2 Mansion House, West Boldon

Structural Calculations



	2 Mansion House	26/04/21
	Timber Roof Remedial Works	KM

Introduction

The purpose of this calculation report is to provide structural analysis and design checks to the timber roof & truss system at 2 Mansion House, West Boldon, in its existing and proposed states.

Existing State

Structural checks performed on roofing members in accordance with British Standard and / or Eurocodes show that several members are currently failing.

Refer to **Appendix A** for full analysis and design of roof truss. It is apparent that the truss system is poorly framed out in such a way that typical members do not frame out, thus not performing as intended like a truss.

Refer to **Appendix B** for full analysis and design of remaining roof members including floor joists, rafters, lower & upper level purlins and hip rafters.

Proposed State (Remedial Works)

The proposed remedial works are summarised below and in Figure 1;

- Introducing a series of 100mm x 150mm C24 timber posts to break up the span of lower level purlins, all supported of the ground floor slab

- Introducing a 100mm x 3000mm C24 central king post to existing truss essentially de-trussing the current truss and changing the load paths of typical members. Again, post is supported off the ground floor slab

- Strengthening existing hip rafter by means of a steel angle section (UKA 150x75x10.0 S275) which is coach bolted through existing 75mm x 200mm timber section, see Figure 2

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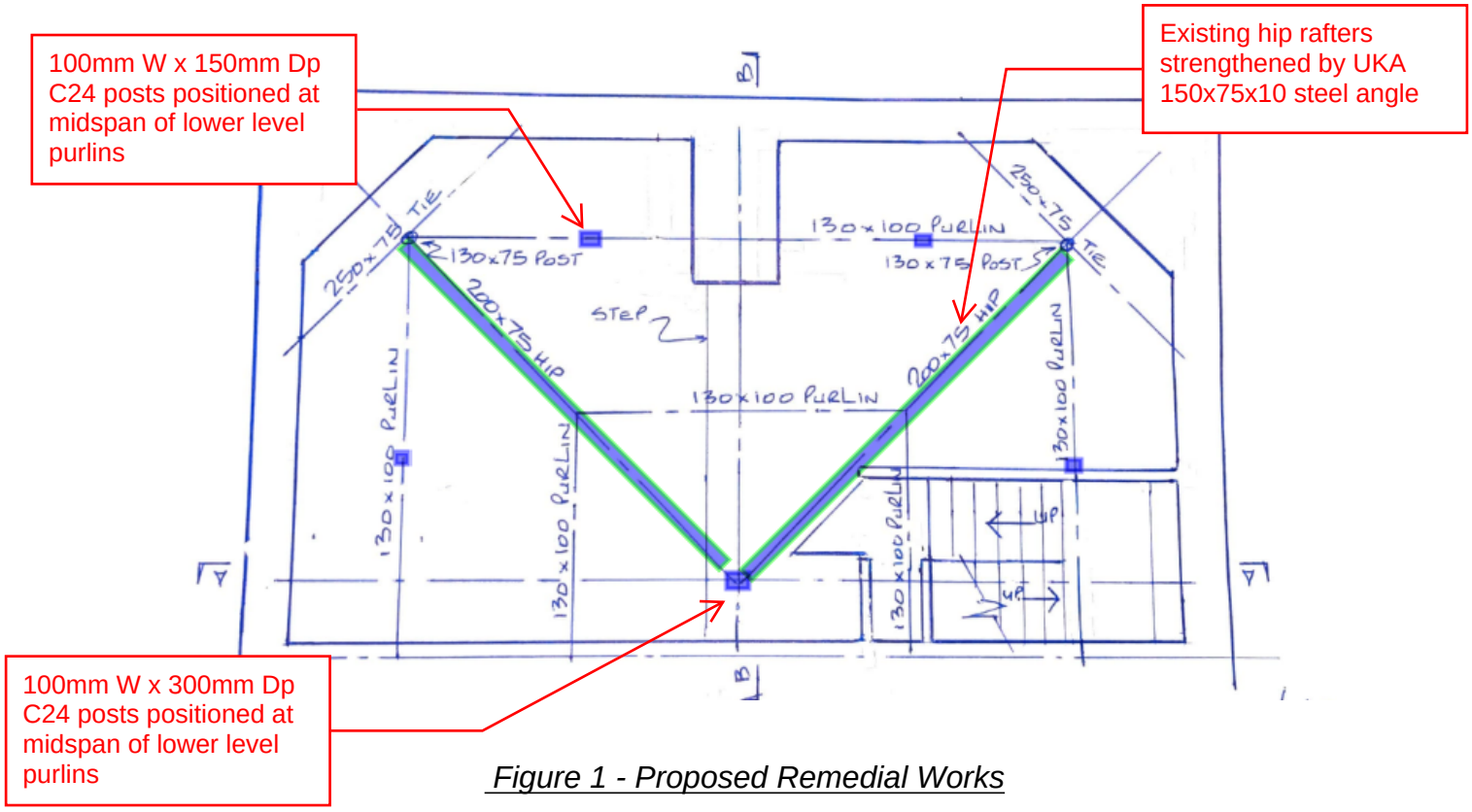

Figure 1 - Proposed Remedial Works

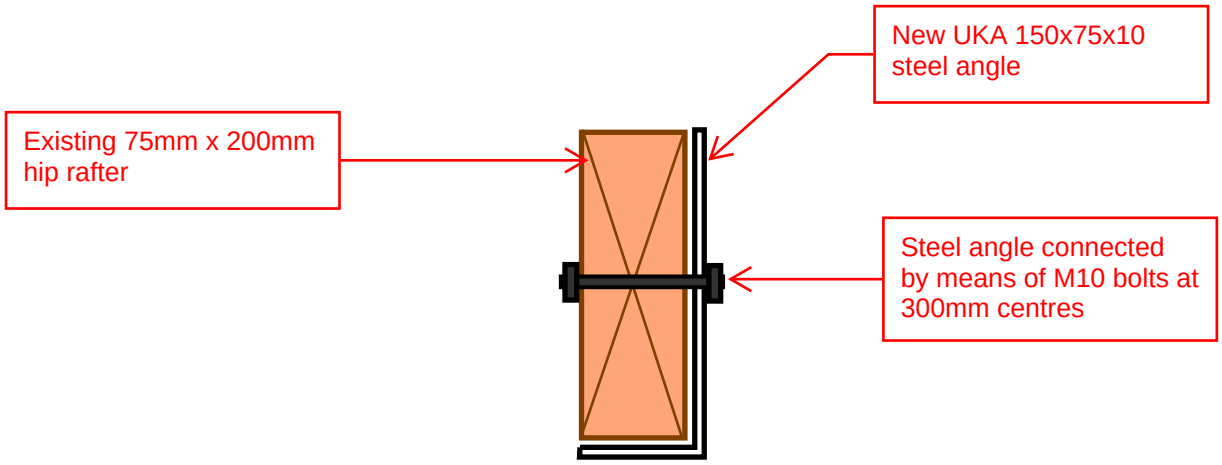

Figure 2 - Hip Rafter Strengthening

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APPENDIX A

EXISTING TRUSS ANALYSIS & DESIGN CHECKS

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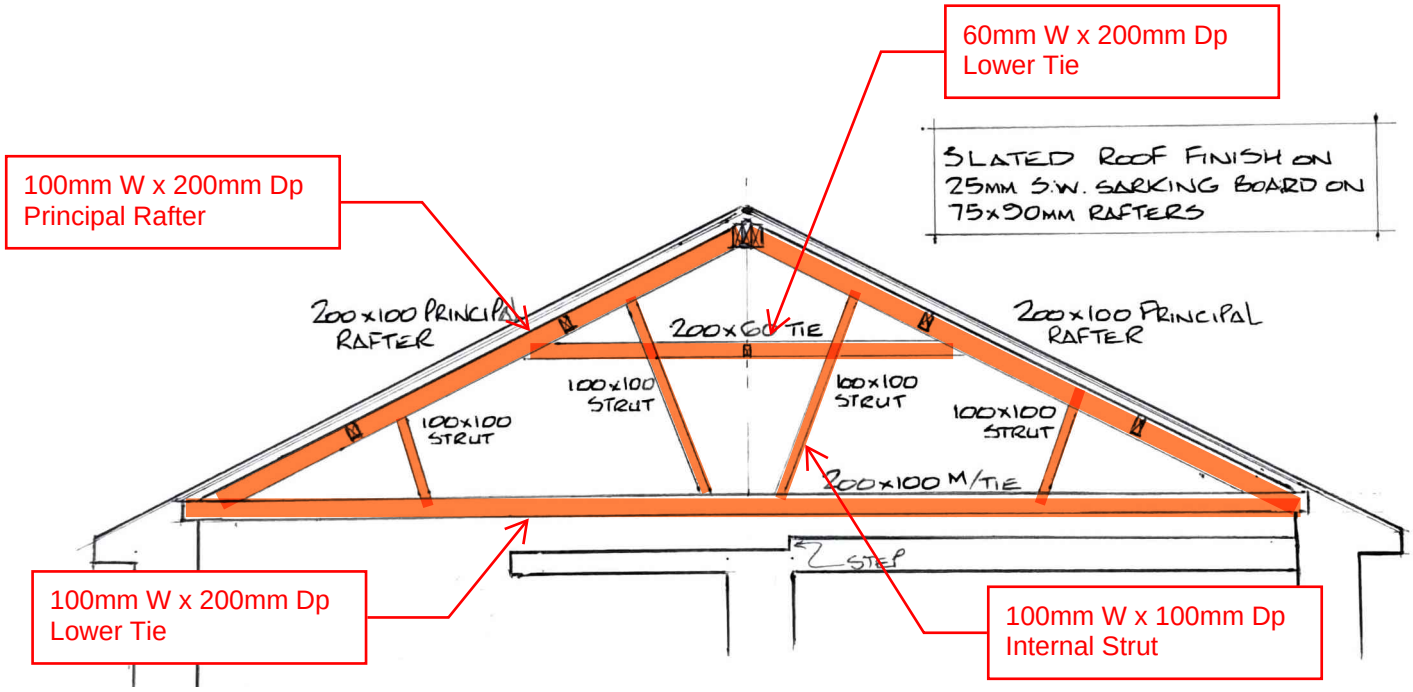
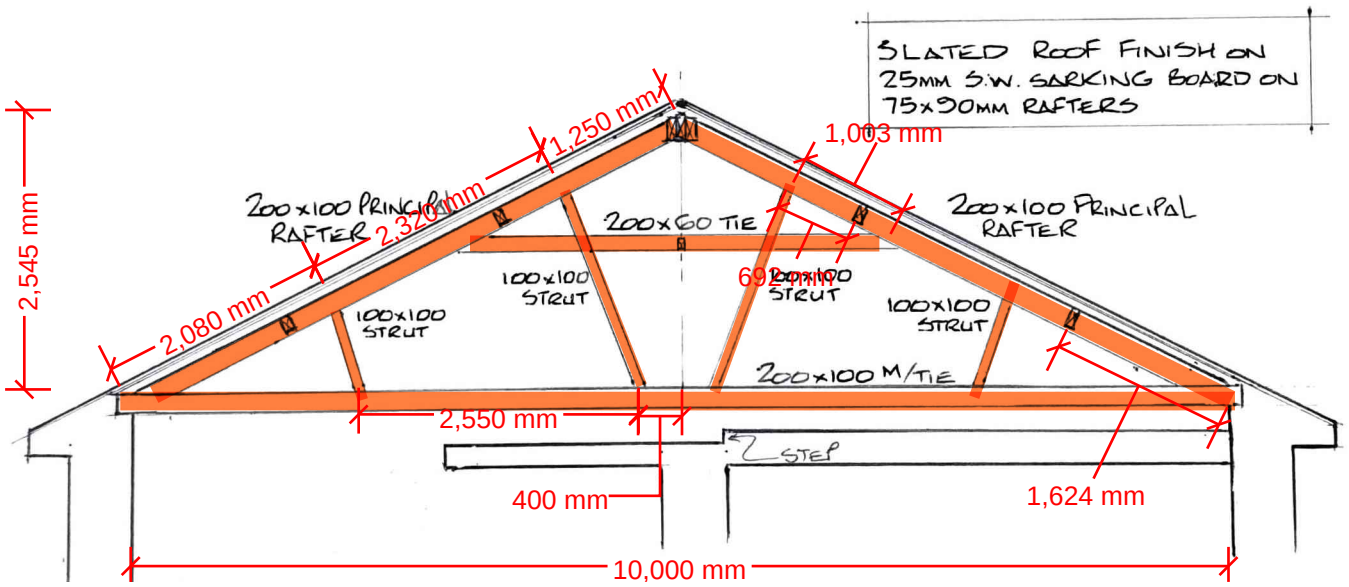
	Timber Roof Truss Analysis & Member Checks	KM



Existing Roof Truss



Existing Roof Truss

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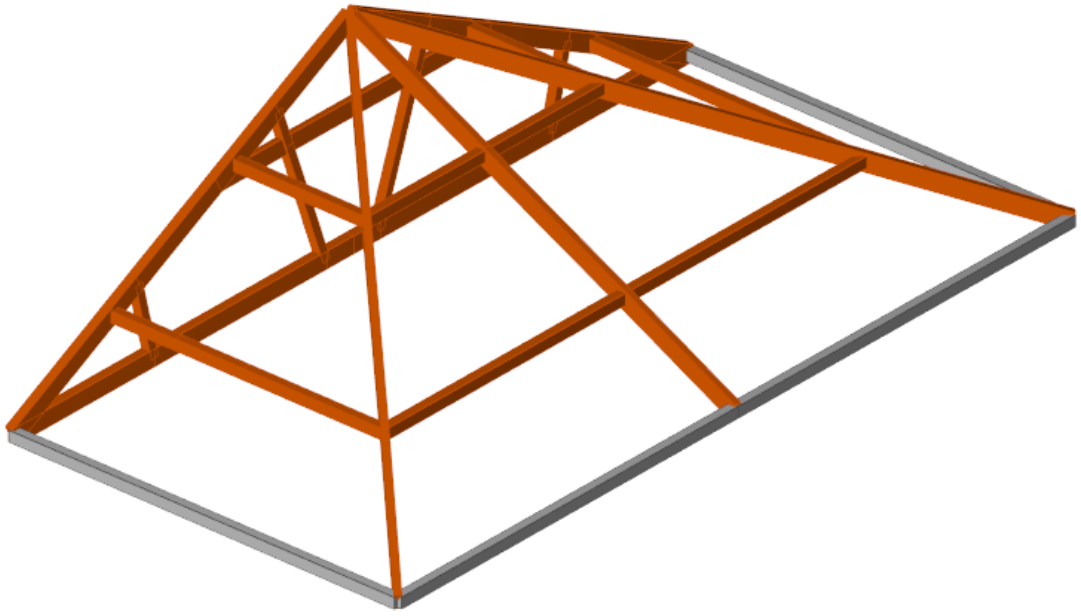
	Timber Roof Truss Analysis & Member Checks	KM

Loading

Truss self-weight	As calculated
Roof dead load	1.65 kN/m^2
Roof Imposed load	0.75 kN/m^2
Wind Load	0.80 kN/m^2

Roof Hip Mode



Structural Model

Both short-term loading and instantaneous loading durations have been assessed in accordance to BS EN 1995. It was found that short-term duration checks resulted in more onerous results, therefore instantaneous checks have been omitted from the subsequent checks.

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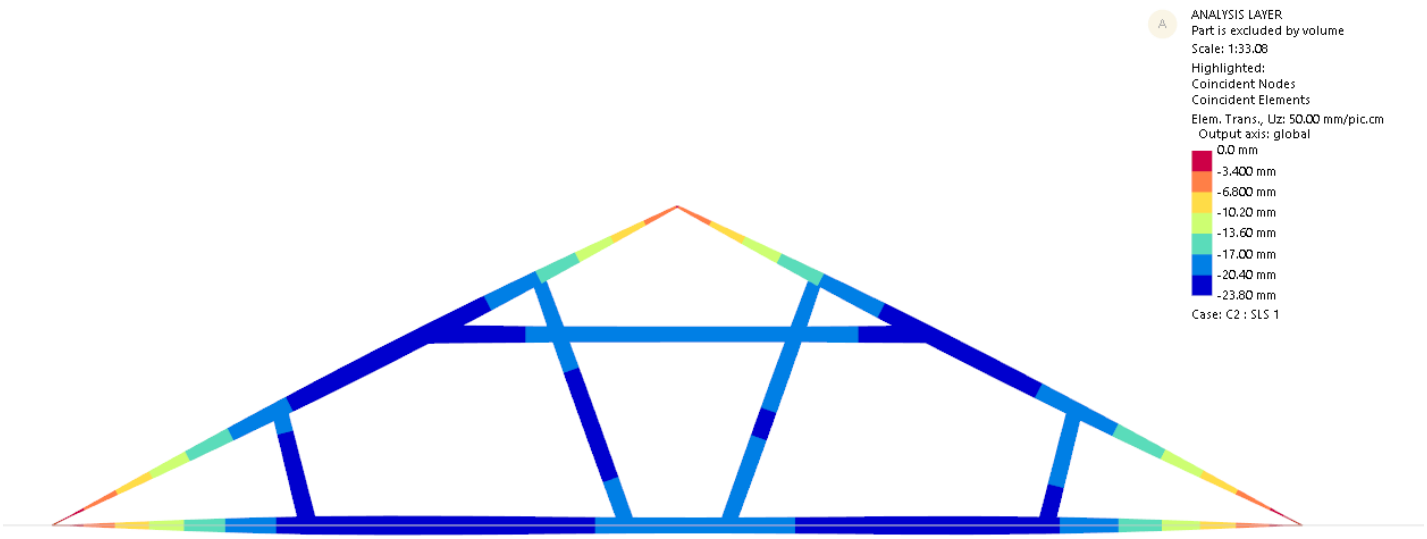
	Timber Roof Truss Analysis & Member Checks	KM

Analysis Results

SLS Deflection



Analysis Results - SLS Deflections

Allowable deflection = span / 250 = 40mm

Maximum deflection = 23.8mm, therefore OK

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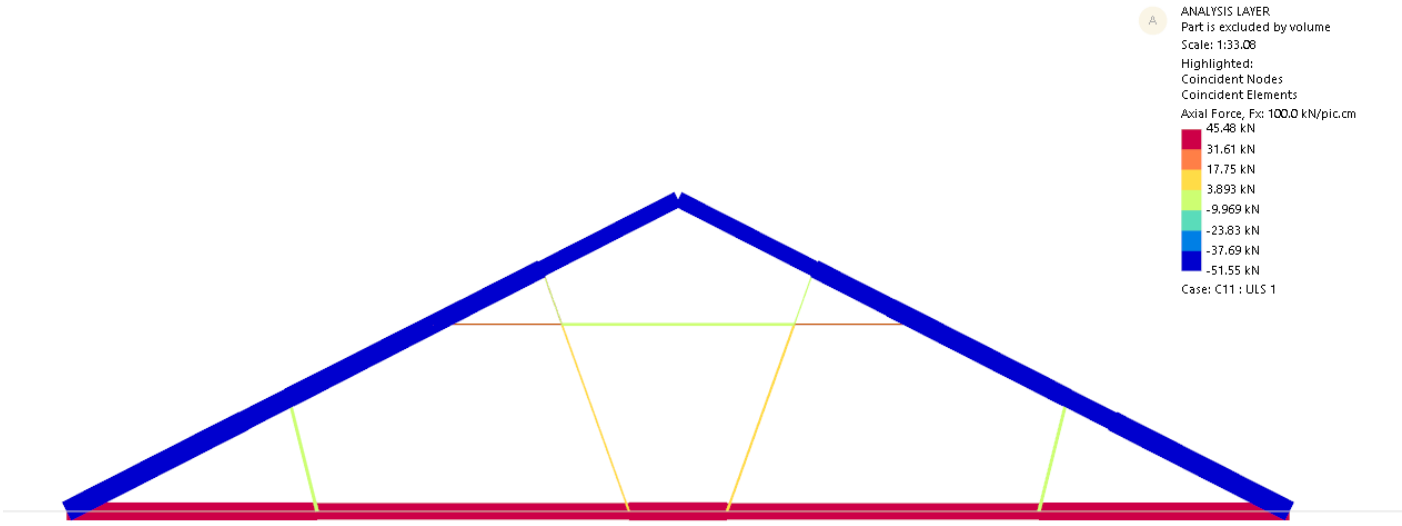
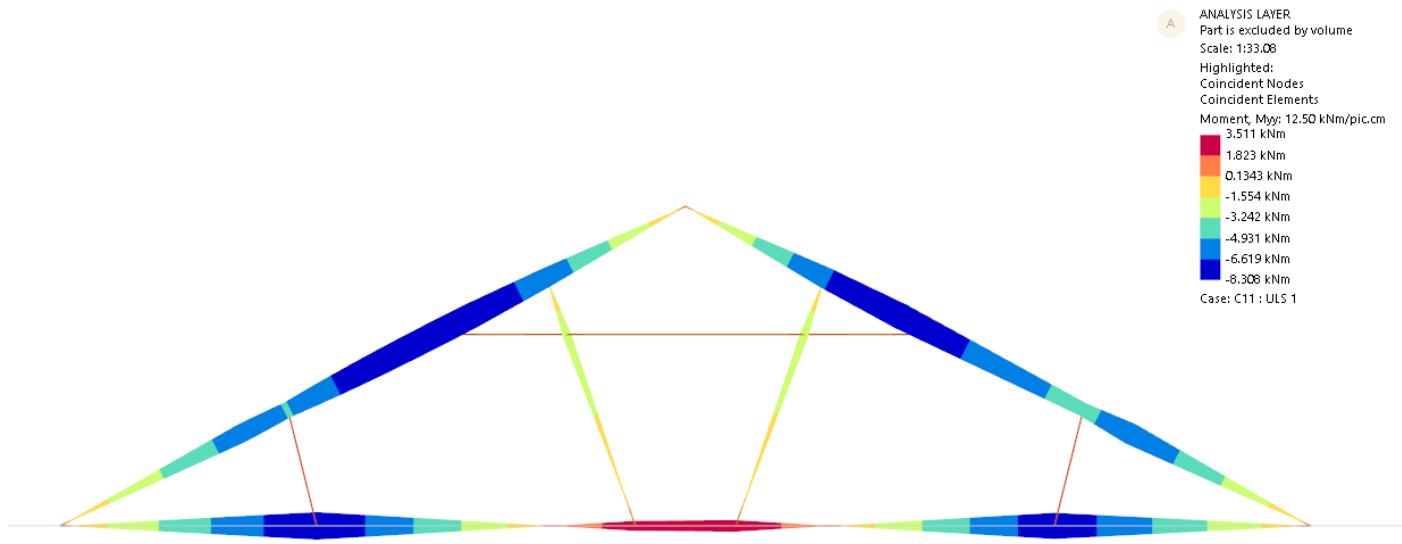
Analysis Results

ULS Results



Analysis Results - Axial Force (kN)



Analysis Results -Bending Moment (kNm)

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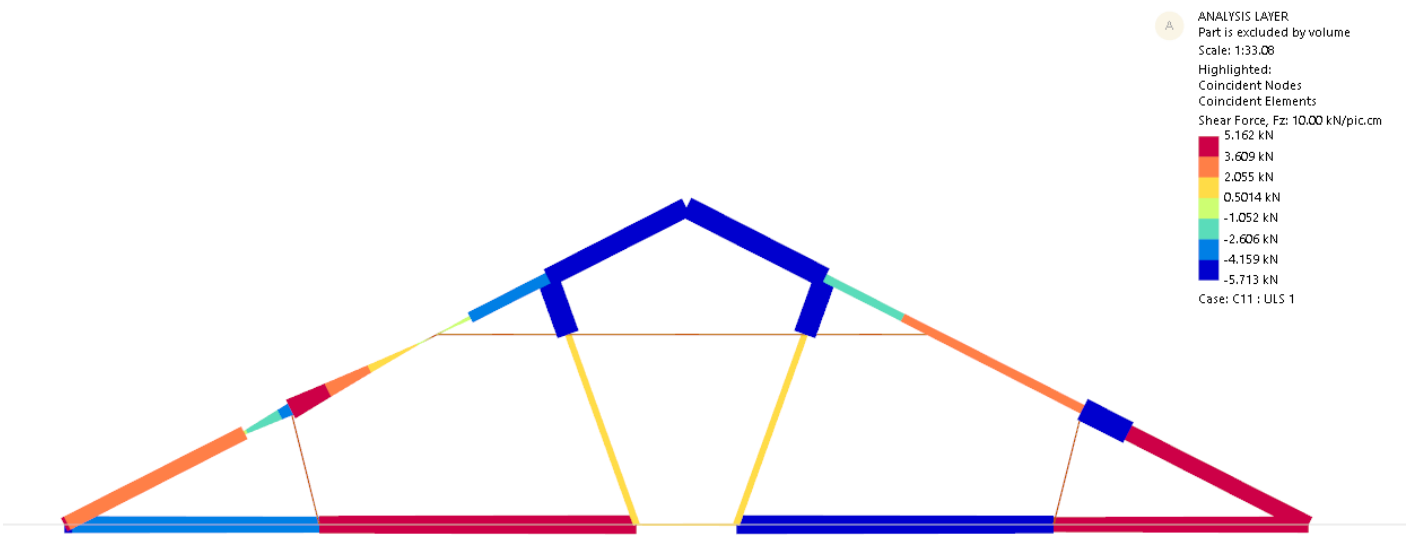
	Timber Roof Truss Analysis & Member Checks	KM

Analysis Results

ULS Results



Analysis Results - Shear (kN)

Typical Member Checks

Typical Member Design Results

Member	Effective Lengths		Design Forces			Pass / Fail
	$Le_{,major}$ (m)	$Le_{,minor}$ (m)	Axial (kN)	Bending Moment (kN/m)	Shear (kN)	
100mm x 200mm Rafter	5.65	2.32	51.55	8.35	5.75	Fail
100mm x 200mm Lower Tie	10.00	10.00	-45.50	8.35	4.65	Fail
100mm x 100mm Strut	2.15	2.15	6.25	2.70	5.55	Fail
60mm x 200mm Upper Tie	4.05	4.05	-7.55	0.1	0.25	Pass

Summary Table

Tekla Tedds	Project: 2 Mansion House, West Boldon				Job no.	
	Calcs for: 100mm x 200mm Lower Tie				Start page no./Revision: 1	
	Calcs by	Calcs date: 10/03/2021	Checked by	Checked date	Approved by	Approved date

100MM WIDE X 200MM DEEP LOWER TIE

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

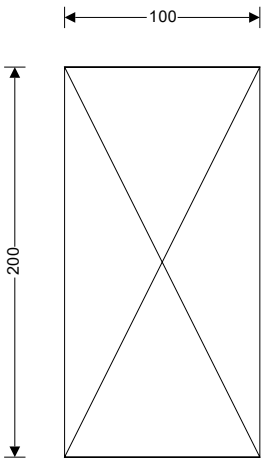
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **200** mm

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



100x200 timber section

Cross-sectional area, A, 20000 mm^2

Section modulus, W_y, 666666.7 mm^3

Section modulus, W_z, 333333 mm^3

Second moment of area, I_y, 66666667 mm^4

Second moment of area, I_z, 16666667 mm^4

Radius of gyration, i_y, 57.7 mm

Radius of gyration, i_z, 28.9 mm

Timber strength class C24

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

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

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

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

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

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

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

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

Characteristic density, ρ_k, 350 kg/m^3

Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Minor axis L_z = **10000** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **10000** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **8.35** kNm

Design shear force - Major axis $F_{y,d}$ = **4.65** kN

Design perpendicular compression - Major axis $F_{c,y,90,d}$ = **4.65** kN

Design axial tension force P_d = **45.5** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Tensile stress	N/mm^2	6.5	2.3	0.349	PASS
Bearing stress	N/mm^2	1.1	0.5	0.413	PASS
Bending stress	N/mm^2	10.8	12.5	1.160	FAIL
Shear stress	N/mm^2	1.8	0.5	0.289	PASS
Bending and axial force				1.508	FAIL

Beam stability check				1.324	FAIL

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.9**

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

Depth factor for tension - exp.3.1 $k_{h,t}$ = **1**

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**

Check tension parallel to the grain - Section 6.1.2

Axial tension P_d = **45.5** kN

Design tensile stress $\sigma_{t,0,d} = P_d / A$ = **2.275** N/mm^2

Design tensile strength $f_{t,0,d} = k_{h,t} \times k_{mod} \times f_{t.0.k} / \gamma_M$ = **6.525** N/mm^2

$\sigma_{t,0,d} / f_{t,0,d}$ = **0.349**

PASS - Design tensile strength exceeds design tensile stress

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **4.65** kN

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

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.465** N/mm^2

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times f_{c.90.k} / \gamma_M$ = **1.125** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.413**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **4.65** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.521** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.289**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **8.35** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **12.525** N/mm^2

Design bending strength $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **1.16**

FAIL - Design bending stress exceeds design bending strength

Check combined bending and axial tension - Section 6.2.3

Combined loading checks - exp.6.17 & 6.18 $\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d}$ = **1.508**

$\sigma_{t,0,d} / f_{t,0,d} + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **1.160**

FAIL - Combined bending and axial tension utilisation is not acceptable

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

Effective length - Table 6.1 L_{ef} = 10000 mm = **10000** mm

Critical bending stress - exp.6.32 $\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **28.860** N/mm^2

Relative slenderness for bending - exp.6.30 $\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.912**

Lateral buckling factor - exp.6.34 $k_{crit} = 1.56 - 0.75 \times \lambda_{rel,m}$ = **0.876**

Beam stability check - exp.6.33 $\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d})$ = **1.324**

FAIL - Beam stability is not acceptable

100MM WIDE X 200MM DEEP PRINCIPAL RAFTER

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

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Service class - cl.2.3.1.3 1

Timber section details

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

Breadth of sections b = **100** mm

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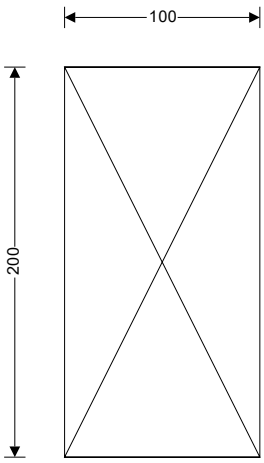
Timber strength class - EN 338:2016 Table 1 **C24**



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Radius of gyration, i_z, 28.9 mm

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

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

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

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

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

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

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

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

Characteristic density, ρ_k, 350 kg/m^3

Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Major axis L_y = **5650** mm

Effective length - Major axis $L_{e,y}$ = L_y = **5650** mm

Unbraced length - Minor axis L_z = **2320** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **2320** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **8.35** kNm

Design shear force - Major axis $F_{y,d}$ = **5.75** kN

Design perpendicular compression - Major axis $F_{c,y,90,d}$ = **5.75** kN

Design axial compression force P_d = **51.55** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm^2	9.5	2.6	0.273	PASS
Bearing stress	N/mm^2	1.1	0.6	0.511	PASS
Bending stress	N/mm^2	10.8	12.5	1.160	FAIL

Shear stress	N/mm²	1.8	0.6	0.358	PASS
Bending and axial force				1.234	FAIL
Column stability check				2.019	FAIL
Beam stability check				1.956	FAIL

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**

Load configuration factor - cl.6.1.5(4) $k_{c,90}$ = **1**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **51.55** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **2.577** N/mm²

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **9.450** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.273**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **5.75** kN

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

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.575** N/mm²

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times f_{c.90.k} / \gamma_M$ = **1.125** N/mm²

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.511**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **5.75** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.644** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.358**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **8.35** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **12.525** N/mm²

Design bending strength $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **1.16**

FAIL - Design bending stress exceeds design bending strength

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **1.234**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.886**

FAIL - Combined bending and axial compression utilisation is not acceptable

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

Effective length for y-axis bending $L_{e,y}$ = **5650** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **97.861**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.659**

Effective length for z-axis bending $L_{e,z}$ = **2320** mm

Slenderness ratio	$\lambda_z = L_{e,z} / i_z =$ **80.367**
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **1.363**
	Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is required
Straightness factor	$\beta_c =$ **0.2**
Instability factors - exp.6.25, 6.26, 6.27 & 6.28	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **2.013**
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **1.535**
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.317**
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) =$ **0.446**
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} =$ **2.019**
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} =$ **1.423**
	FAIL - Column stability is not acceptable

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

Effective length - Table 6.1	$L_{ef} = 2320$ mm = **2320** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef}) =$ **124.397** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})} =$ **0.439**
Lateral buckling factor - exp.6.34	$k_{crit} =$ **1.000**
Beam stability check - exp.6.35	$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) =$ **1.956**
	FAIL - Beam stability is not acceptable

100MM WIDE X 100MM INTERNAL STRUT

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

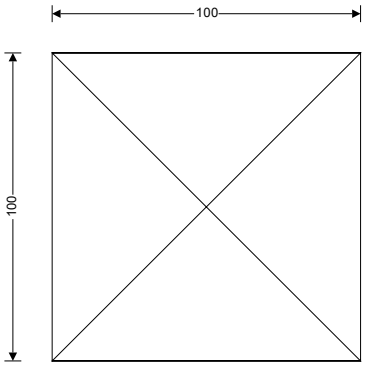
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **100** mm

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



100x100 timber section
Cross-sectional area, A, 10000 mm²
Section modulus, W_y, 166666.7 mm³
Section modulus, W_z, 166667 mm³
Second moment of area, I_y, 8333333 mm⁴
Second moment of area, I_z, 8333333 mm⁴
Radius of gyration, i_y, 28.9 mm
Radius of gyration, i_z, 28.9 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

Unbraced length - Major axis L_y = **2150** mm

Effective length - Major axis $L_{e,y} = L_y$ = **2150** mm

Unbraced length - Minor axis L_z = **2150** mm

Effective length - Minor axis $L_{e,z} = L_z$ = **2150** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **2.7** kNm

Design shear force - Major axis $F_{y,d}$ = **5.55** kN

Design axial compression force P_d = **6.25** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm²	9.5	0.6	0.066	PASS
Bending stress	N/mm²	11.7	16.2	1.383	FAIL
Shear stress	N/mm²	1.8	1.2	0.690	PASS
Bending and axial force				1.388	FAIL
Column stability check				1.514	FAIL
Beam stability check				2.044	FAIL

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.9**

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

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**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **6.25** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.625** N/mm²

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **9.450** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.066**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **5.55** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **1.243** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.690**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **2.7** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **16.2** N/mm²

Design bending strength $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times f_{m.k} / \gamma_M$ = **11.712** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **1.383**

FAIL - Design bending stress exceeds design bending strength

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **1.388**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.973**

FAIL - Combined bending and axial compression utilisation is not acceptable

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

Effective length for y-axis bending $L_{e,y}$ = **2150** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **74.478**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.263**

Effective length for z-axis bending $L_{e,z}$ = **2150** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **74.478**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.263**

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

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.394**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **1.394**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.504**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **0.504**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **1.514**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **1.099**

FAIL - Column stability is not acceptable

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

Effective length - Table 6.1	L_{ef} = 2150 mm = **2150** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **268.465** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.299**
Lateral buckling factor - exp.6.34	k_{crit} = **1.000**
Beam stability check - exp.6.35	$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **2.044**

FAIL - Beam stability is not acceptable

Tekla Tedds	Project: 2 Mansion House, West Boldon				Job no.	
	Calcs for: 60mm x 200mm Upper Tie				Start page no./Revision: 1	
	Calcs by	Calcs date: 10/03/2021	Checked by	Checked date	Approved by	Approved date

100MM WIDE X 200MM DEEP LOWER TIE

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

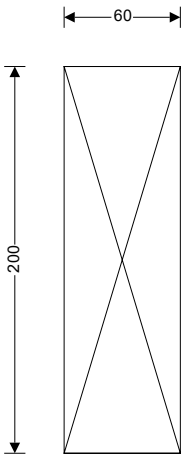
Timber section details

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

Breadth of sections b = **60** mm

Depth of sections h = **200** mm

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



60x200 timber section
Cross-sectional area, A, 12000 mm^2
Section modulus, W_y, 400000 mm^3
Section modulus, W_z, 120000 mm^3
Second moment of area, I_y, 40000000 mm^4
Second moment of area, I_z, 3600000 mm^4
Radius of gyration, i_y, 57.7 mm
Radius of gyration, i_z, 17.3 mm
Timber strength class C24
Characteristic bending strength, $f_{m.k}$, 24 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Minor axis L_z = **4050** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **4050** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **0.1** kNm

Design shear force - Major axis $F_{y,d}$ = **0.25** kN

Design axial tension force P_d = **7.55** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Tensile stress	N/mm^2	6.5	0.6	0.096	PASS
Bending stress	N/mm^2	10.8	0.3	0.023	PASS
Shear stress	N/mm^2	1.8	0.0	0.026	PASS
Bending and axial force				0.120	PASS
Beam stability check				0.028	PASS

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.9**
Deformation factor - Table 3.2	k_{def} = **0.6**
Depth factor for tension - exp.3.1	$k_{h,t}$ = **1**
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**

Check tension parallel to the grain - Section 6.1.2

Axial tension	P_d = **7.55** kN
Design tensile stress	$\sigma_{t,0,d} = P_d / A$ = **0.629** N/mm^2
Design tensile strength	$f_{t,0,d} = k_{h,t} \times k_{mod} \times f_{t.0.k} / \gamma_M$ = **6.525** N/mm^2
	$\sigma_{t,0,d} / f_{t,0,d}$ = **0.096**

PASS - Design tensile strength exceeds design tensile stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **0.25** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.047** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.026**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **0.1** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **0.25** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.023**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial tension - Section 6.2.3

Combined loading checks - exp.6.17 & 6.18	$\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d}$ = **0.120**
	$\sigma_{t,0,d} / f_{t,0,d} + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.113**

PASS - Combined bending and axial tension utilisation is acceptable

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

Effective length - Table 6.1	L_{ef} = 4050 mm = **4050** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **25.653** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.967**
Lateral buckling factor - exp.6.34	$k_{crit} = 1.56 - 0.75 \times \lambda_{rel,m}$ = **0.835**
Beam stability check - exp.6.33	$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d})$ = **0.028**

PASS - Beam stability is acceptable

	2 Mansion House	26/04/21
	Timber Roof Truss Analysis & Member Checks	KM

APPENDIX B

EXISTING MEMBERS ANALYSIS & DESIGN CHECKS

21107	2 MANSION HOUSE - WEST BOLDON ROOF STRUCTURE CHECKS.	8/3/21 IFM

ROOF CONSTRUCTION

SLATED ROOF FINISH ON 25mm S.W. BATTENS BOARD ON 75x90mm RAFTERS SPANNING BETWEEN MAIN TIMBER PURLINS.
TIMBER PURLINS SUPPORTED FROM MAIN TIMBER TRUSS & HIP RAFTERS.
FLOOR IS SUPPORTED INDEPENDENTLY FROM TRUSS WITH JOISTS SPANNING BETWEEN SOLID WALLS.

* 75x90 TIMBER RAFTERS (@600 c/c)

(EXCLUDING RAFTERS) ROOF DL = 0.51 kN/m²/cos30 = 0.59 kN/m² x 0.6 = 0.35 kN/m
IL = 0.75 kN/m² x 0.6m = 0.45 kN/m

FROM TEDDS CALCULATIONS OK.

* 130x100 TIMBER PURLINS (L = 3650mm WORST)

SUPPORTING 1.5m OF ROOF.
ROOF DL = 0.87 kN/m²/cos30 = 1.06 kN/m² x 1.5m = 1.6 kN/m
IL = 0.75 kN/m² x 1.5m = 1.1 kN/m
FROM TEDDS CALCULATIONS, PURLINS AT LOWER LEVEL ARE NOT ADEQUATE BY CALCULATION. PURLINS AT HIGHER LEVEL ARE FOUND ADEQUATE.

* FROM TEDDS CALCULATIONS, FLOOR JOISTS ARE ADEQUATE TO SUPPORT A IL = 2.5 kN/m²

21107	2 MANSION HOUSE ROOF STRUCTURE CHECKS	8/3/21 IFM

* 200 × 75 HIP RAFTERS $L = 5300mm$.

HIP RAFTERS SUPPORT A POINT LOAD FROM PURLINS 2.7m FROM SUPPORT.

$P_{DL} = 1.06 kN/m^2 \times 2m \times 1m = 2.12 kN$

$P_{IL} = 0.75 kN/m^2 \times 2m \times 1m = 1.50 kN$

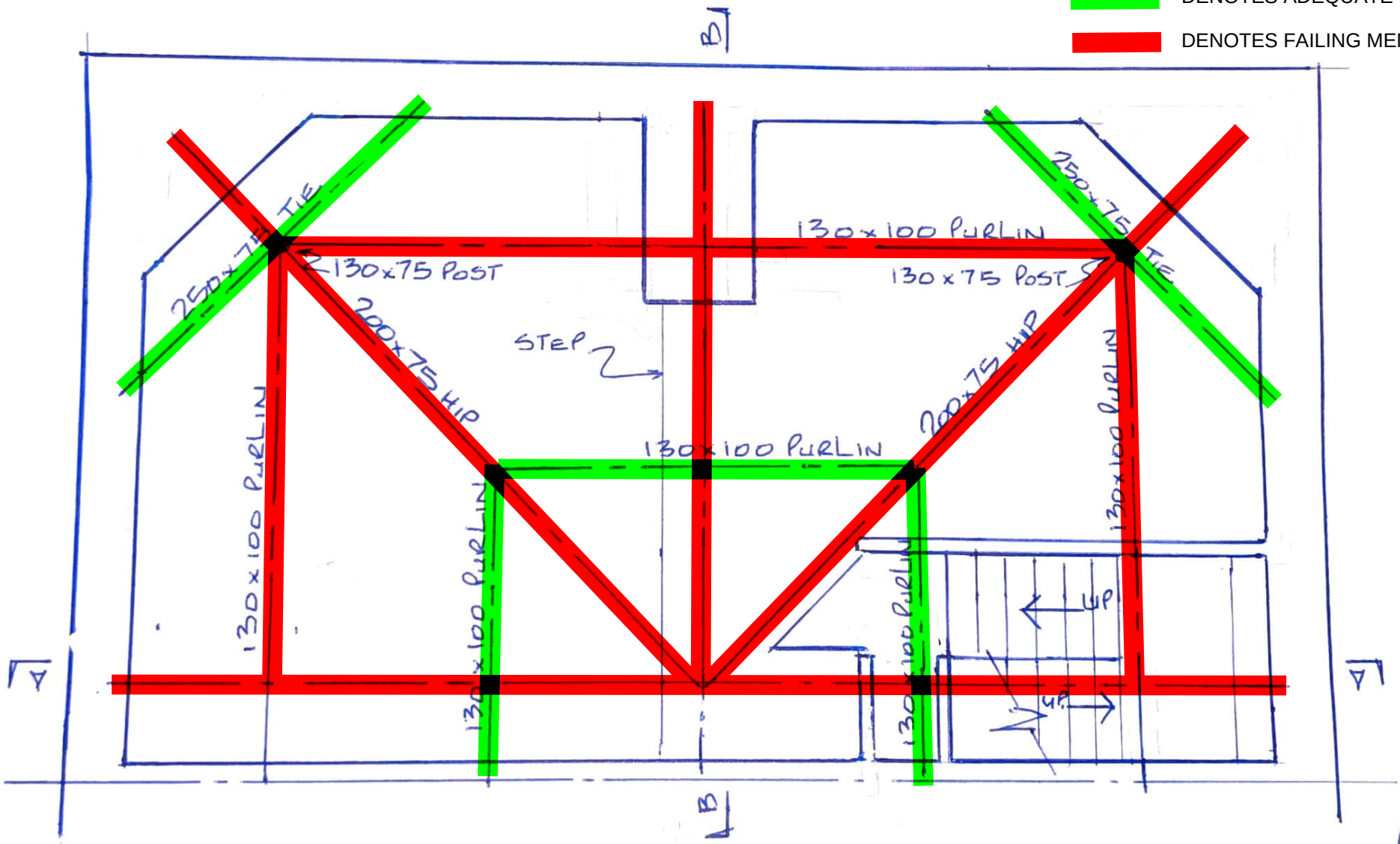
FROM TEDDS CALCULATIONS, HIP RAFTERS ARE NOT FOUND ADEQUATE UNDER CURRENT LOADINGS

2 MANSION HOUSE ~ WEST BOLDON

SECOND FLOOR PLAN

DENOTES ADEQUATE MEMBERS

DENOTES FAILING MEMBERS



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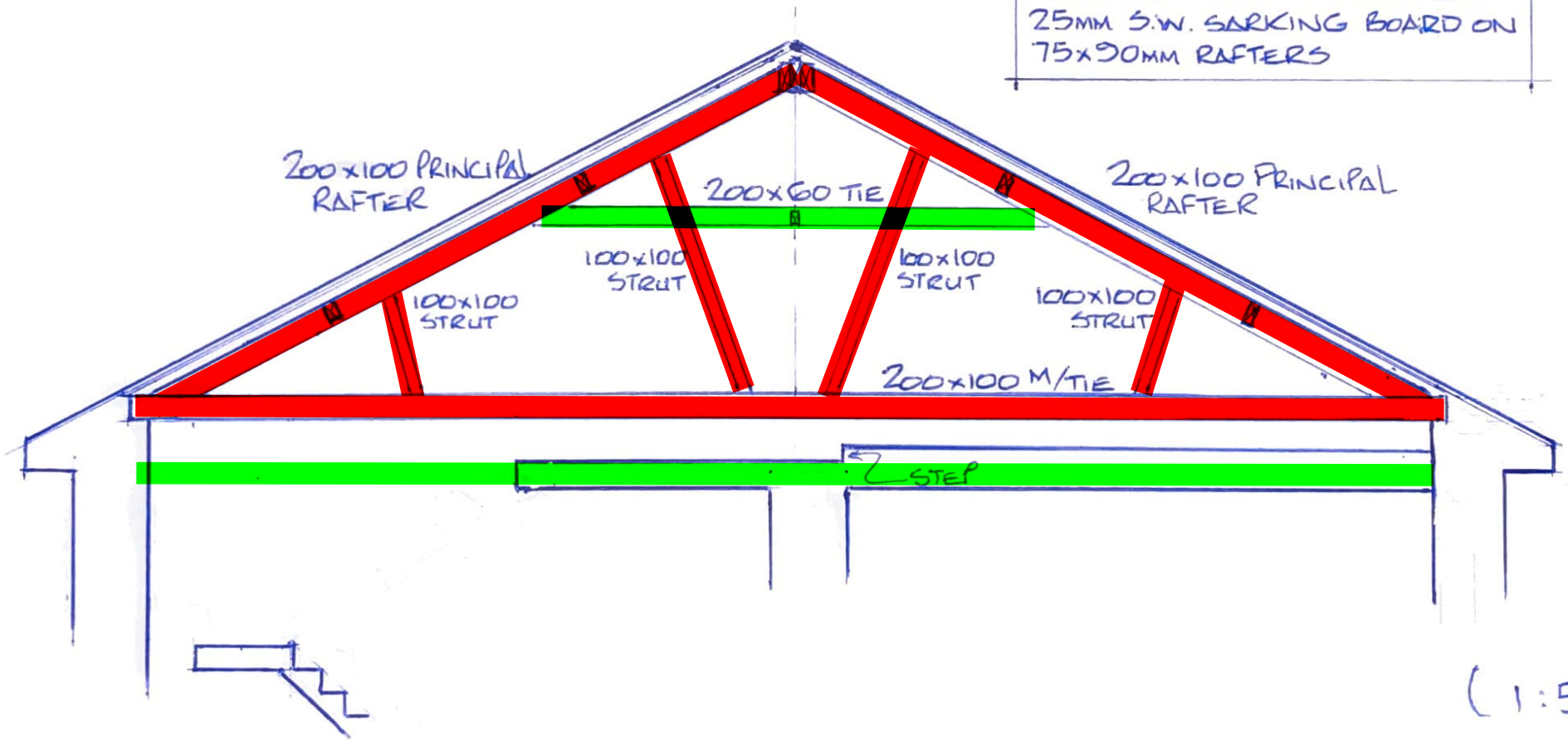
2 MANSION HOUSE ~ WEST BOLDON

SECTION A-A

DENOTES ADEQUATE MEMBERS

DENOTES FAILING MEMBERS

SLATED ROOF FINISH ON
25MM S.W. SARKING BOARD ON
75x90MM RAFTERS



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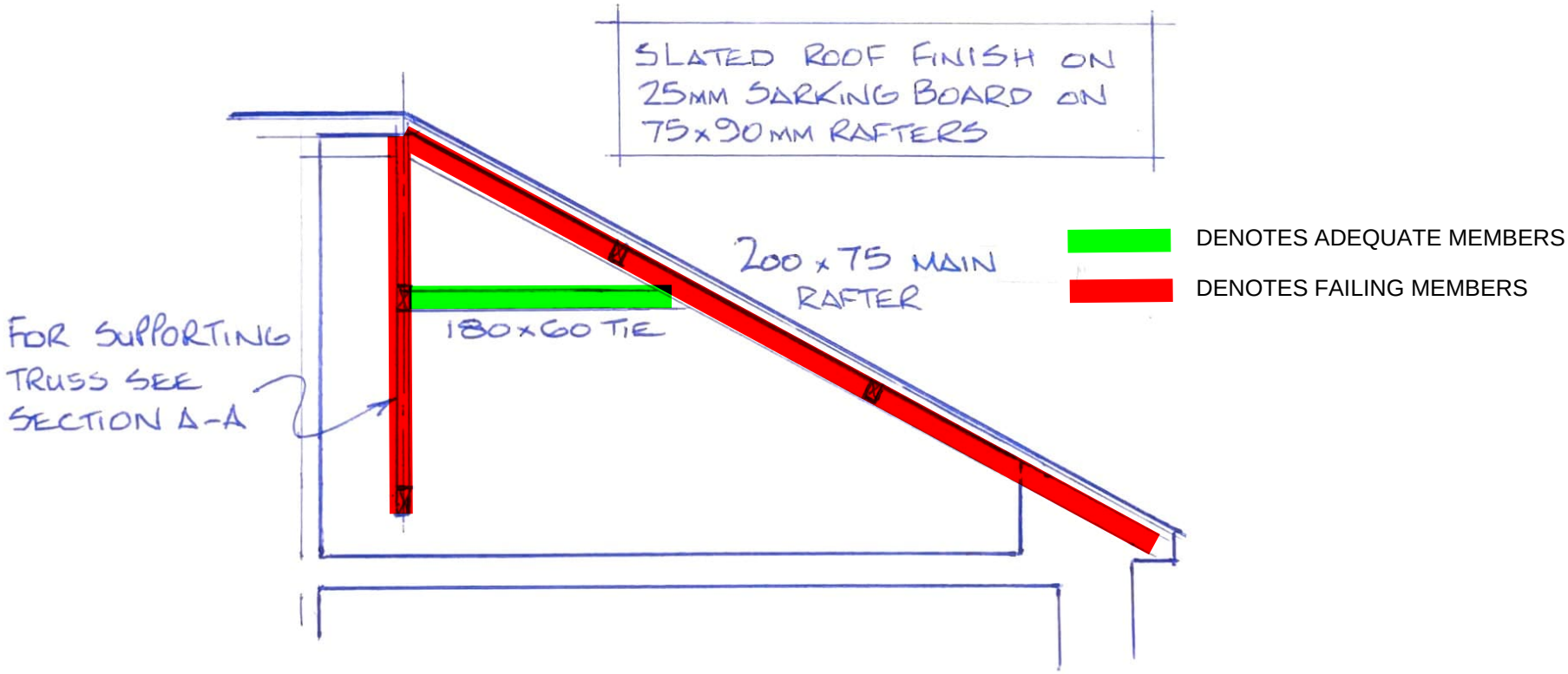
2 MANSION HOUSE ~ WEST BOLDON

SECTION B-B



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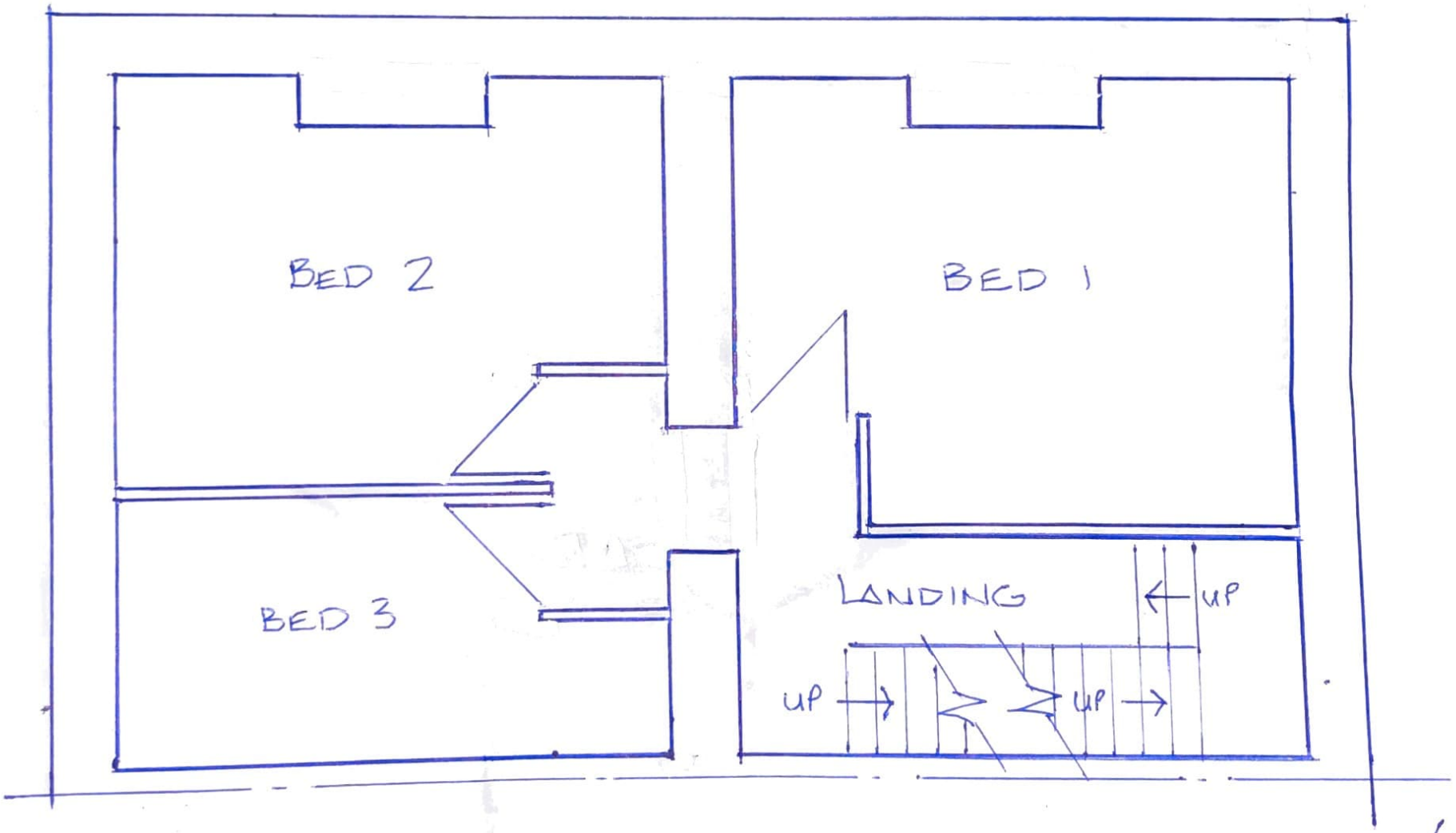
2 MANSION HOUSE ~ WEST BOLDON

FIRST FLOOR PLAN



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Project				Job no.	
2 Mansion House				21107	
Calcs for				Start page no./Revision	
Floor joists check				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
U	08/03/2021	IFM	08/03/2021	IFM	08/03/2021

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

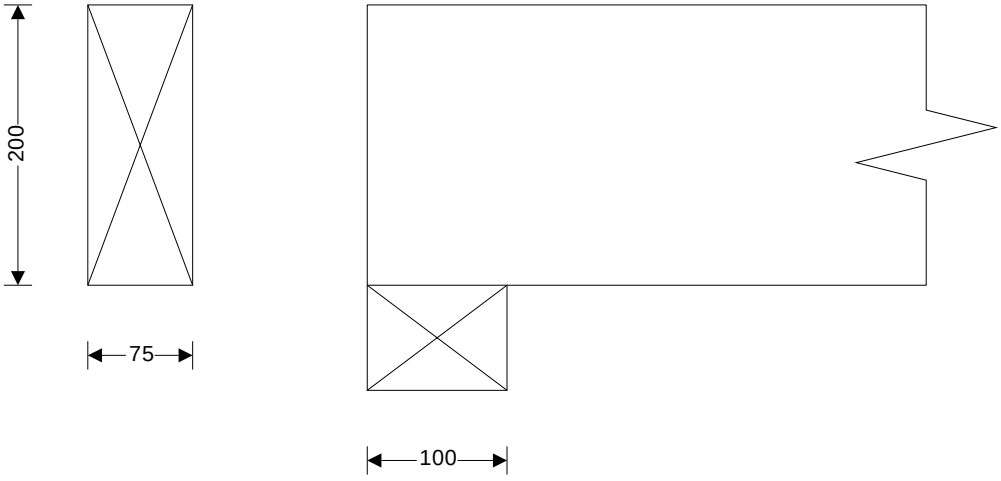
Joist details

Joist breadth	b = **75** mm
Joist depth	h = **200** mm
Joist spacing	s = **400** mm
Timber strength class	**C24**
Service class of timber	**1**



Span details

Number of spans	N_{span} = **1**
Length of bearing	L_b = **100** mm
Effective length of span	L_{s1} = **4500** mm



Section properties

Second moment of area	I = b ´ h^3 / 12 = **50000000** mm^4
Section modulus	Z = b ´ h^2 / 6 = **500000** mm^3

Loading details

Joist self weight	F_{swt} = b ´ h ´ r_{char} ´ g_{acc} = **0.05** kN/m
Dead load	F_{d_udl} = **0.56** kN/m^2
Imposed UDL(Long term)	F_{i_udl} = **2.50** kN/m^2
Imposed point load (Medium term)	F_{i_pt} = **1.40** kN

Modification factors

Service class for bending parallel to grain	K_{2m} = **1.00**

Service class for compression	K_{2c} = **1.00**
Service class for shear parallel to grain	K_{2s} = **1.00**
Service class for modulus of elasticity	K_{2e} = **1.00**
Section depth factor	K_7 = **1.05**
Load sharing factor	K_8 = **1.10**

Consider long term loads

Load duration factor	K_3 = **1.00**
Maximum bending moment	M = **3.229** kNm
Maximum shear force	V = **2.870** kN
Maximum support reaction	R = **2.870** kN
Maximum deflection	d = **12.994** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.626** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **6.457** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	τ = **0.710** N/mm^2
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm^2
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.287** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)	σ_{cp1} = **2.400** N/mm^2
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm^2
Applied bearing stress	σ_{c_max} = R / (b × L_b) = **0.383** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	d_{adm} = min(L_{s1} × 0.003, 14 mm) = **13.500** mm
Bending deflection (based on E_{mean})	$d_{bending}$ = **12.612** mm
Shear deflection	d_{shear} = **0.383** mm
Total deflection	$d = d_{bending} + d_{shear}$ = **12.994** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **2.272** kNm
Maximum shear force	V = **2.020** kN
Maximum support reaction	R = **2.020** kN
Maximum deflection	d = **7.915** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **4.545** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	t = **0.710** N/mm^2
Permissible shear stress	$t_{adm} = t \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm^2
Applied shear stress	$t_{max} = 3 \times V / (2 \times b \times h)$ = **0.202** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)	s_{cp1} = **2.400** N/mm^2
Permissible bearing stress	$s_{c_adm} = s_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.300** N/mm^2
Applied bearing stress	$s_{c_max} = R / (b \times L_b)$ = **0.269** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

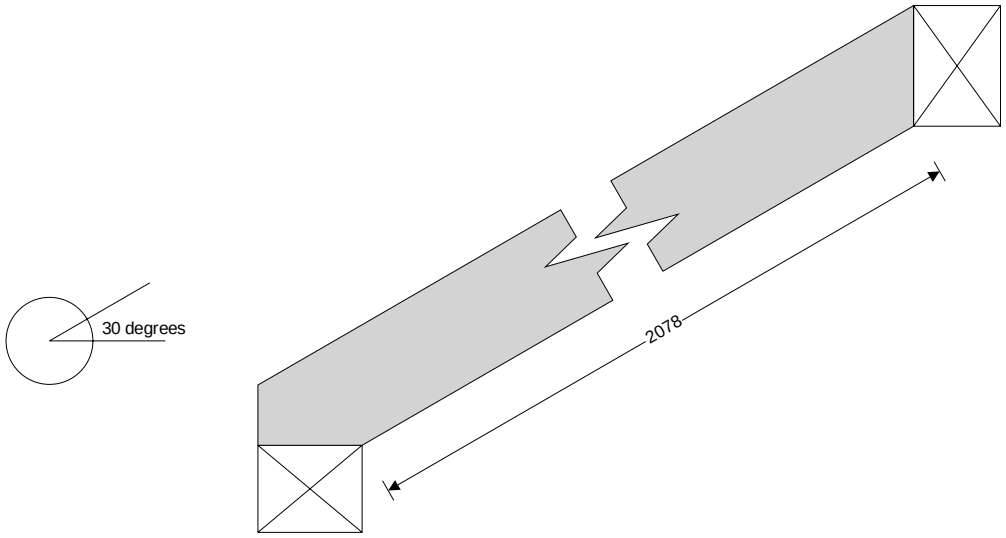
Permissible deflection	$d_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **13.500** mm
Bending deflection (based on E_{mean})	$d_{bending}$ = **7.646** mm
Shear deflection	d_{shear} = **0.269** mm
Total deflection	$d = d_{bending} + d_{shear}$ = **7.915** mm

PASS - Actual deflection within permissible limits

Project				Job no.	
2 Mansion House - West Boldon				21107	
Calcs for				Start page no./Revision	
Timber Rafters				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
U	08/03/2021	IFM	08/03/2021	IFM	08/03/2021

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections	b = **75** mm
Depth of timber sections	h = **90** mm
Rafter spacing	s = **600** mm
Rafter slope	a = **30.0** deg
Clear span of rafter on horizontal	L_{clh} = **1800** mm
Clear span of rafter on slope	$L_{cl} = L_{clh} / \cos(a)$ = **2078** mm
Rafter span	**Single span**
Timber strength class	**C16**

Section properties

Cross sectional area of rafter	$A = b \times h$ = **6750** mm^2
Section modulus	$Z = b \times h^2 / 6$ = **101250** mm^3
Second moment of area	$I = b \times h^3 / 12$ = **4556250** mm^4
Radius of gyration	$r = \sqrt{(I / A)}$ = **26.0** mm

Loading details

Rafter self weight	$F_j = b \times h \times r_{char} \times g_{acc}$ = **0.02** kN/m
Dead load on slope	F_d = **0.62** kN/m^2
Imposed load on plan	F_u = **0.75** kN/m^2
Imposed point load	F_p = **0.90** kN

Modification factors

Section depth factor	$K_7 = (300\ mm / h)^{0.11}$ = **1.14**
Load sharing factor	K_8 = **1.10**

Consider long term load condition

Load duration factor	K_3 = **1.00**
Total UDL perpendicular to rafter	$F = F_d \times \cos(a) \times s + F_j \times \cos(a)$ = **0.340** kN/m
Notional bearing length	$L_b = F \times L_{cl} / [2 \times (b \times s_{cp1} \times K_8 - F)]$ = **2** mm
Effective span	$L_{eff} = L_{cl} + L_b$ = **2080** mm

Check bending stress

Bending stress parallel to grain s_m = **5.300** N/mm²

Permissible bending stress $s_{m_adm} = s_m \times K_3 \times K_7 \times K_8$ = **6.656** N/mm²

Applied bending stress $s_{m_max} = F \times L_{eff}^2 / (8 \times Z)$ = **1.816** N/mm²

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain s_c = **6.800** N/mm²

Minimum modulus of elasticity E_{min} = **5800** N/mm²

Compression member factor K_{12} = **0.51**

Permissible compressive stress $s_{c_adm} = s_c \times K_3 \times K_8 \times K_{12}$ = **3.816** N/mm²

Applied compressive stress $s_{c_max} = F \times L_{eff} \times (\cot(a) + 3 \times \tan(a)) / (2 \times A)$ = **0.181** N/mm²

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $s_e = \pi^2 \times E_{min} / l^2$ = **8.928** N/mm²

Euler coefficient $K_{eu} = 1 - (1.5 \times s_{c_max} \times K_{12} / s_e)$ = **0.984**

Combined axial compression and bending check $s_{m_max} / (s_{m_adm} \times K_{eu}) + s_{c_max} / s_{c_adm}$ = **0.325 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain t = **0.670** N/mm²

Permissible shear stress $t_{adm} = t \times K_3 \times K_8$ = **0.737** N/mm²

Applied shear stress $t_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.079** N/mm²

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $d_{adm} = 0.003 \times L_{eff}$ = **6.241** mm

Bending deflection $d_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **2.068** mm

Shear deflection $d_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.059** mm

Total deflection $d_{max} = d_b + d_s$ = **2.127** mm

PASS - Total deflection within permissible limits

<u>Consider medium term load condition</u>

Load duration factor K_3 = **1.25**

Total UDL perpendicular to rafter $F = [F_u \times \cos(a)^2 + F_d \times \cos(a)] \times s + F_j \times \cos(a)$ = **0.677** kN/m

Notional bearing length $L_b = F \times L_{cl} / [2 \times (b \times s_{cp1} \times K_8 - F)]$ = **4** mm

Effective span $L_{eff} = L_{cl} + L_b$ = **2082** mm

Check bending stress

Bending stress parallel to grain s_m = **5.300** N/mm²

Permissible bending stress $s_{m_adm} = s_m \times K_3 \times K_7 \times K_8$ = **8.319** N/mm²

Applied bending stress $s_{m_max} = F \times L_{eff}^2 / (8 \times Z)$ = **3.627** N/mm²

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain s_c = **6.800** N/mm²

Minimum modulus of elasticity E_{min} = **5800** N/mm²

Compression member factor K_{12} = **0.46**

Permissible compressive stress $s_{c_adm} = s_c \times K_3 \times K_8 \times K_{12}$ = **4.302** N/mm²

Applied compressive stress $s_{c_max} = F \cdot L_{eff} \cdot (\cot(a) + 3 \cdot \tan(a)) / (2 \cdot A) =$ **0.362** N/mm²

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $s_e = p^2 \cdot E_{min} / l^2 =$ **8.911** N/mm²

Euler coefficient $K_{eu} = 1 - (1.5 \cdot s_{c_max} \cdot K_{12} / s_e) =$ **0.972**

Combined axial compression and bending check $s_{m_max} / (s_{m_adm} \cdot K_{eu}) + s_{c_max} / s_{c_adm} =$ **0.533 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $t =$ **0.670** N/mm²

Permissible shear stress $t_{adm} = t \cdot K_3 \cdot K_8 =$ **0.921** N/mm²

Applied shear stress $t_{max} = 3 \cdot F \cdot L_{eff} / (4 \cdot A) =$ **0.157** N/mm²

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $d_{adm} = 0.003 \cdot L_{eff} =$ **6.247** mm

Bending deflection $d_b = 5 \cdot F \cdot L_{eff}^4 / (384 \cdot E_{mean} \cdot I) =$ **4.137** mm

Shear deflection $d_s = 12 \cdot F \cdot L_{eff}^2 / (5 \cdot E_{mean} \cdot A) =$ **0.119** mm

Total deflection $d_{max} = d_b + d_s =$ **4.255** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor $K_3 =$ **1.50**

Total UDL perpendicular to rafter $F = F_d \cdot \cos(a) \cdot s + F_j \cdot \cos(a) =$ **0.340** kN/m

Notional bearing length $L_b = [F \cdot L_{cl} + F_p \cdot \cos(a)] / [2 \cdot (b \cdot s_{cp1} \cdot K_8 - F)] =$ **4** mm

Effective span $L_{eff} = L_{cl} + L_b =$ **2083** mm

Check bending stress

Bending stress parallel to grain $s_m =$ **5.300** N/mm²

Permissible bending stress $s_{m_adm} = s_m \cdot K_3 \cdot K_7 \cdot K_8 =$ **9.983** N/mm²

Applied bending stress $s_{m_max} = F \cdot L_{eff}^2/(8 \cdot Z)+F_p \cdot \cos(a) \cdot L_{eff}/(4 \cdot Z) =$ **5.828** N/mm²

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain $s_c =$ **6.800** N/mm²

Minimum modulus of elasticity $E_{min} =$ **5800** N/mm²

Compression member factor $K_{12} =$ **0.42**

Permissible compressive stress $s_{c_adm} = s_c \cdot K_3 \cdot K_8 \cdot K_{12} =$ **4.668** N/mm²

Applied compressive stress $s_{c_max} = F \cdot L_{eff} \cdot (\cot(a)+3 \cdot \tan(a))/(2 \cdot A)+F_p \cdot \sin(a)/A =$ **0.248** N/mm²

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $s_e = p^2 \cdot E_{min} / l^2 =$ **8.909** N/mm²

Euler coefficient $K_{eu} = 1 - (1.5 \cdot s_{c_max} \cdot K_{12} / s_e) =$ **0.983**

Combined axial compression and bending check $s_{m_max} / (s_{m_adm} \cdot K_{eu}) + s_{c_max} / s_{c_adm} =$ **0.647 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $t =$ **0.670** N/mm²

Permissible shear stress $t_{adm} = t \cdot K_3 \cdot K_8 =$ **1.106** N/mm²

Applied shear stress $t_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(a) / (2 \times A) =$ **0.252** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $d_{adm} = 0.003 \times L_{eff} =$ **6.248** mm

Bending deflection $d_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(a)/ 48) / (E_{mean} \times I) =$ **5.734** mm

Shear deflection $d_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(a))/(5 \times E_{mean} \times A) =$ **0.191** mm

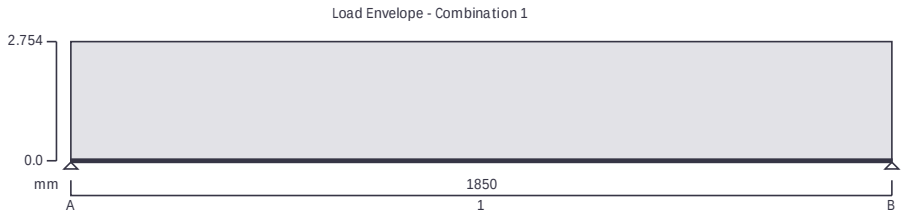
Total deflection $d_{max} = d_b + d_s =$ **5.925** mm

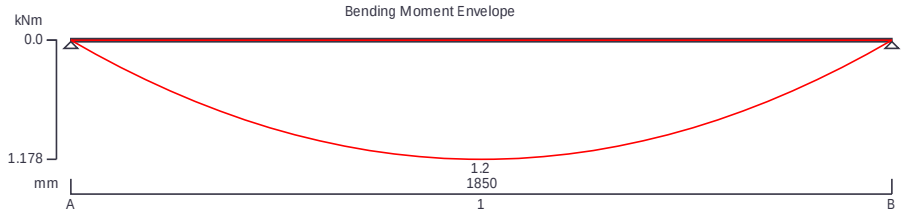
PASS - Total deflection within permissible limits

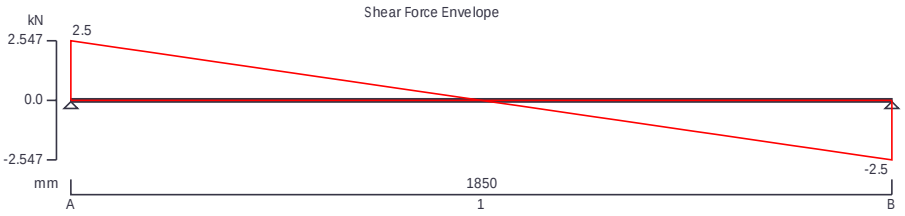
Tekla Tedds	Project: 2 Mansion House				Job no.: 21107	
	Calcs for: Timber purlins at higher level				Start page no./Revision: 1	
	Calcs by: U	Calcs date: 08/03/2021	Checked by: IFM	Checked date: 08/03/2021	Approved by: IFM	Approved date: 08/03/2021

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead self weight of beam ´ 1

Dead full UDL 1.600 kN/m

Imposed full UDL 1.100 kN/m

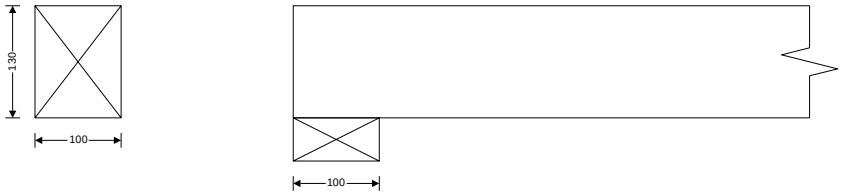
Load combinations

Load combination 1	Support A	Dead ´ 1.00
Imposed ´ 1.00		
Span 1	Dead ´ 1.00	
Imposed ´ 1.00		
Support B	Dead ´ 1.00	
Imposed ´ 1.00		

Analysis results

Maximum moment	M_{max} = **1.178** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.178** kNm	
Maximum shear	F_{max} = **2.547** kN	F_{min} = **-2.547** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.547** kN	
Total load on beam	W_{tot} = **5.094** kN	

Reactions at support A	R_{A_max} = **2.547** kN	R_{A_min} = **2.547** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.530** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **1.018** kN	
Reactions at support B	R_{B_max} = **2.547** kN	R_{B_min} = **2.547** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.530** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **1.017** kN	



Timber section details

Breadth of sections	b = **100** mm
Depth of sections	h = **130** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N ´ b = **100** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **1850** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N ´ b ´ h = **13000** mm^2
Section modulus	Z_x = N ´ b ´ h^2 / 6 = **281667** mm^3
Z_y = h ´ (N ´ b)^2 / 6 = **216667** mm^3	
Second moment of area	I_x = N ´ b ´ h^3 / 12 = **18308333** mm^4
I_y = h ´ (N ´ b)^3 / 12 = **10833333** mm^4	
Radius of gyration	i_x = Ö(I_x / A) = **37.5** mm
i_y = Ö(I_y / A) = **28.9** mm	

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.10**
Load sharing - cl.2.9	K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19	**5.00**
Actual depth-to-breadth ratio	h / (N ´ b) = **1.30**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	s_{c_adm} = s_{cp1} ´ K_3 ´ K_4 ´ K_8 = **2.400** N/mm^2
Applied bearing stress	s_{c_a} = R_{A_max} / (N ´ b ´ L_b) = **0.255** N/mm^2

s_{c_a} / s_{c_adm} = **0.106**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	$s_{m_adm} = s_m \times K_3 \times K_7 \times K_8$ = **8.223** N/mm^2
Applied bending stress	s_{m_a} = M / Z_x = **4.182** N/mm^2

s_{m_a} / s_{m_adm} = **0.509**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$t_{adm} = t \times K_3 \times K_8$ = **0.710** N/mm^2
Applied shear stress	$t_a = 3 \times F / (2 \times A)$ = **0.294** N/mm^2

t_a / t_{adm} = **0.414**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min}$ = **7200** N/mm^2
Permissible deflection	d_{adm} = min(0.551 in, 0.003 × L_{s1}) = **5.550** mm
Bending deflection	d_{b_s1} = **3.186** mm
Shear deflection	d_{v_s1} = **0.242** mm
Total deflection	$d_a = d_{b_s1} + d_{v_s1}$ = **3.428** mm

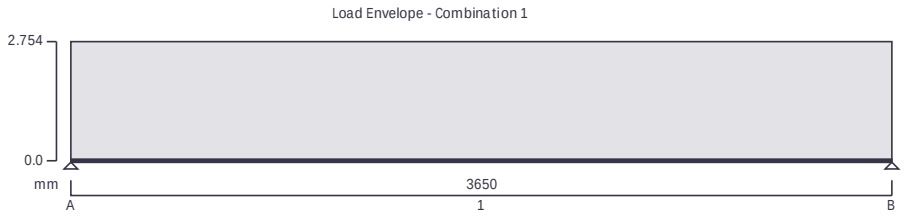
d_a / d_{adm} = **0.618**

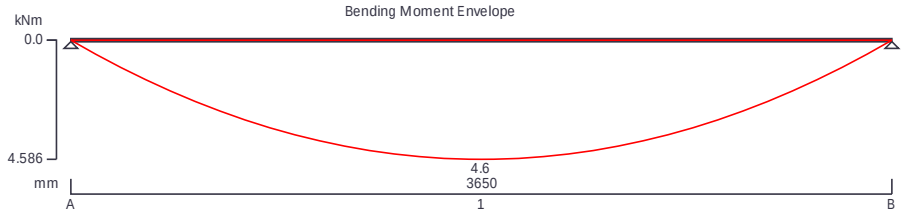
PASS - Total deflection is less than permissible deflection

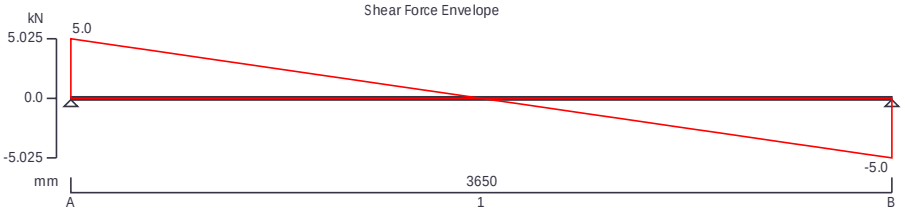
Tekla Tedds	Project: 2 Mansion House				Job no.: 21107	
	Calcs for: Timber purlins at lower level				Start page no./Revision: 1	
	Calcs by: U	Calcs date: 08/03/2021	Checked by: IFM	Checked date: 08/03/2021	Approved by: IFM	Approved date: 08/03/2021

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead self weight of beam ´ 1

Dead full UDL 1.600 kN/m

Imposed full UDL 1.100 kN/m

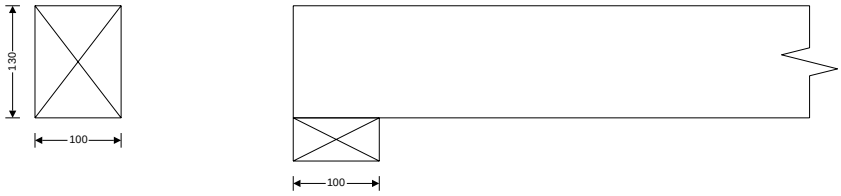
Load combinations

Load combination 1	Support A	Dead ´ 1.00
Imposed ´ 1.00		
Span 1	Dead ´ 1.00	
Imposed ´ 1.00		
Support B	Dead ´ 1.00	
Imposed ´ 1.00		

Analysis results

Maximum moment	M_{max} = **4.586** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4.586** kNm	
Maximum shear	F_{max} = **5.025** kN	F_{min} = **-5.025** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **5.025** kN	
Total load on beam	W_{tot} = **10.050** kN	

Reactions at support A	R_{A_max} = **5.025** kN	R_{A_min} = **5.025** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.018** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **2.008** kN	
Reactions at support B	R_{B_max} = **5.025** kN	R_{B_min} = **5.025** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.018** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **2.007** kN	



Timber section details

Breadth of sections	b = **100** mm
Depth of sections	h = **130** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N ´ b = **100** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **3650** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N ´ b ´ h = **13000** mm^2
Section modulus	Z_x = N ´ b ´ h^2 / 6 = **281667** mm^3
Z_y = h ´ (N ´ b)^2 / 6 = **216667** mm^3	
Second moment of area	I_x = N ´ b ´ h^3 / 12 = **18308333** mm^4
I_y = h ´ (N ´ b)^3 / 12 = **10833333** mm^4	
Radius of gyration	i_x = Ö(I_x / A) = **37.5** mm
i_y = Ö(I_y / A) = **28.9** mm	

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.10**
Load sharing - cl.2.9	K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19	**5.00**
Actual depth-to-breadth ratio	h / (N ´ b) = **1.30**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	s_{c_adm} = s_{cp1} ´ K_3 ´ K_4 ´ K_8 = **2.400** N/mm^2
Applied bearing stress	s_{c_a} = R_{A_max} / (N ´ b ´ L_b) = **0.503** N/mm^2

s_{c_a} / s_{c_adm} = **0.209**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $s_{m_adm} = s_m \cdot K_3 \cdot K_7 \cdot K_8 =$ **8.223** N/mm²

Applied bending stress $s_{m_a} = M / Z_x =$ **16.280** N/mm²

s_{m_a} / s_{m_adm} = **1.980**

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress $t_{adm} = t \cdot K_3 \cdot K_8 =$ **0.710** N/mm²

Applied shear stress $t_a = 3 \cdot F / (2 \cdot A) =$ **0.580** N/mm²

t_a / t_{adm} = **0.817**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} =$ **7200** N/mm²

Permissible deflection $d_{adm} = \min(0.551\text{ in}, 0.003 \cdot L_{s1}) =$ **10.950** mm

Bending deflection $d_{b_s1} =$ **48.275** mm

Shear deflection $d_{v_s1} =$ **0.941** mm

Total deflection $d_a = d_{b_s1} + d_{v_s1} =$ **49.215** mm

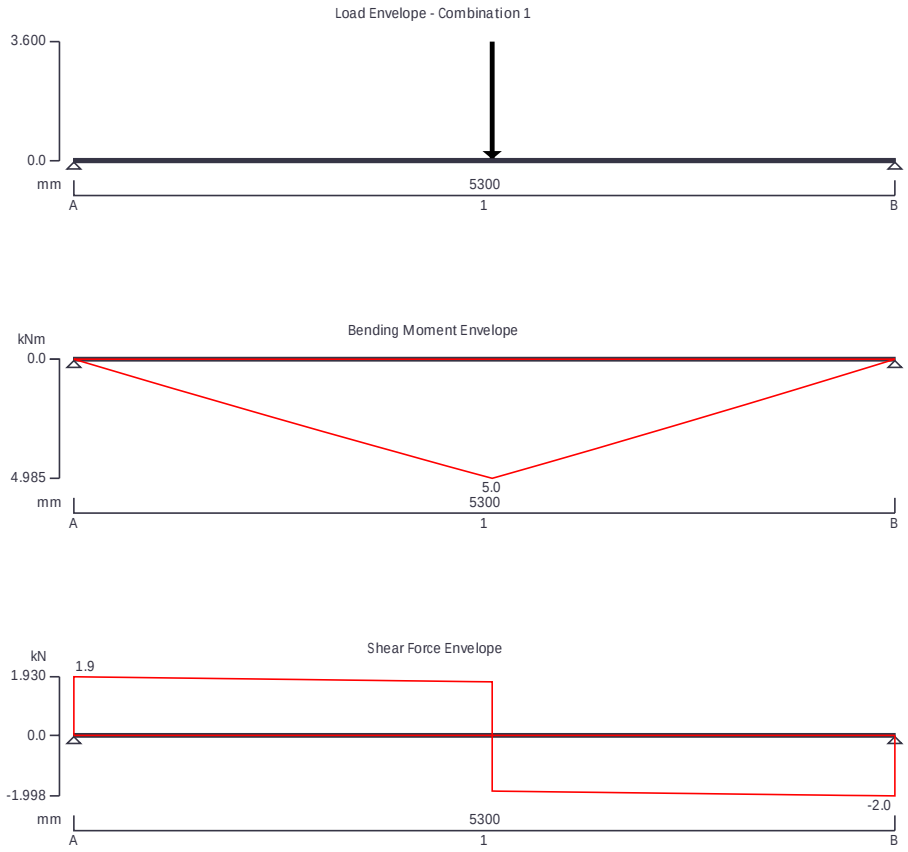
d_a / d_{adm} = **4.495**

FAIL - Total deflection exceeds permissible deflection

Tekla Tedds	Project: 2 Mansion House				Job no.: 21107	
	Calcs for: Hip rafters				Start page no./Revision: 1	
	Calcs by: U	Calcs date: 08/03/2021	Checked by: IFM	Checked date: 08/03/2021	Approved by: IFM	Approved date: 08/03/2021

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam ´ 1

Dead point load 2.100 kN at 2700 mm

Dead point load 1.500 kN at 2700 mm

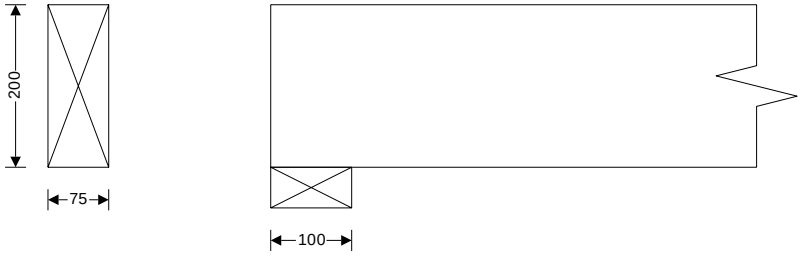
Load combinations

Load combination 1	Support A	Dead ´ 1.00
Imposed ´ 1.00		
Span 1	Dead ´ 1.00	
Imposed ´ 1.00		
Support B	Dead ´ 1.00	
Imposed ´ 1.00		

Analysis results

Maximum moment	M_{max} = **4.985** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4.985** kNm	
Maximum shear	F_{max} = **1.930** kN	F_{min} = **-1.998** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.998** kN	
Total load on beam	W_{tot} = **3.927** kN	

Reactions at support A	R_{A_max} = **1.930** kN	R_{A_min} = **1.930** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.930** kN	
Reactions at support B	R_{B_max} = **1.998** kN	R_{B_min} = **1.998** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.998** kN	



Timber section details

Breadth of sections	b = **75** mm
Depth of sections	h = **200** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N ´ b = **75** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **5300** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N ´ b ´ h = **15000** mm^2
Section modulus	Z_x = N ´ b ´ h^2 / 6 = **500000** mm^3
Z_y = h ´ (N ´ b)^2 / 6 = **187500** mm^3	
Second moment of area	I_x = N ´ b ´ h^3 / 12 = **50000000** mm^4
I_y = h ´ (N ´ b)^3 / 12 = **7031250** mm^4	
Radius of gyration	i_x = √(I_x / A) = **57.7** mm
i_y = √(I_y / A) = **21.7** mm	

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.05**
Load sharing - cl.2.9	K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19	**5.00**
Actual depth-to-breadth ratio	h / (N ´ b) = **2.67**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	σ_{c_adm} = σ_{cp1} ´ K_3 ´ K_4 ´ K_8 = **2.400** N/mm^2
Applied bearing stress	σ_{c_a} = R_{B_max} / (N ´ b ´ L_b) = **0.266** N/mm^2

s_{c_a} / s_{c_adm} = **0.111**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress — $s_{m_adm} = s_m \times K_3 \times K_7 \times K_8$ = **7.842** N/mm²

Applied bending stress — $s_{m_a} = M / Z_x$ = **9.970** N/mm²

s_{m_a} / s_{m_adm} = **1.271**

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress — $t_{adm} = t \times K_3 \times K_8$ = **0.710** N/mm²

Applied shear stress — $t_a = 3 \times F / (2 \times A)$ = **0.200** N/mm²

t_a / t_{adm} = **0.281**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection — $E = E_{min}$ = **7200** N/mm²

Permissible deflection — $d_{adm} = \min(0.551\text{ in}, 0.003 \times L_{s1})$ = **13.995** mm

Bending deflection — d_{b_s1} = **32.764** mm

Shear deflection — d_{v_s1} = **0.886** mm

Total deflection — $d_a = d_{b_s1} + d_{v_s1}$ = **33.651** mm

d_a / d_{adm} = **2.404**

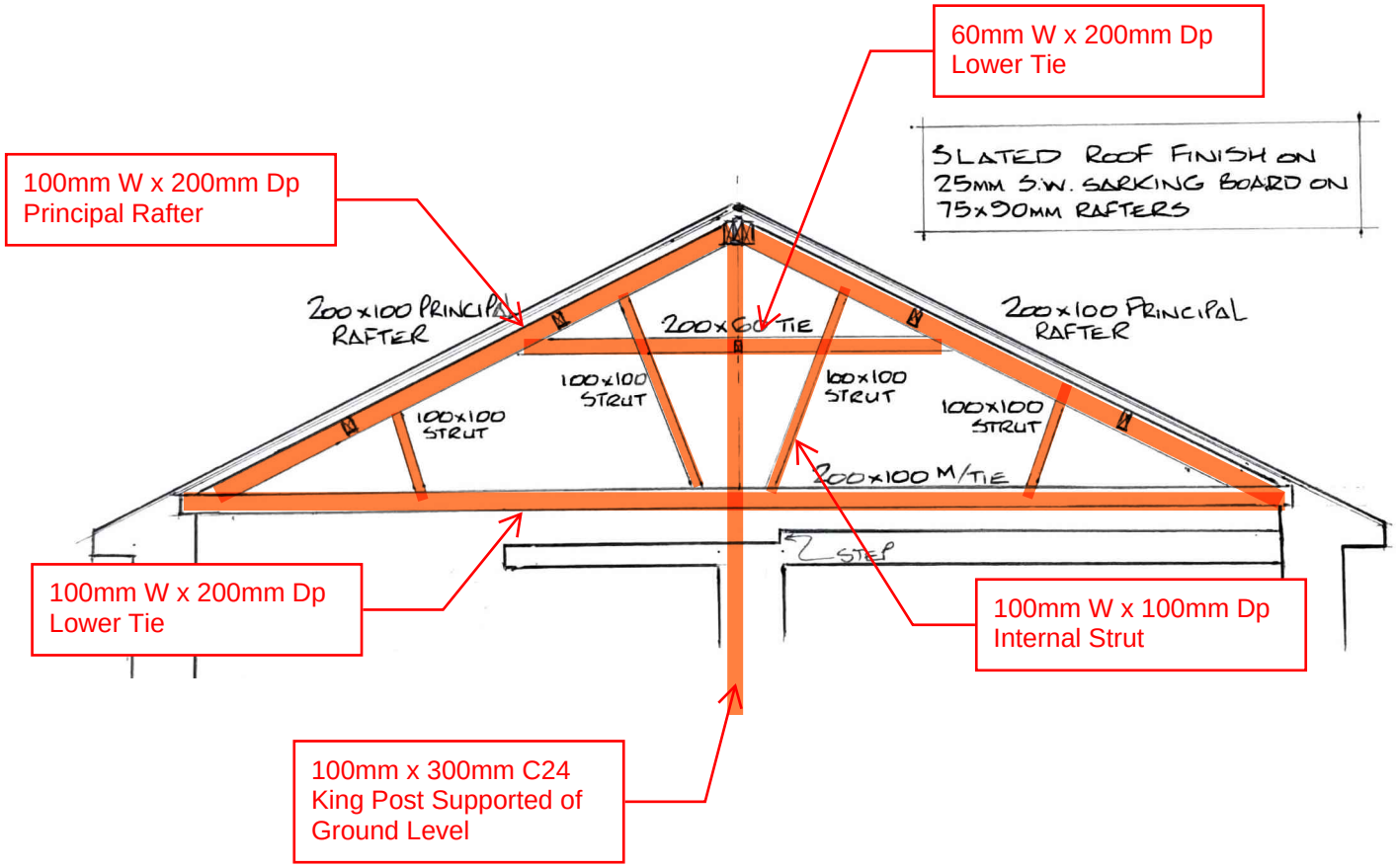
FAIL - Total deflection exceeds permissible deflection

	2 Mansion House	26/04/21
	Timber Roof Truss Analysis & Member Checks	KM

APPENDIX C

ANALYSIS & DESIGN CHECKS OF TRUSS MEMBERS FOR PROPOSED ARRANGEMENT

	2 Mansion House	26/04/21
	Timber Roof Truss Analysis & Member Checks	KM

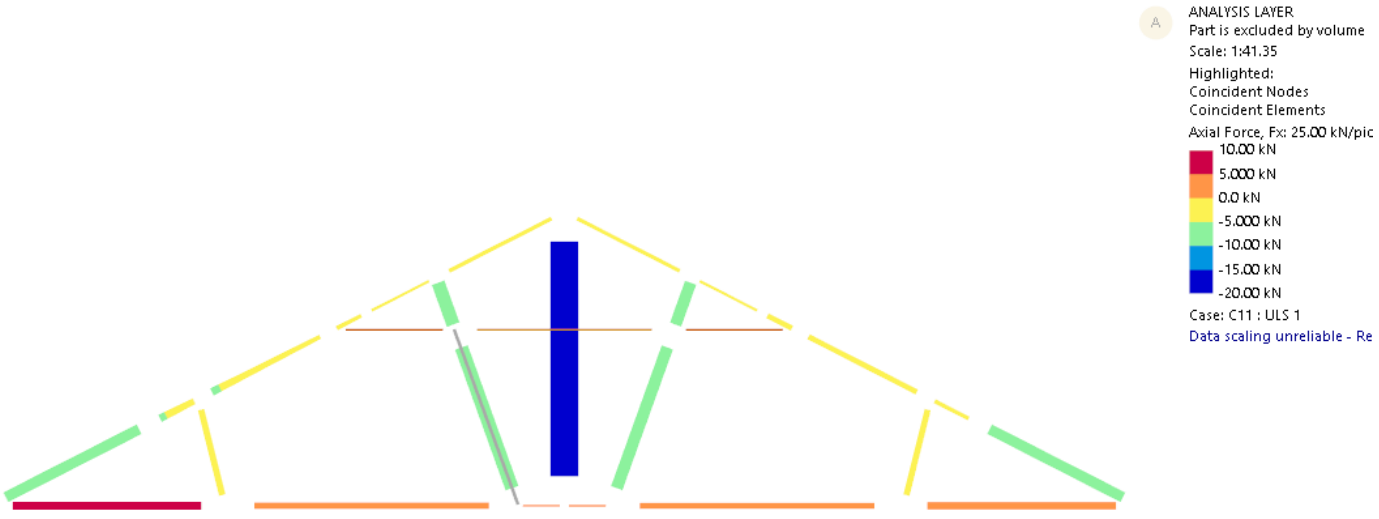


New Roof Truss Arrangement

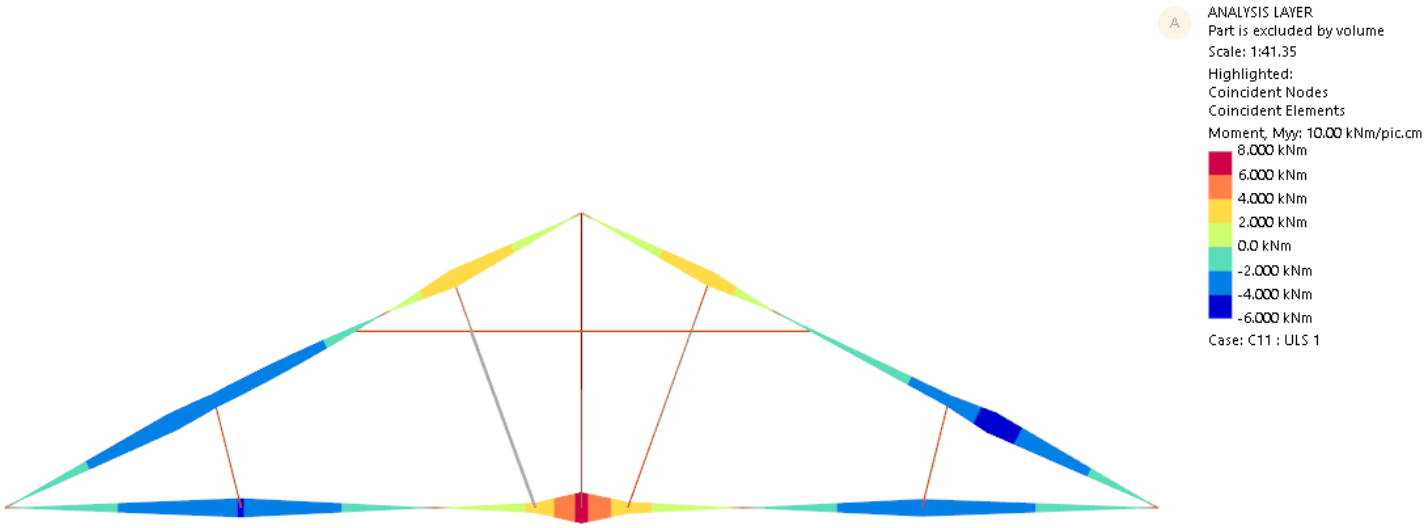
	2 Mansion House	10/03/21
	Timber Roof Truss Analysis & Member Checks	KM

Analysis Results

ULS Results



Analysis Results - Axial Force (kN)

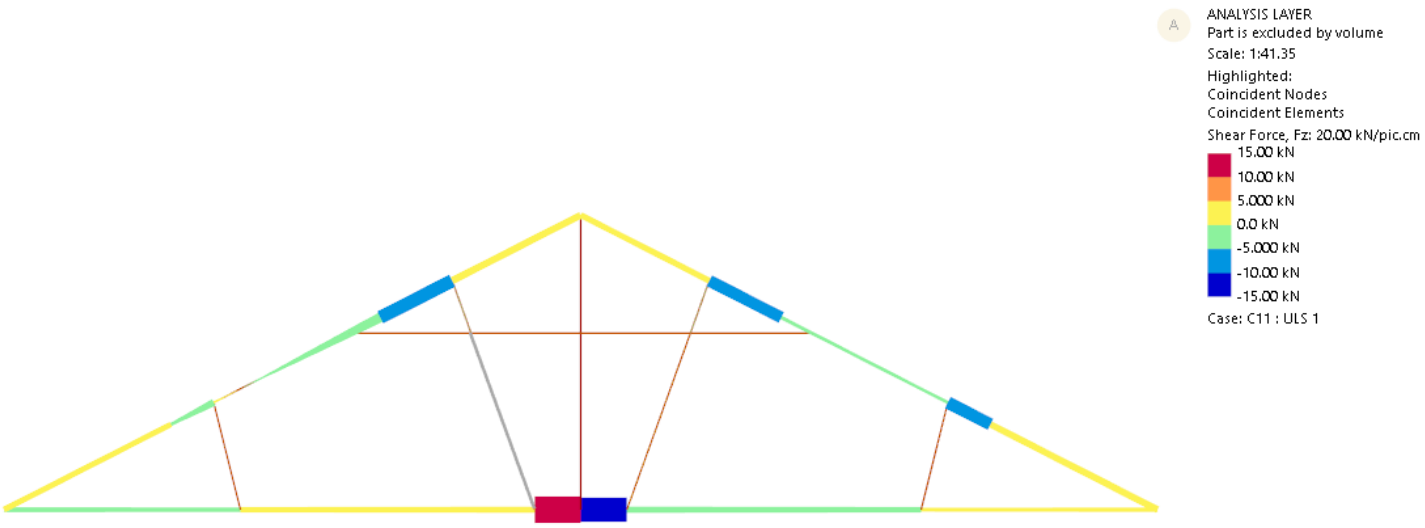


Analysis Results -Bending Moment (kNm)

	2 Mansion House	26/04/21
	Timber Roof Truss Analysis & Member Checks	KM

Analysis Results

ULS Results



Analysis Results - Shear (kN)

Typical Member Checks

Member Design Results - Proposed Arrangement

Member	Effective Lengths		Design Forces			Pass / Fail
	$Le_{,major}$ (m)	$Le_{,minor}$ (m)	Axial (kN)	Bending Moment (kN/m)	Shear (kN)	
100mm x 200mm Rafter	5.65	2.32	-7.50	4.70	3.60	Pass
100mm x 200mm Lower Tie	5.00	10.00	-5.00	6.65	13.20	Pass
100mm x 100mm Strut	2.15	2.15	6.25	0.25	0.25	Pass
60mm x 200mm Upper Tie	4.05	4.05	-7.55	0.1	0.25	Pass
New post pikcing up lower purlin: 100mm x 300mm	4.00	4.00	40.00	0.25	0.25	Pass
New post picking up existing truss: 100mm x 300mm	4.00	4.00	40.00	0.25	0.25	Pass

Summary Table

100MM WIDE X 200MM DEEP LOWER TIE

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

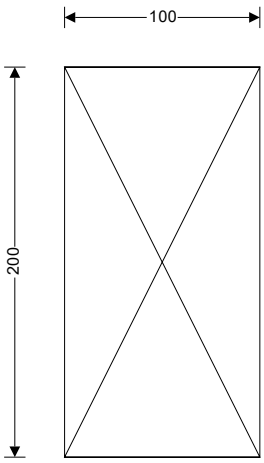
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **200** mm

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



100x200 timber section

Cross-sectional area, A, 20000 mm^2

Section modulus, W_y, 666666.7 mm^3

Section modulus, W_z, 333333 mm^3

Second moment of area, I_y, 66666667 mm^4

Second moment of area, I_z, 16666667 mm^4

Radius of gyration, i_y, 57.7 mm

Radius of gyration, i_z, 28.9 mm

Timber strength class C24

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

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

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

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

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

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

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

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

Characteristic density, ρ_k, 350 kg/m^3

Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Minor axis L_z = **1000** mm

Effective length - Minor axis $L_{e,z} = 0.9 \times L_z$ = **900** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **6.65** kNm

Design shear force - Major axis $F_{y,d}$ = **13.2** kN

Design perpendicular compression - Major axis $F_{c,y,90,d}$ = **4.65** kN

Design axial tension force P_d = **5** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Tensile stress	N/mm^2	6.5	0.3	0.038	PASS
Bearing stress	N/mm^2	1.1	0.5	0.413	PASS
Bending stress	N/mm^2	10.8	10.0	0.924	PASS
Shear stress	N/mm^2	1.8	1.5	0.821	PASS
Bending and axial force				0.962	PASS

Beam stability check				0.924	PASS

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.9**
Deformation factor - Table 3.2	k_{def} = **0.6**
Depth factor for tension - exp.3.1	$k_{h,t}$ = **1**
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**

Check tension parallel to the grain - Section 6.1.2

Axial tension	P_d = **5** kN
Design tensile stress	$\sigma_{t,0,d} = P_d / A$ = **0.250** N/mm^2
Design tensile strength	$f_{t,0,d} = k_{h,t} \times k_{mod} \times f_{t.0.k} / \gamma_M$ = **6.525** N/mm^2
	$\sigma_{t,0,d} / f_{t,0,d}$ = **0.038**

PASS - Design tensile strength exceeds design tensile stress

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d}$ = **4.65** kN
Effective contact length	$L_{b,ef} = L_b$ = **100** mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.465** N/mm^2
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times f_{c.90.k} / \gamma_M$ = **1.125** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.413**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **13.2** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **1.478** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.821**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **6.65** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **9.975** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.924**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial tension - Section 6.2.3

Combined loading checks - exp.6.17 & 6.18	$\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d}$ = **0.962**
	$\sigma_{t,0,d} / f_{t,0,d} + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.685**

PASS - Combined bending and axial tension utilisation is acceptable

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

Effective length - Table 6.1	$L_{ef} = 0.9 \times 1000$ mm = **900** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **320.667** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.274**
Lateral buckling factor - exp.6.34	k_{crit} = **1.000**
Beam stability check - exp.6.33	$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d})$ = **0.924**

PASS - Beam stability is acceptable

100MM WIDE X 200MM DEEP PRINCIPAL RAFTER

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

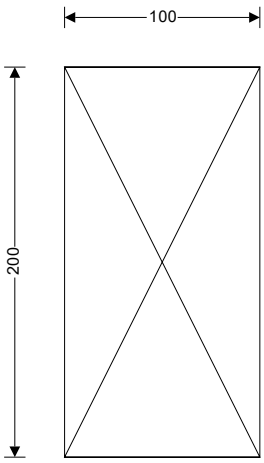
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **200** mm

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



100x200 timber section

Cross-sectional area, A, 20000 mm^2

Section modulus, W_y, 666666.7 mm^3

Section modulus, W_z, 333333 mm^3

Second moment of area, I_y, 66666667 mm^4

Second moment of area, I_z, 16666667 mm^4

Radius of gyration, i_y, 57.7 mm

Radius of gyration, i_z, 28.9 mm

Timber strength class C24

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

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

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

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

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

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

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

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

Characteristic density, ρ_k, 350 kg/m^3

Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Minor axis L_z = **2320** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **2320** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **4.7** kNm

Design shear force - Major axis $F_{y,d}$ = **3.6** kN

Design axial tension force P_d = **7.5** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Tensile stress	N/mm^2	6.5	0.4	0.057	PASS
Bending stress	N/mm^2	10.8	7.1	0.653	PASS
Shear stress	N/mm^2	1.8	0.4	0.224	PASS
Bending and axial force				0.710	PASS
Beam stability check				0.653	PASS

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.9**
Deformation factor - Table 3.2	k_{def} = **0.6**
Depth factor for tension - exp.3.1	$k_{h,t}$ = **1**
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**

Check tension parallel to the grain - Section 6.1.2

Axial tension	P_d = **7.5** kN
Design tensile stress	$\sigma_{t,0,d} = P_d / A$ = **0.375** N/mm^2
Design tensile strength	$f_{t,0,d} = k_{h,t} \times k_{mod} \times f_{t.0.k} / \gamma_M$ = **6.525** N/mm^2
	$\sigma_{t,0,d} / f_{t,0,d}$ = **0.057**

PASS - Design tensile strength exceeds design tensile stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **3.6** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.403** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.224**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **4.7** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **7.05** N/mm^2
Design bending strength	$f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d}$ = **0.653**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial tension - Section 6.2.3

Combined loading checks - exp.6.17 & 6.18	$\sigma_{t,0,d} / f_{t,0,d} + \sigma_{m,y,d} / f_{m,y,d}$ = **0.710**
	$\sigma_{t,0,d} / f_{t,0,d} + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.514**

PASS - Combined bending and axial tension utilisation is acceptable

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

Effective length - Table 6.1	L_{ef} = 2320 mm = **2320** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **124.397** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.439**
Lateral buckling factor - exp.6.34	k_{crit} = **1.000**
Beam stability check - exp.6.33	$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d})$ = **0.653**

PASS - Beam stability is acceptable

100MM WIDE X 100MM INTERNAL STRUT

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

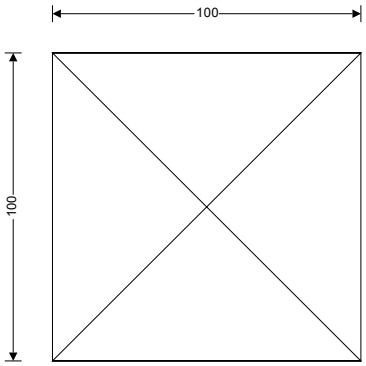
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **100** mm

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



100x100 timber section
Cross-sectional area, A, 10000 mm²
Section modulus, W_y, 166666.7 mm³
Section modulus, W_z, 166667 mm³
Second moment of area, I_y, 8333333 mm⁴
Second moment of area, I_z, 8333333 mm⁴
Radius of gyration, i_y, 28.9 mm
Radius of gyration, i_z, 28.9 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

Unbraced length - Major axis L_y = **2150** mm

Effective length - Major axis $L_{e,y} = L_y$ = **2150** mm

Unbraced length - Minor axis L_z = **2150** mm

Effective length - Minor axis $L_{e,z} = L_z$ = **2150** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **0.25** kNm

Design shear force - Major axis $F_{y,d}$ = **0.25** kN

Design axial compression force P_d = **6.25** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm²	9.5	0.6	0.066	PASS
Bending stress	N/mm²	11.7	1.5	0.128	PASS
Shear stress	N/mm²	1.8	0.1	0.031	PASS
Bending and axial force				0.132	PASS
Column stability check				0.259	PASS
Beam stability check				0.148	PASS

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.9**

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

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**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **6.25** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.625** N/mm²

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **9.450** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.066**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **0.25** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.056** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.031**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **0.25** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **1.5** N/mm²

Design bending strength $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times f_{m.k} / \gamma_M$ = **11.712** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.128**

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 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.132**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.094**

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 $L_{e,y}$ = **2150** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **74.478**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.263**

Effective length for z-axis bending $L_{e,z}$ = **2150** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **74.478**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.263**

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

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.394**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **1.394**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.504**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **0.504**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.259**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.221**

PASS - Column stability is acceptable

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

Effective length - Table 6.1	L_{ef} = 2150 mm = **2150** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **268.465** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.299**
Lateral buckling factor - exp.6.34	k_{crit} = **1.000**
Beam stability check - exp.6.35	$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **0.148**

PASS - Beam stability is acceptable

Tekla Tedds	Project: 2 Mansion House, West Boldon				Job no.	
	Calcs for: 100mm x 100mm Internal Strut				Start page no./Revision: 1	
	Calcs by	Calcs date: 26/04/2021	Checked by	Checked date	Approved by	Approved date

100MM WIDE X 100MM INTERNAL STRUT

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

Tedds calculation version 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

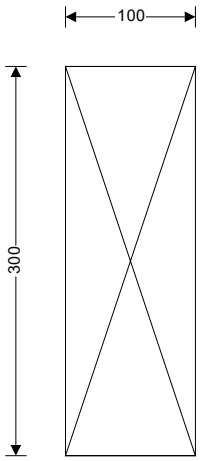
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **300** mm

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



100x300 timber section
Cross-sectional area, A, 30000 mm^2
Section modulus, W_y, 1500000 mm^3
Section modulus, W_z, 500000 mm^3
Second moment of area, I_y, 225000000 mm^4
Second moment of area, I_z, 25000000 mm^4
Radius of gyration, i_y, 86.6 mm
Radius of gyration, i_z, 28.9 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Major axis L_y = **4000** mm

Effective length - Major axis $L_{e,y}$ = L_y = **4000** mm

Unbraced length - Minor axis L_z = **4000** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **4000** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **0.25** kNm

Design shear force - Major axis $F_{y,d}$ = **0.25** kN

Design axial compression force P_d = **40** kN

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm^2	9.5	1.3	0.141	PASS
Bending stress	N/mm^2	10.8	0.2	0.015	PASS
Shear stress	N/mm^2	1.8	0.0	0.010	PASS
Bending and axial force				0.035	PASS

Column stability check				0.859	PASS
Beam stability check				0.849	PASS

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**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **40** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **1.333** N/mm^2

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **9.450** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.141**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **0.25** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.019** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.800** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.010**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **0.25** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **0.167** N/mm^2

Design bending strength $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **10.8** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.015**

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 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.035**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.031**

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 $L_{e,y}$ = **4000** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **46.188**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.783**

Effective length for z-axis bending $L_{e,z}$ = **4000** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **138.564**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **2.35**

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

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **0.855**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **3.465**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.835**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **0.166**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.184**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.859**

PASS - Column stability is acceptable

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

Effective length - Table 6.1	L_{ef} = 4000 mm = **4000** mm
Critical bending stress - exp.6.32	$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **48.100** N/mm^2
Relative slenderness for bending - exp.6.30	$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.706**
Lateral buckling factor - exp.6.34	k_{crit} = **1.000**
Beam stability check - exp.6.35	$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **0.849**

PASS - Beam stability is acceptable

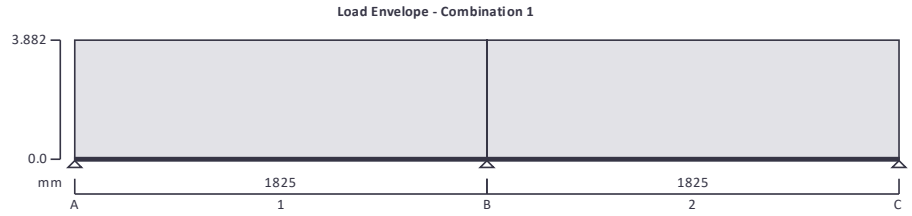
APPENDIX D

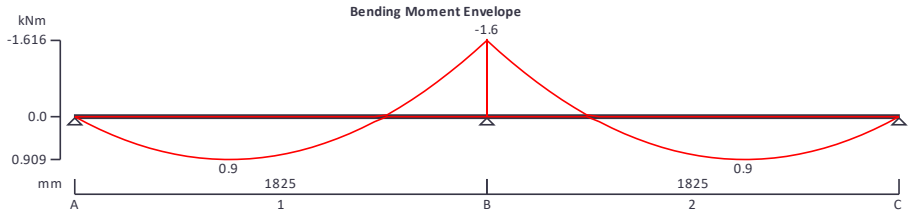
ANALYSIS & DESIGN CHECKS OF EXISTING ROOFING MEMBERS FOR PROPOSED ARRANGEMENT

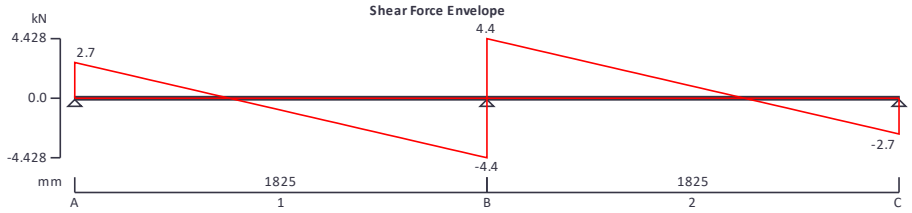
Tekla Tedds	Project: 2 Mansion House				Job no.: 21107	
	Calcs for: Timber purlins at lower level (supported at midspan)				Start page no./Revision: 1	
	Calcs by: K	Calcs date: 26/04/2021	Checked by: IFM	Checked date: 26/04/2021	Approved by: IFM	Approved date: 26/04/2021

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.600 kN/m
Imposed full UDL 1.100 kN/m

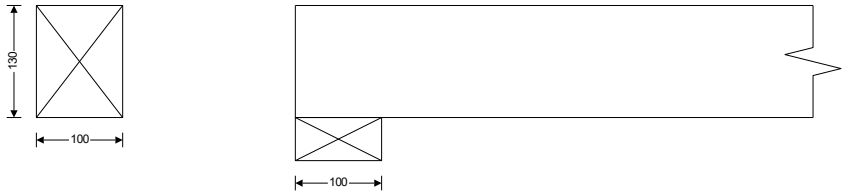
Load combinations

Load combination 1	Support A	Dead × 1.35
		Imposed × 1.50
	Span 1	Dead × 1.35
		Imposed × 1.50
	Support B	Dead × 1.35
		Imposed × 1.50
	Span 2	Dead × 1.35
		Imposed × 1.50
	Support C	Dead × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **0.909** kNm	M_{min} = **-1.616** kNm

Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.616** kNm	
Maximum shear	F_{max} = **4.428** kN	F_{min} = **-4.428** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **4.428** kN	
Total load on beam	W_{tot} = **14.170** kN	
Reactions at support A	R_{A_max} = **2.657** kN	R_{A_min} = **2.657** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.132** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **0.753** kN	
Reactions at support B	R_{B_max} = **8.856** kN	R_{B_min} = **8.856** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.772** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **2.509** kN	
Reactions at support C	R_{C_max} = **2.657** kN	R_{C_min} = **2.657** kN
Unfactored dead load reaction at support C	R_{C_Dead} = **1.132** kN	
Unfactored imposed load reaction at support C	$R_{C_Imposed}$ = **0.753** kN	



Timber section details

Breadth of sections	b = **100** mm
Depth of sections	h = **130** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span 1	L_{s1} = **1825** mm
Length of span 2	L_{s2} = **1825** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **13000** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **281667** mm^3
	Z_y = h × (N × b)^2 / 6 = **216667** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **18308333** mm^4
	I_y = h × (N × b)^3 / 12 = **10833333** mm^4
Radius of gyration	i_x = √(I_x / A) = **37.5** mm
	i_y = √(I_y / A) = **28.9** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.10**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.10**
Load sharing - cl.2.9	K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19 **5.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.30**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.640** N/mm²

Applied bearing stress $\sigma_{c_a} = R_{B_max} / (N \times b \times L_b)$ = **0.886** N/mm²

$\sigma_{c_a} / \sigma_{c_adm}$ = **0.335**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **8.223** N/mm²

Applied bending stress $\sigma_{m_a} = M / Z_x$ = **5.738** N/mm²

$\sigma_{m_a} / \sigma_{m_adm}$ = **0.698**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.710** N/mm²

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.511** N/mm²

τ_a / τ_{adm} = **0.720**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **7200** N/mm²

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1})$ = **5.475** mm

Bending deflection δ_{b_s1} = **1.255** mm

Shear deflection δ_{v_s1} = **0.132** mm

Total deflection $\delta_a = \delta_{b_s1} + \delta_{v_s1}$ = **1.387** mm

δ_a / δ_{adm} = **0.253**

PASS - Total deflection is less than permissible deflection

TIMBER MEMBER 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 2.1.03

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **2.000**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

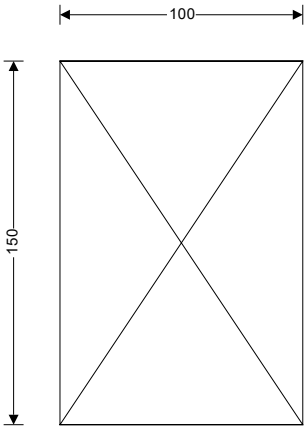
Timber section details

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

Breadth of sections b = **100** mm

Depth of sections h = **150** mm

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



100x150 timber section
Cross-sectional area, A, 15000 mm^2
Section modulus, W_y, 375000 mm^3
Section modulus, W_z, 250000 mm^3
Second moment of area, I_y, 28125000 mm^4
Second moment of area, I_z, 12500000 mm^4
Radius of gyration, i_y, 43.3 mm
Radius of gyration, i_z, 28.9 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Unbraced length - Major axis L_y = **4000** mm

Effective length - Major axis $L_{e,y}$ = L_y = **4000** mm

Unbraced length - Minor axis L_z = **4000** mm

Effective length - Minor axis $L_{e,z}$ = L_z = **4000** mm

Bearing length L_b = **100** mm

Analysis results

Design bending moment - Major axis $M_{y,d}$ = **0.25** kNm

Design bending moment - Minor axis $M_{z,d}$ = **0.25** kNm

Design shear force - Major axis $F_{y,d}$ = **1** kN

Design shear force - Minor axis $F_{z,d}$ = **1** kN

Design axial compression force P_d = **8.8** kN

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.7**

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

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

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **8.8** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.587** N/mm²

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **7.350** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.080**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check shear force - Section 6.1.7

Design shear force - major axis $F_{y,d}$ = **1** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.149** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.400** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.107**

Design shear force - minor axis $F_{z,d}$ = **1** kN

Design shear stress - exp.6.60 $\tau_{z,d} = 1.5 \times F_{z,d} / (k_{cr} \times b \times h)$ = **0.149** N/mm²

Design shear strength $f_{v,z,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **1.400** N/mm²

$\tau_{z,d} / f_{v,z,d}$ = **0.107**

Combined shear checks $\sqrt{((\tau_{y,d} / f_{v,y,d})^2 + (\tau_{z,d} / f_{v,z,d})^2)}$ = **0.151**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment in major axis $M_{y,d}$ = **0.25** kNm

Design bending stress in major axis $\sigma_{m,y,d} = M_{y,d} / W_y$ = **0.667** N/mm²

Design bending strength in major axis $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **8.400** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.079**

Design bending moment in minor axis $M_{z,d}$ = **0.25** kNm

Design bending stress in minor axis $\sigma_{m,z,d} = M_{z,d} / W_z$ = **1.000** N/mm²

Design bending strength in minor axis $f_{m,z,d} = k_{h,m,z} \times k_{mod} \times f_{m.k} / \gamma_M$ = **9.110** N/mm²

$\sigma_{m,z,d} / f_{m,z,d}$ = **0.110**

Combined bending checks - eq.6.11 & eq.6.12 $\sigma_{m,y,d} / f_{m,y,d} + k_m \times \sigma_{m,z,d} / f_{m,z,d}$ = **0.156**

$k_m \times \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d}$ = **0.165**

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 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} + k_m \times \sigma_{m,z,d} / f_{m,z,d}$ = **0.163**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d}$ = **0.172**

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 $L_{e,y}$ = **4000** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **92.376**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.566**

Effective length for z-axis bending $L_{e,z}$ = **4000** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **138.564**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **2.35**

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

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.853**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **3.465**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.352**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) =$ **0.166**

Column stability checks - exp.6.23 & 6.24

$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} + k_m \times \sigma_{m,z,d} / f_{m,z,d} =$ **0.383**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} + \sigma_{m,z,d} / f_{m,z,d} =$ **0.645**

PASS - Column stability is acceptable

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

Effective length - Table 6.1

$L_{ef} = 4000$ mm $+ 2 \times h =$ **4300** mm

Critical bending stress - exp.6.32

$\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef}) =$ **89.488** N/mm^2

Relative slenderness for bending - exp.6.30

$\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})} =$ **0.518**

Lateral buckling factor - exp.6.34

$k_{crit} =$ **1.000**

Beam stability check - exp.6.35

$(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) =$ **0.486**

PASS - Beam stability is acceptable

	2 Mansion House	26/04/21
	Timber Roof Truss Analysis & Member Checks	KM

New Steel Angle Supporting Hip Rafter

STEEL ANGLE STRENGTHENING TO HIP RAFTER.

From ORIGINAL HIP RAFTER ANALYSIS;

M_{yy} = 7.0 kNm

V_{ED} = 2.7 kNm

DEFLECTION GOVERNED;

$\delta_{ALLOWABLE}$ = Span/250 = 5300/250 = 21.2mm.

$\delta_{MAX} = PL^3/48EI$

$I_{REQ} \geq PL^3/48E(\delta_{MAX})$

$\geq (2.1 + 1.5)(5300)^3/48(205)(21.2)$

$\geq 256\ cm^4$

UKA 150 x 75 x 10 $\rightarrow$ I_{yy} = 533 cm^4

BENDING CHECK:

$W_{el,y} = 53.3\ cm^3$

$\sigma_{MAX} = 7 \times 10^6 / 53.3 \times 10^3$ = 131 N/mm^2 < 275 N/mm^2

$\therefore$ S275 STEEL

STEEL PASSED BY INSPECTION.

USE UKA 150 x 75 x 10.0