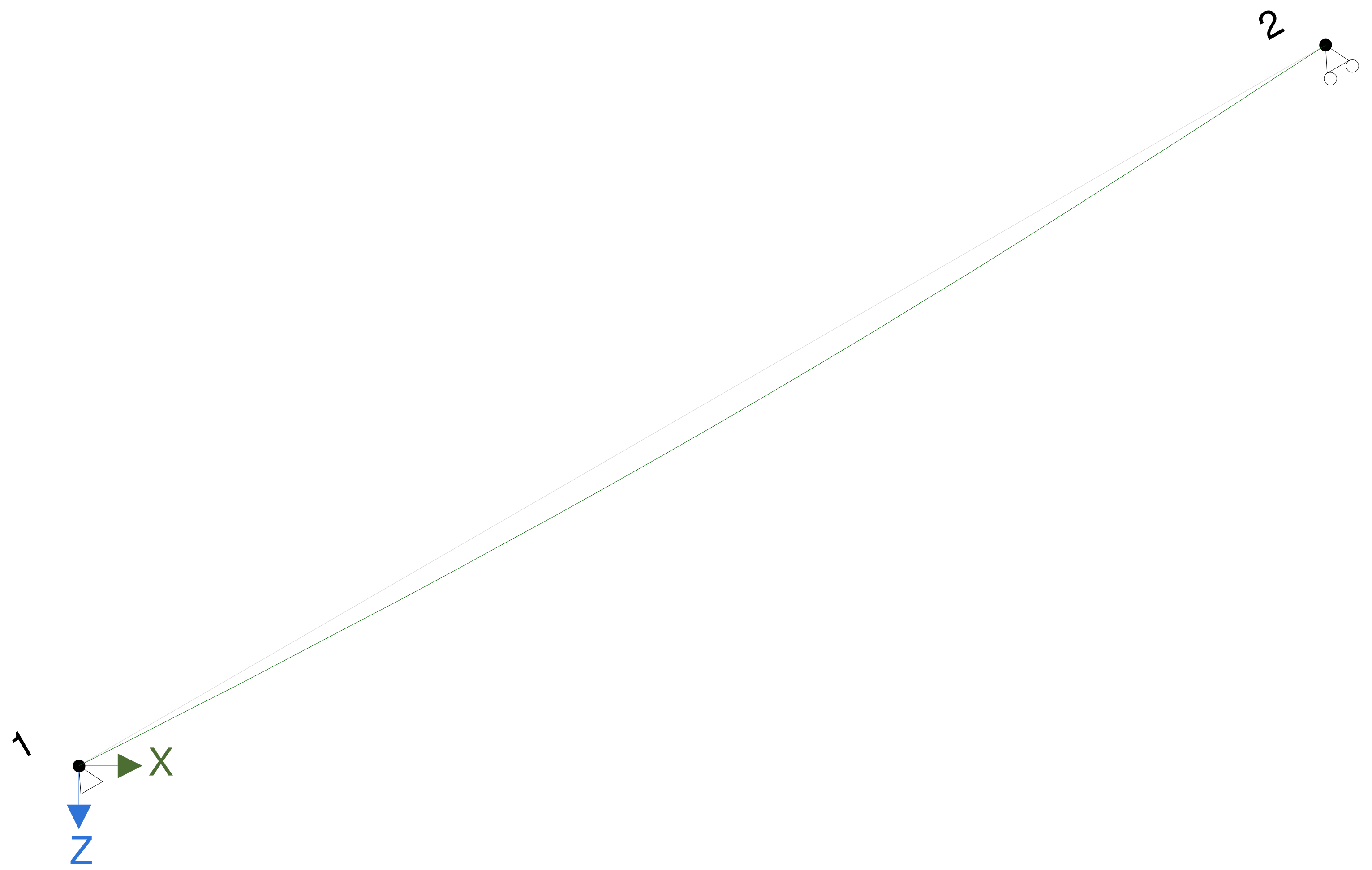
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.38767	Member
2	0	0	-0.38767	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.28716	Member
2	0	0	-0.28716	Member

Total base reactions

Load case/combination	Force FX (kN)	Force FZ (kN)
1.35G + 1.50Q (Strength)	0	2.6
1.00G + 1.00Q (Service)	0	1.9

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

	Project Moss Farm Cottages				Job no.	
	Calcs for Rafters				Start page no./Revision 4	
	Calcs by MC	Calcs date 29/03/2023	Checked by	Checked date	Approved by	Approved date

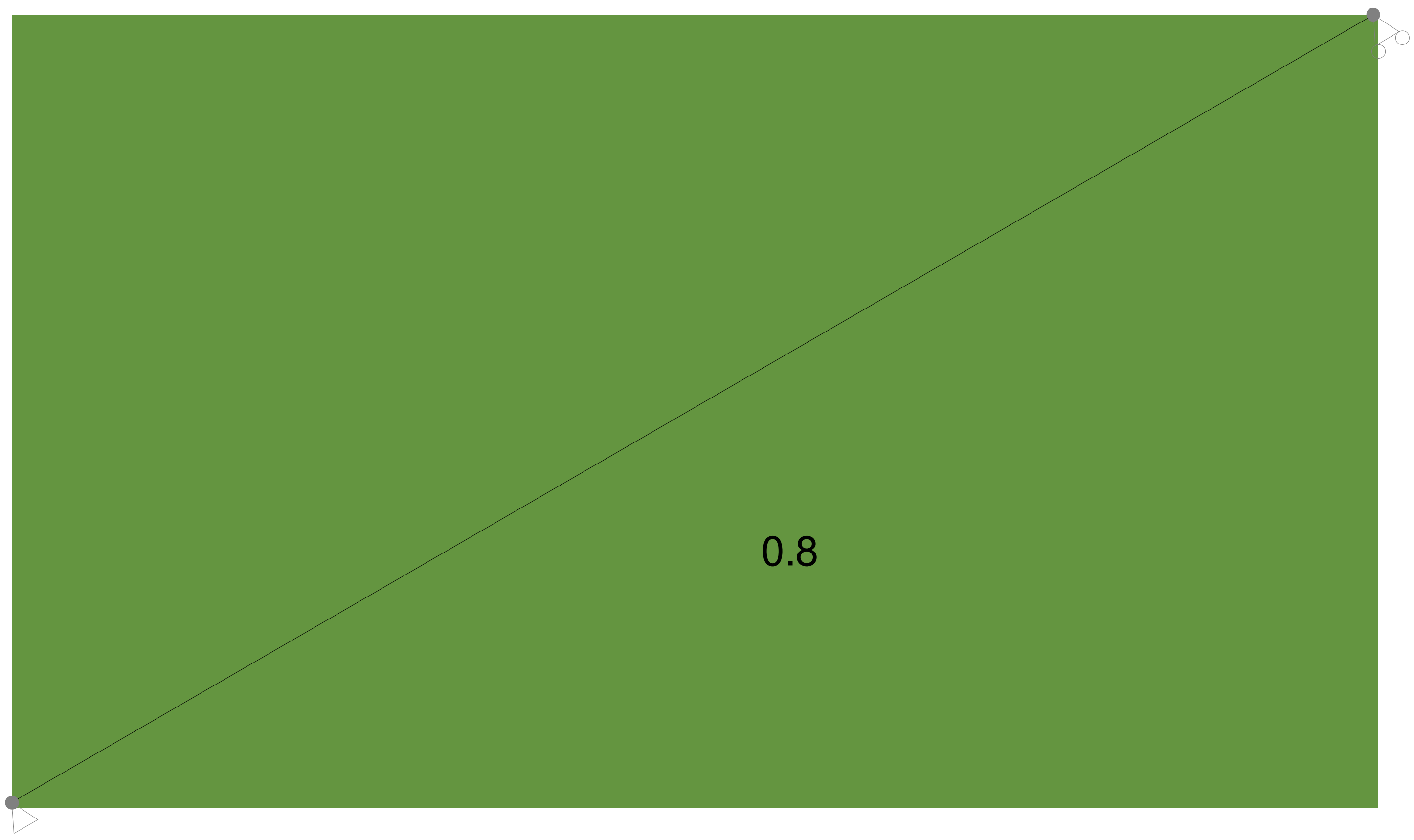
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.783	1	-1.3	-1.1	0
		2	0	-1.1	0

Load combination: 1.00G + 1.00Q (Service)

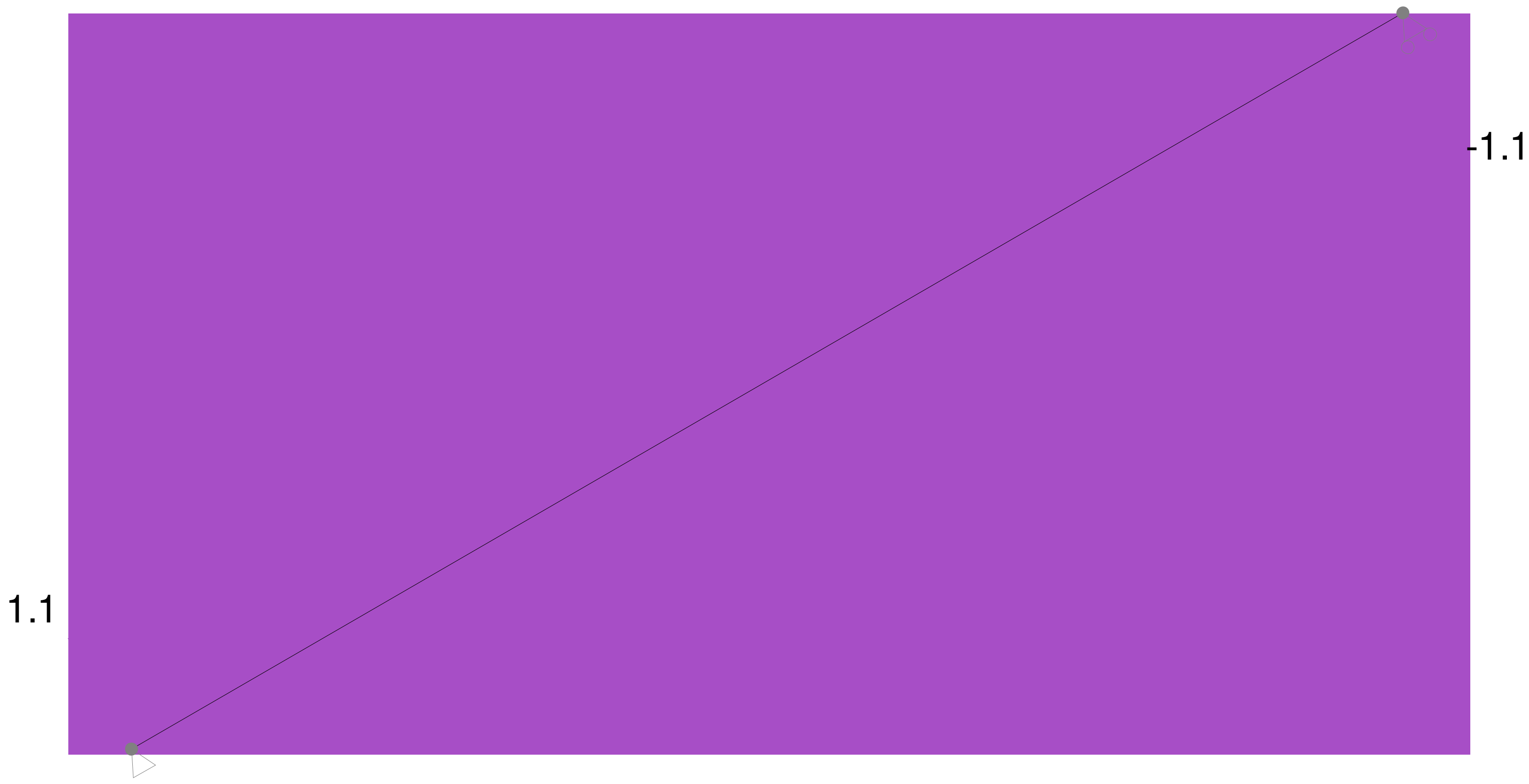
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.783	1	-0.9	-0.8	0
		2	0	-0.8	0

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	1.1 (max abs)		0 (min)	
	1.392	0		0.8 (max)	
	2.783	-1.1		0 (min)	

Tedds calculation version 2.2.07

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

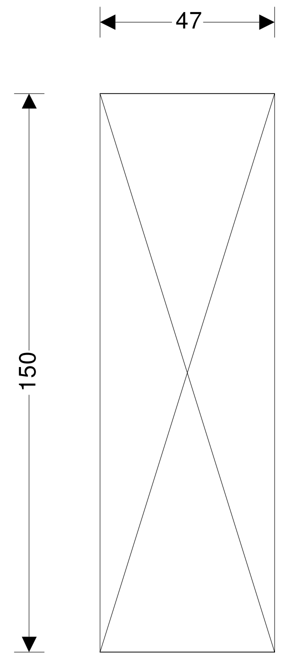
Timber section details

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

Breadth of sections b = **47** mm

Depth of sections h = **150** mm

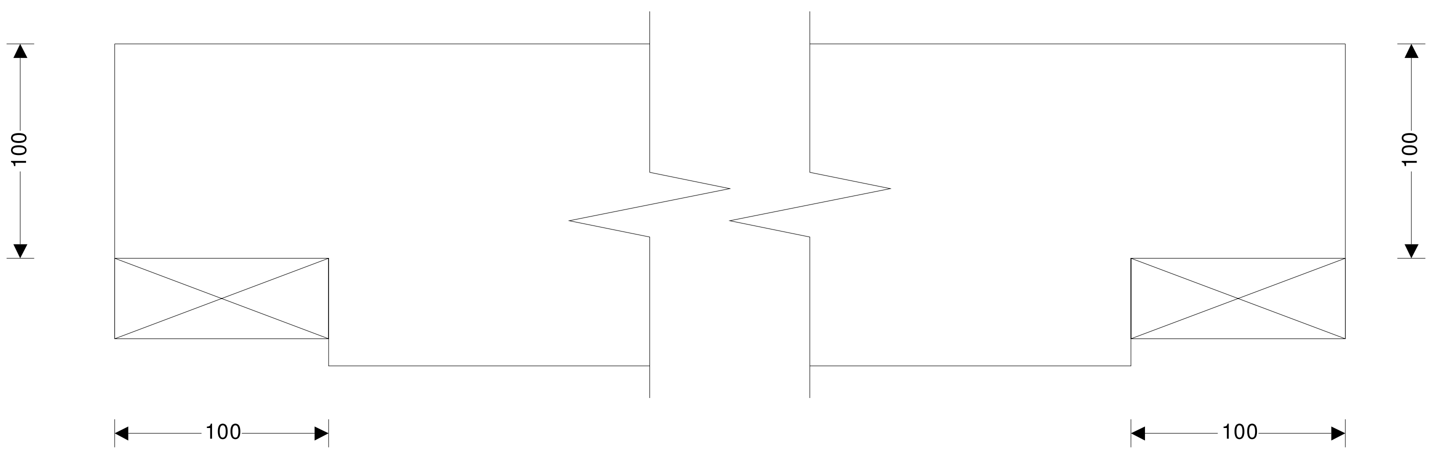
Timber strength class - EN 338:2016 Table 1 **C16**



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

Span details

Bearing length L_b = **100** mm



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

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.8**
Deformation factor - Table 3.2 k_{def} = **0.8**
Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.67**
System strength factor - cl.6.6 k_{sys} = **1.1**
Notch reduction factor - exp.6.61 k_v = **1**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **1.28** kN
Design compressive stress $\sigma_{c,0,d} = P_d / A =$ **0.182** N/mm^2
Design compressive strength $f_{c,0,d} = k_{mod} * k_{sys} * f_{c.0.k} / \gamma_M =$ **11.508** N/mm^2
$\sigma_{c,0,d} / f_{c,0,d} =$ **0.016**

PASS - Design parallel compression strength exceeds design parallel compression stress

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

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **1.109** kN
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 * F_{y,d} / (k_{cr} * b * h) =$ **0.352** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} * k_{sys} * f_{v.k} / \gamma_M =$ **2.166** N/mm²

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

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 $L_{e,y} = 0.9 * 2783$ mm = **2505** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y =$ **57.844**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi * \sqrt{(f_{c.0.k} / E_{0.05})} =$ **1.033**

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 $\lambda_{rel,z} = \lambda_z / \pi * \sqrt{(f_{c.0.k} / E_{0.05})} =$ **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 $k_y = 0.5 * (1 + \beta_c * (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **1.107**

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

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.665**

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

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} * f_{c,0,d}) =$ **0.024**

$\sigma_{c,0,d} / (k_{c,z} * f_{c,0,d}) =$ **0.015**

PASS - Column stability is acceptable

<u>**Check design 1392 mm along span**</u>

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} =$ **0.771** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y =$ **4.376** N/mm²

Design bending strength $f_{m,y,d} = k_{mod} * k_{sys} * f_{m.k} / \gamma_M =$ **10.831** N/mm²

$\sigma_{m,y,d} / f_{m,y,d} =$ **0.404**

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 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} =$ **0.404**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m * \sigma_{m,y,d} / f_{m,y,d} =$ **0.283**

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 $L_{e,y} = 0.9 * 2783$ mm = **2505** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y =$ **57.844**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi * \sqrt{(f_{c.0.k} / E_{0.05})} =$ **1.033**

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 $\lambda_{rel,z} = \lambda_z / \pi * \sqrt{(f_{c.0.k} / E_{0.05})} =$ **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 $k_y = 0.5 * (1 + \beta_c * (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **1.107**

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

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.665**

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

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} * f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} =$ **0.428**

$\sigma_{c,0,d} / (k_{c,z} * f_{c,0,d}) + k_m * \sigma_{m,y,d} / f_{m,y,d} =$ **0.298**

PASS - Column stability is acceptable

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

Check design 1392 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y =$ **4.6** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250 =$ **11.1** mm

$\delta_y / \delta_{y,Allowable} =$ **0.409**

PASS - Allowable deflection exceeds final deflection