1663GE – Chippenham Town Hall, Decarbonisation

DESIGN STATEMENT AND STRUCTURAL CALCULATIONS

May 2025

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Document

Project name	Chippenham Town Hall, Decarbonisation
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Document name	Design Statement and Structural Calculations
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Document author	Ralph Pelly MEng (Hons) CEng MIStructE
Approved by	Ralph Pelly MEng (Hons) CEng MIStructE

Version Control

Revision	Date	Authorised	Description
/	12/06/25	Ralph Pelly	First issue.

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1 Design Statement

1.1 General Project Information

1.1.1 The Project

The project involves installation of new mechanical and electrical services:

1. In the Cheese Hall, serving the main hall
2. On the roof of the Town Hall and within the roof space above the Town Hall, serving the Town Hall.

The scope of service provided by Giraffe Engineering is to verify the capacity of the existing structure to support any newly applied design loads from the proposed services, and to design and specify any alterations required to the existing structure in order to facilitate the proposals and to strengthen the existing structure where necessary.

1.1.2 Existing structures

The existing structures comprise a group of interconnected buildings in Chippenham Town Centre, including the Chippenham Town Hall building, the Neeld Community and Arts Centre and the 'Cheese Hall' which adjoins the Neeld Community and Arts Centre Main Hall.

The existing structures are of masonry construction typically with timber floor joists and timber roof structure.

1.1.3 Contractor Designed Elements

Contractor designed elements are listed below:

1. Temporary works
2. Secondary steelwork for cladding and services
3. Access walkway to roof spaces

Where applicable, Giraffe Engineering will review fabrication drawings for design conformity and general conformance to contract documents only. The contractor is responsible for confirming dimensions and full compliance with contract documents.

1.2 Design principles

1.2.1 Disproportionate collapse and robustness

The Building is Class 2A in accordance with the Building Regulations. Resultantly, effective anchorage of floors and roofs to walls is provided by full embedment of floor plates into load bearing walls or provision of mechanical anchorage around floor plate perimeters.

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1.2.2 Design working life

The design working life of the structural proposals is 50 years, in accordance with BS EN 1990 Table 2.1 Category 4, 'normal' category of building.

1.2.3 Fire Resistance

Fire protection is to be specified by others.

Fire protection of timber and steelwork is to be provided by plasterboard finishes unless noted otherwise.

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1.3 Design standards

1.3.1 Reference documents

The structural design has been carried out in accordance with the following standards:

- BS EN 1990 Basis of structural design
- BS EN 1991-1-1 Actions on structures – Building loads
- BS EN 1991-1-3 Actions on structures – Snow loads
- BS EN 1991-1-4 Actions on structures – Wind loads
- BS EN 1992-1-1 Design of concrete structures
- BS EN 1993-1-1 Design of steel structures
- BS EN 1995-1-1 Design of timber structures
- BS EN 1996-1-1 Design of masonry structures
- Building Regulations – Approved document A

1.3.2 Deflection Limits

Standard deflection limits have been adopted, as per the recommendation given in the Eurocodes including their National Annexes.

Typically, these are as listed below unless noted otherwise:

Load case	Deflection limit
Live load only	Span/360
Live load only, over glazing, sliding doors, bifold doors	Span/500, maximum 10mm
Dead + live load, with brittle finishes such as plasterboard (including creep deflection)	Span/250 (maximum 14mm for inhabited floors)
Dead + live load, without brittle finishes (including creep deflection)	Span/125 (maximum 14mm for inhabited floors)
Lateral deflection of columns	Height/300 in total and per storey (maximum 10mm per storey)

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2 Loading

2.1 Permanent loads

G1 - Flat roof	kN/m²
Lead roof covering, assume max 4mm thick	0.4
Sheathing	0.20
Roof joists	0.10
Ceiling, services and insulation	0.1
Total	**0.9**

G2 - Internal timber floor	kN/m²
Finishes	0.10
Floorboards	0.10
Joists (150x50 @400c/c)	0.15
Ceiling and services	0.15
Total	**0.5**

G3 - Attic floor	kN/m²
Attic services	0.2
Insulation	0.05
Ceiling joists	0.1
Plasterboard ceiling, 15mm	0.15
Services below ceiling	0.1
Total	**0.6**

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G4 - Single skin masonry wall	kN/m^2
Brickwork (100mm)	2.10
Dense block	2.00
Lightweight block	1.00
Insulation	0.05
Timber rainscreen	0.15
Total	**2.10**

2.2 Imposed loads

Q1 - Storage Area	kN/m^2
Category E11, storage area	2.0
Total	**2.0**

Q2 - Roof	kN/m^2
Category H, maintenance	0.75
Total	**0.60**

Q3 - Roof space	kN/m^2
Category H, maintenance	0.60
Total	**0.60**

3 Stability

Stability of the existing structures is achieved by masonry shear walls in both orthogonal directions transferring lateral loads into the foundations. Floor boards/decking at internal floor levels and bracing/trusses at roof level act as horizontal diaphragms to convey lateral loads to the masonry shear walls.

The proposed alterations will not impair the existing stability systems.

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4 Cheese Hall

4.1 Second Floor

4.1.1 Second floor joists

Span = 6.4m
Centres = 350mm
Size 50 x 295mm

Loading:
Existing floor loading:
DL (G2 internal floor) = 0.5kN/m^2 x 0.35 = 0.175 kN/m
IL (Q1 storage area) = 2.0kN/m^2 x 0.35 = 0.7 kN/m
New plant loading:
Heat pump ancilliaries, e.g. buffer vessel (150kg), expansion vessels (30kg), pressurisation unit (5kg) and pumps (4No, 2.5kg each) + allowance for pipework and valves.
Spread over area approx. 1.2 x 1.2m = 1.44m^2
Total load = 245 kg = 2.45 kN
2.45 / 1.44 = 1.7 kN/m^2
Therefore loading from new plant is within the design imposed loading of 2.0 kN/m^2.

Calc ref:
PASS C401

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5 Town Hall

5.1 Services in roof space

New ventilation ducts and walkway to be installed within roof space above town hall.

Design imposed load on walkway = 0.6 kN/m^2

Design dead load due to services = 0.2 kN/m^2

4No. primary timber trusses, at approx. 3.0m centres

5.1.1 Ceiling joists supporting new services Ceiling joists 50 x 100 at 400 centres, span between primary truss bottom chords. Span = 3.0m Centres = 400mm Loading: DL (G3 attic floor) = 0.6 x 0.4 = 0.24 kN/m LL = 0.25 kN/m^2 (allowance for storage) = 0.1 kN/m PASS Existing joists can support additional load of services.	Calc ref: C501
5.1.2 Ceiling joists supporting new walkway Loading: DL (G3 attic floor) = 0.6 x 0.4 = 0.24 kN/m LL (Q3 roof space) = 0.6 kN/m^2 x 0.4 = 0.24 kN/m PASS Recommended deflection limit of span/250 is exceeded. Deflection limit relaxed to span/180.	Calc ref: C502

5.2 Plant on roof

New AHU installed on roof of town hall adjacent to existing roof plant enclosure.

Existing roof structure consists of timber joists and steel beam grid:

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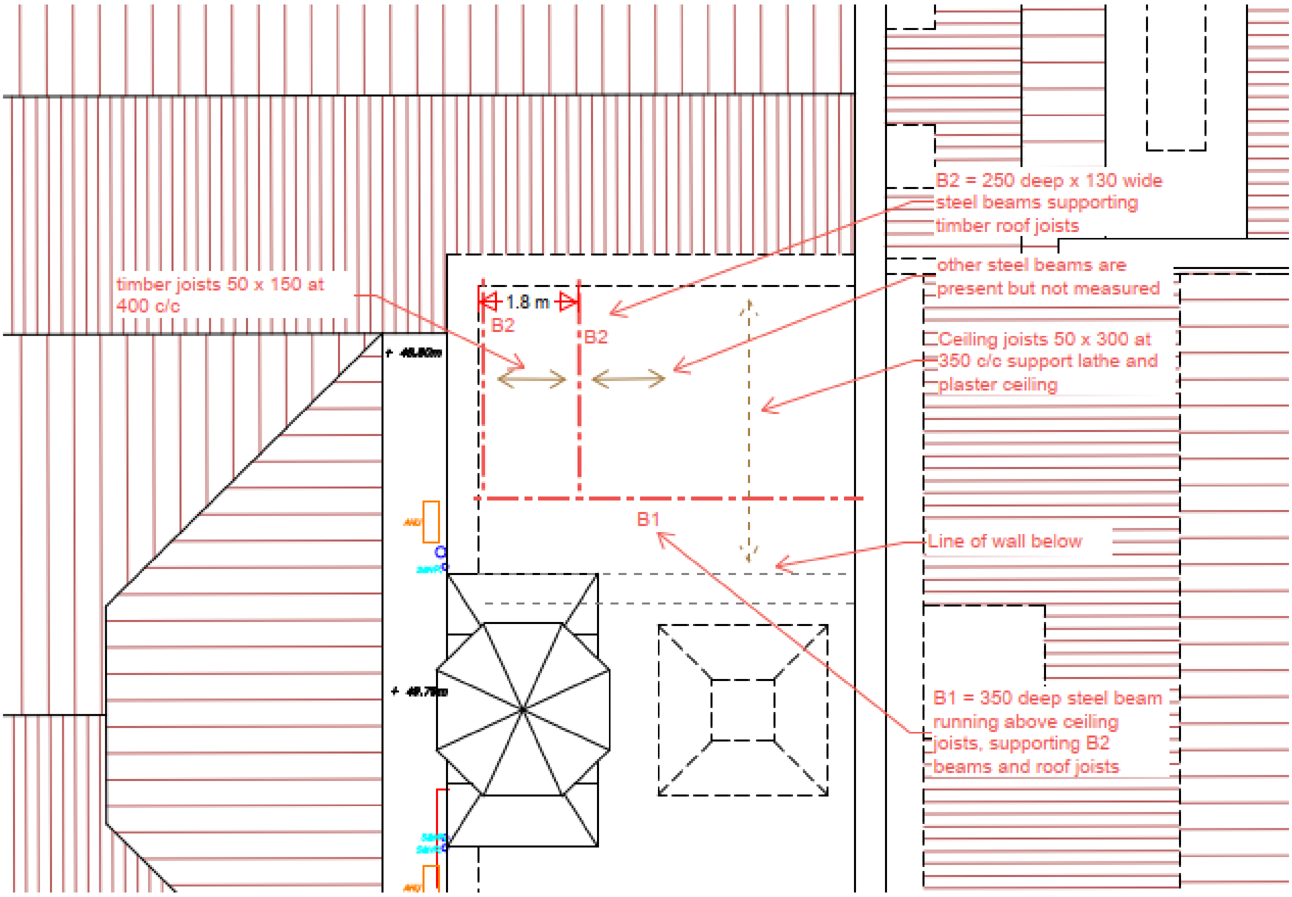

5.2.1 Timber roof joists

Span = 1.8m
Loading:
Loaded width = 0.4m
DL (G1 flat roof) = 0.9 kN/m^2
LL (Q2 roof maintenance) = 0.75 kN/m^2

50 x 150 joists at 400 centres
PASS

Calc ref:
C503

5.2.2 Beam B2

Span = 3.9m
Loading:
Loaded width = 1.8m

Assume 254 x 102 x 22 UB
PASS

Calc ref:
C504

5.2.3 Beam B1

Span = 6.8m
Loading:
Loaded width = 5.2/2 = 2.6m

Assume 356 x 127 x 33 UB
PASS

C505

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The existing roof structure is sufficient to support the proposed new plant loading.

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Appendix 2: Calculation references

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

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

Tedds calculation version 2.2.20

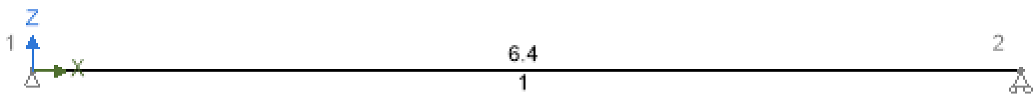
Beam results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm²	2.5	0.7	0.265	PASS
Bending stress	N/mm²	16.2	9.7	0.595	PASS
Shear stress	N/mm²	2.7	0.7	0.245	PASS
Deflection	mm	25.6	25.3	0.987	PASS

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - C24 (EC5) - 50x295

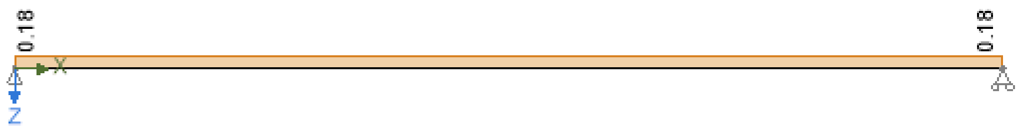


Span	Length (m)	Section	Start Support	End Support
1	6.4	50x295	Pinned	Roller Pin X
50x295: Area 147 cm², Inertia Major 10697 cm⁴, Inertia Minor 307 cm⁴, Shear area parallel to Minor 123 cm², Shear area parallel to Major = 123 cm²				
C24 (EC5): Density 420 kg/m³, Youngs 11 kN/mm², Shear 0.69 kN/mm², Thermal 0 °C⁻¹				

Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	0.18 kN/m

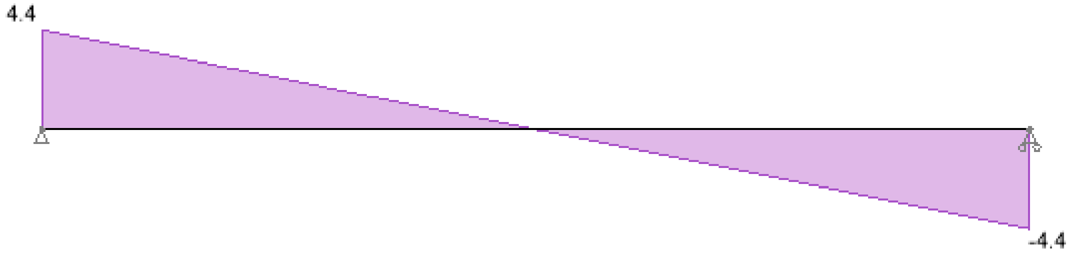
Member	Load case	Load Type	Orientation	Description
Beam	Imposed	UDL	GlobalZ	0.7 kN/m

Results

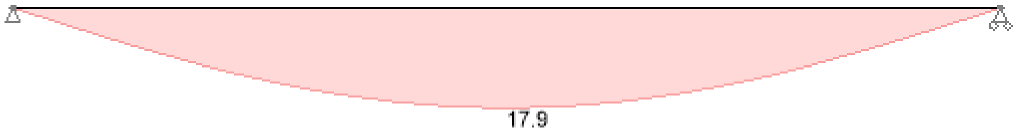
Forces

Strength combinations - Moment envelope (kNm)

Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Beam - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 1

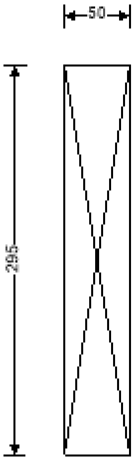
Timber section details

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

Breadth of sections b = **50** mm

Depth of sections h = **295** mm

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



50x295 timber section
Cross-sectional area, A, 14750 mm^2
Section modulus, W_y, 725208.3 mm^3
Section modulus, W_z, 122917 mm^3
Second moment of area, I_y, 106968229 mm^4
Second moment of area, I_z, 3072917 mm^4
Radius of gyration, i_y, 85.2 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length — L_b = **100** mm

<u>Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)</u>

Modification factors

Duration of load and moisture content - Table 3.1 — k_{mod} = **0.8**
Deformation factor - Table 3.2 — k_{def} = **0.6**
Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.67**
Load configuration factor - cl.6.1.5(4) — $k_{c,90}$ = **1.5**
System strength factor - cl.6.6 — k_{sys} = **1.1**

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

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis — $F_{c,y,90,d}$ = **4.378** kN
Effective contact length — $L_{b,ef} = L_b + \min(L_b, 30\text{ mm})$ = **130** mm
Design perpendicular compressive stress - exp.6.4 — $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.674** N/mm^2
Design perpendicular compressive strength — $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.692** N/mm^2
$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.265**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **4.378** kN
Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.665** N/mm^2
Design shear strength — $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.708** N/mm^2
$\tau_{y,d} / f_{v,y,d}$ = **0.245**

PASS - Design shear strength exceeds design shear stress

<u>Check design 3200 mm along span</u>

Check bending moment - Section 6.1.6

Design bending moment — $M_{y,d}$ = **7.006** kNm
Design bending stress — $\sigma_{m,y,d} = M_{y,d} / W_y$ = **9.66** N/mm^2
Design bending strength — $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **16.246** N/mm^2

$\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.595**

PASS - Design bending strength exceeds design bending stress

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design 3200 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **17.9** mm

Quasi-permanent variable load factor ψ_2 = **0.6**

Final deflection with creep $\delta_{y,Final} = 0.2 \times \delta_y \times (1 + k_{def}) + 0.8 \times \delta_y \times (1 + \psi_2 \times k_{def})$ = **25.3** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **25.6** mm

$\delta_{y,Final}$ / $\delta_{y,Allowable}$ = **0.987**

PASS - Allowable deflection exceeds final deflection

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

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

Tedds calculation version 2.2.20

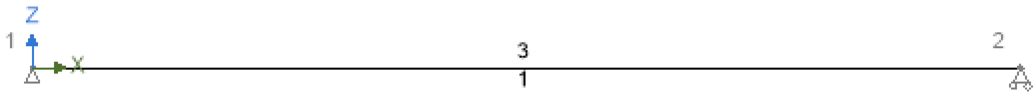
Beam results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm²	2.9	0.1	0.041	PASS
Bending stress	N/mm²	19.8	6.8	0.342	PASS
Shear stress	N/mm²	3.0	0.3	0.111	PASS
Beam stability check				0.342	PASS
Deflection	mm	12	11.0	0.914	PASS

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - C24 (EC5) - 50x100



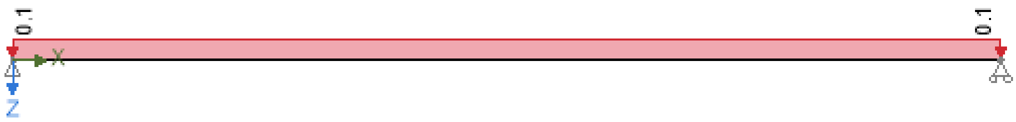
Span	Length (m)	Section	Start Support	End Support
1	3	50x100	Pinned	Roller Pin X
50x100: Area 50 cm², Inertia Major 417 cm⁴, Inertia Minor 104 cm⁴, Shear area parallel to Minor 42 cm², Shear area parallel to Major = 42 cm²				
C24 (EC5): Density 420 kg/m³, Youngs 11 kN/mm², Shear 0.69 kN/mm², Thermal 0 °C⁻¹				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

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	Calcs for C501 ceiling joists - supporting services				Start page no./Revision 2	
	Calcs by RP	Calcs date 08/05/2025	Checked by	Checked date	Approved by	Approved date

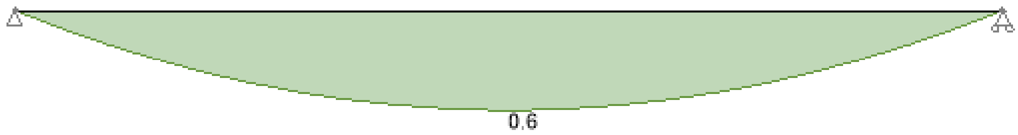
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	0.24 kN/m
Beam	Imposed	UDL	GlobalZ	0.1 kN/m

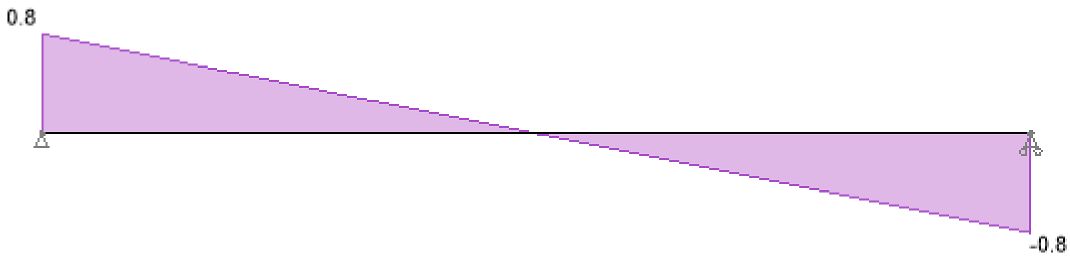
Results

Forces

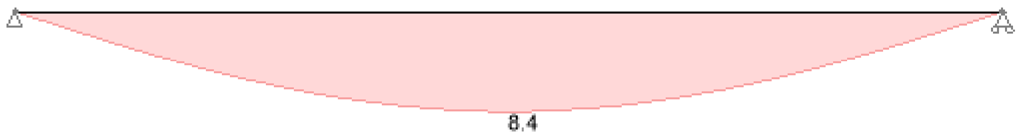
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Beam - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

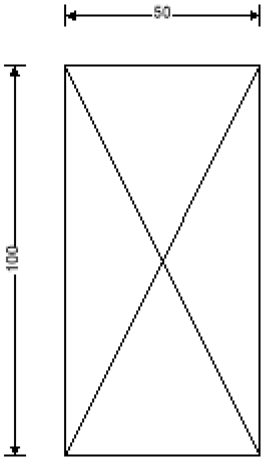
Timber section details

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

Breadth of sections b = **50** mm

Depth of sections h = **100** mm

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



50x100 timber section
Cross-sectional area, A, 5000 mm^2
Section modulus, W_y, 83333.3 mm^3
Section modulus, W_z, 41667 mm^3
Second moment of area, I_y, 4166667 mm^4
Second moment of area, I_z, 1041667 mm^4
Radius of gyration, i_y, 28.9 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m,k}$, 24 N/mm^2
Characteristic shear strength, $f_{v,k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0,05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length — L_b = **100** mm

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Modification factors

Duration of load and moisture content - Table 3.1	k_{mod} = **0.9**
Deformation factor - Table 3.2	k_{def} = **0.6**
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**
Load configuration factor - cl.6.1.5(4)	$k_{c,90}$ = **1.5**
System strength factor - cl.6.6	k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d}$ = **0.753** kN
Effective contact length	$L_{b,ef} = L_b + \min(L_b, 30\text{ mm})$ = **130** mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.116** N/mm^2
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.904** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.041**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **0.753** kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.337** N/mm^2
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **3.046** N/mm^2
	$\tau_{y,d} / f_{v,y,d}$ = **0.111**

PASS - Design shear strength exceeds design shear stress

Check design 1500 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **0.565** kNm
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y$ = **6.774** N/mm^2

Design bending strength $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **19.821** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.342**

PASS - Design bending strength exceeds design bending stress

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

Effective length - Table 6.1 $L_{ef} = 0.9 \times 3000$ mm $+ 2 \times h$ = **2900** mm

Critical bending stress - exp.6.32 $\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef})$ = **49.759** N/mm^2

Relative slenderness for bending - exp.6.30 $\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})}$ = **0.694**

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Beam stability check - exp.6.33 $\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d})$ = **0.342**

PASS - Beam stability is acceptable

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design 1500 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **8.4** mm

Quasi-permanent variable load factor ψ_2 = **0**

Final deflection with creep $\delta_{y,Final} = 0.5 \times \delta_y \times (1 + k_{def}) + 0.5 \times \delta_y \times (1 + \psi_2 \times k_{def})$ = **11** mm

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

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.914**

PASS - Allowable deflection exceeds final deflection

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

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

Tedds calculation version 2.2.20

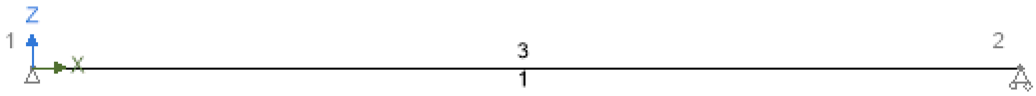
Beam results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm²	2.9	0.2	0.058	PASS
Bending stress	N/mm²	19.8	9.6	0.485	PASS
Shear stress	N/mm²	3.0	0.5	0.157	PASS
Beam stability check				0.485	PASS
Deflection	mm	16.7	15.2	0.914	PASS

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - C24 (EC5) - 50x100



Span	Length (m)	Section	Start Support	End Support
1	3	50x100	Pinned	Roller Pin X
50x100: Area 50 cm², Inertia Major 417 cm⁴, Inertia Minor 104 cm⁴, Shear area parallel to Minor 42 cm², Shear area parallel to Major = 42 cm²				
C24 (EC5): Density 420 kg/m³, Youngs 11 kN/mm², Shear 0.69 kN/mm², Thermal 0 °C⁻¹				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

Giraffe Engineering 5 Union Road Chippenham SN15 1HW	Project Chippenham Town Hall				Job no. 1663GE	
	Calcs for C502 ceiling joists - supporting walkway				Start page no./Revision 2	
	Calcs by RP	Calcs date 08/05/2025	Checked by	Checked date	Approved by	Approved date

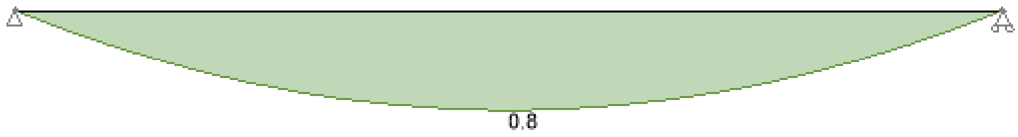
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	0.24 kN/m
Beam	Imposed	UDL	GlobalZ	0.24 kN/m

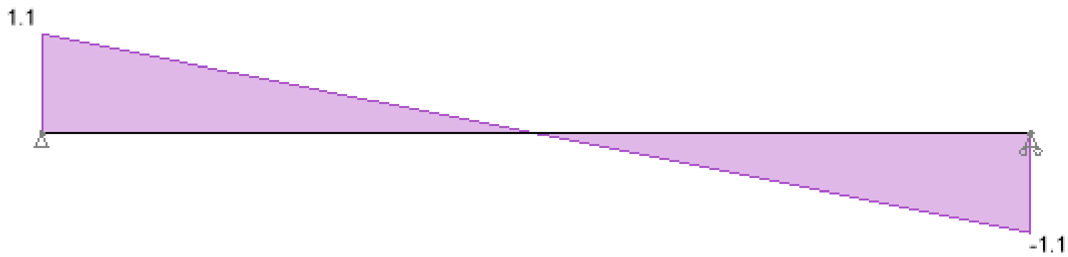
Results

Forces

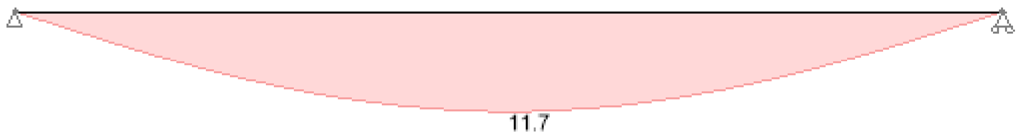
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Beam - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2 Short-term

Service class - cl.2.3.1.3 1

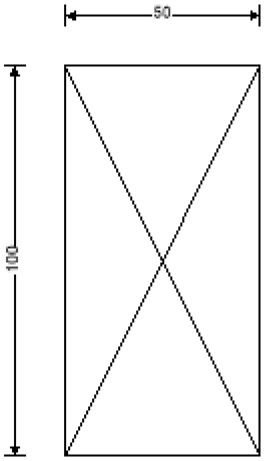
Timber section details

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

Breadth of sections b = **50** mm

Depth of sections h = **100** mm

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



50x100 timber section
Cross-sectional area, A, 5000 mm^2
Section modulus, W_y, 83333.3 mm^3
Section modulus, W_z, 41667 mm^3
Second moment of area, I_y, 4166667 mm^4
Second moment of area, I_z, 1041667 mm^4
Radius of gyration, i_y, 28.9 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m.k}$, 24 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length — L_b = **100** mm

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 — k_{mod} = **0.9**
Deformation factor - Table 3.2 — k_{def} = **0.6**
Depth factor for bending - Major axis - exp.3.1 — $k_{h,m,y} = \min((150 \text{ mm} / h)^{0.2}, 1.3)$ = **1.084**
Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.67**
Load configuration factor - cl.6.1.5(4) — $k_{c,90}$ = **1.5**
System strength factor - cl.6.6 — k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis — $F_{c,y,90,d}$ = **1.068** kN
Effective contact length — $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm})$ = **130** mm
Design perpendicular compressive stress - exp.6.4 — $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.164** N/mm^2
Design perpendicular compressive strength — $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.904** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.058**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **1.068** kN
Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.478** N/mm^2
Design shear strength — $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **3.046** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.157**

PASS - Design shear strength exceeds design shear stress

Check design 1500 mm along span

Check bending moment - Section 6.1.6

Design bending moment — $M_{y,d}$ = **0.801** kNm
Design bending stress — $\sigma_{m,y,d} = M_{y,d} / W_y$ = **9.609** N/mm^2

Design bending strength $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M = \mathbf{19.821}$ N/mm^2

$\sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.485}$

PASS - Design bending strength exceeds design bending stress

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

Effective length - Table 6.1 $L_{ef} = 0.9 \times 3000 \text{ mm} + 2 \times h = \mathbf{2900}$ mm

Critical bending stress - exp.6.32 $\sigma_{m,crit} = 0.78 \times b^2 \times E_{0.05} / (h \times L_{ef}) = \mathbf{49.759}$ N/mm^2

Relative slenderness for bending - exp.6.30 $\lambda_{rel,m} = \sqrt{(f_{m.k} / \sigma_{m,crit})} = \mathbf{0.694}$

Lateral buckling factor - exp.6.34 $k_{crit} = \mathbf{1.000}$

Beam stability check - exp.6.33 $\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = \mathbf{0.485}$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design 1500 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = \mathbf{11.7}$ mm

Quasi-permanent variable load factor $\psi_2 = \mathbf{0}$

Final deflection with creep $\delta_{y,Final} = 0.5 \times \delta_y \times (1 + k_{def}) + 0.5 \times \delta_y \times (1 + \psi_2 \times k_{def}) = \mathbf{15.2}$ mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 180 = \mathbf{16.7}$ mm

$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.914}$

PASS - Allowable deflection exceeds final deflection

	Job Title	Job No.	Page No.	Rev.
GIRAFFE ENGINEERING	Chippenham Town Hall	1663GE		C1
	Calculations/Sketch Title	Date	Author	Checked
	C503 Flat roof joists	12/06/25	RP	-

CALCULATION SHEET

Simply Supported Timber Beam:

RESULT: **PASS**

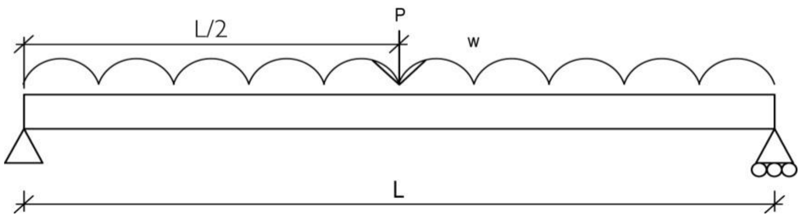
17%

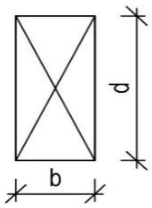
BS EN 1995-1-1

Member Description:

Flat roof joists

Beam Properties:





Span, L =	1.8	m
Width, b =	50	mm
Height, h =	150	mm
Section modulus, W_y =	*1.88E+05*	mm^3
Second Moment of Area, I_{yy} =	*1.41E+07*	N/mm^4

Material:	Softwood
Grade:	C16
Service-Class:	1
Load sharing?	Yes

Modulus of Elasticity, $E_{0,mean}$ =	***8000***	N/mm^2	BS EN 338
Charac. bending strength, $f_{m,y,k}$ =	***16***	N/mm^2	
Charac. shear strength, $f_{v,k}$ =	***3.2***	N/mm2	
Charac. perp. comp. strength, $f_{c,90,k}$ =	***2.2***	N/mm2	

Supports:

Bearing width, b_2 =	50	mm
Bearing length, l =	100	mm

Applied Loading:

Load Duration: Short-term

k_{def} = *0.6*

Major Axis Loading:

Width spanning to beam, W = 0.40 m

	UDL Loads: *kN/m²*	*w, kN/m*	Point Loads: *P, kN*	ψ_0	ψ_2
Dead load, g_k	0.9	0.36	0	-	-
Primary live load, $q_{k,1}$	0.75	0.3	0	-	0
Secondary live load, $q_{k,2}$	0	0	0	0.5	0

Load Case:		UDL Loads: *w, kN/m*	Point Loads: *P, kN*	
SLS	Inst	*0.66*	0.00	§2.2.3
	Creep	*0.22*	0.00	TABLE 3.2
	Final	*0.88*	0.00	§7
ULS		*0.94*	0.00	

CALCULATION SHEET

Serviceability Limit State:

Deflection:

Deflection Limits:

	INSTANT	FINAL		
span/	*360*	*250*		TABLE NA.5
Absolute limit:	-	-	mm	

Major Axis:

	INSTANT	FINAL	
Deflection, δ	**0.8**	**1.1**	mm
Allowable deflection, δ_{allow}	***5.0***	***7.2***	mm
	PASS	**PASS**	
	16%	*15%*	

Vibration:

Frequency: Check vibration? No

Floors must have a frequency greater than 8 Hz.

Otherwise special investigation must be made.

ISructE Manual

$$f_1 = \frac{18}{\sqrt{w_{inst}}}$$

w_{inst} is the instantaneous bending deflection of the floor under dead weight only.

w_{inst} assumes the floor is spanning in one direction simply supported.

$$w_{inst} = \frac{5w_{gk}L^4}{384EI}$$

Instantaneous dead load deflection, w_{inst} =	0.44	mm
Allowable frequency =	-	Hz
Frequency, f_1 =	-	Hz
	PASS	

Static Deflection under Point Load:

Amplification factor, k_{amp} =	*1.05*	
Proportion of load acting on single joist, k_{dist} =	*0.30*	
Deflection under 1kN midspan point load, $w_{inst,Q}$ =	-	mm
Allowable deflection, a =	-	mm
	PASS	

Unit Impulse Velocity Response:

For typical timber floors, this check is not required.

IStructE Manual

Job Title	Job No.	Page No.	Rev.
Chippenham Town Hall	1663GE		C1
Calculations/Sketch Title	Date	Author	Checked
C503 Flat roof joists	12/06/25	RP	-

CALCULATION SHEET

Ultimate Limit State:

Is beam subject to lateral torsional buckling? No

Bending: §6.1.6

$M_{y,Ed} = w_{ULS} \cdot L^2/8$

Design moment, $M_{y,Ed}$ = ***0.38*** kNm

$\sigma_{m,y,d} = M_{y,Ed}/W_y$

Partial safety factor, γ_M = *1.3* TABLE 2.3

k_{mod} = *0.9* TABLE 3.1

k_{sys} = *1.1*

k_h = *1.00* §3.3

$f_{m,y,d} = k_{crit} \cdot k_{mod} \cdot k_h \cdot k_{sys} \cdot f_{m,y,k}$

k_{crit} = *1.00*

Design bending stress, $\sigma_{m,y,d}$ = **2.02** N/mm^2

Design bending strength, $f_{m,y,d}$ = **12.18** N/mm^2

PASS

17%

Shear: §6.1.7

$V_{Ed} = w_{ULS} \cdot L/2$

Shear force, V_{Ed} = ***0.84*** kN

$\tau_{v,d} = 3/2\,(V_{Ed}/bh_v)$

k_{cr} = *0.67* TABLE NA.4

$f_{v,d} = k_{mod} \cdot k_{cr} \cdot f_{v,k}/\gamma_M$

Design shear stress, $\tau_{v,d}$ = **0.17** N/mm2

Design shear strength, $f_{v,d}$ = **1.48** N/mm2

PASS

11%

Bearing: §6.1.5

Design Compressive Load, $F_{c,d}$ = ***0.84*** kN

$\sigma_{c,90,d} = F_{c,90,d}/b_2 \cdot l_{ef}$

Simply supported beam with UDL:

$k_{c,90}$ = *1.5* PD 6693-1 §8

Effective bearing length, l_{ef} = *100* mm

$f_{c,90,d} = k_{mod} \cdot k_{c,90} \cdot f_{c,90,k}/\gamma_M$

Design compressive stress, $\sigma_{c,90,d}$ = **0.17** N/mm2

Design compressive strength, $f_{c,90,d}$ = **2.28** N/mm2

PASS

7%

STEEL MEMBER ANALYSIS & DESIGN (EN1993)

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

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

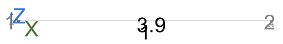
Tedds calculation version 4.4.06

ANALYSIS

Tedds calculation version 1.0.35

Geometry

Geometry (m) - Steel (EC3) - UKB 203x102x23



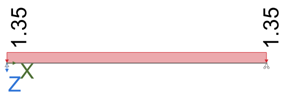
Span	Length (m)	Section	Start Support	End Support
1	3.9	UKB 203x102x23	Pinned	Roller Pin X
UKB 203x102x23: Area 29 cm^2, Inertia Major 2105 cm^4, Inertia Minor 164 cm^4, Shear area parallel to Minor 11 cm^2, Shear area parallel to Major = 17 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

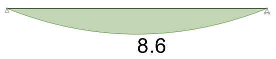
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	1.62 kN/m
Beam	Imposed	UDL	GlobalZ	1.35 kN/m

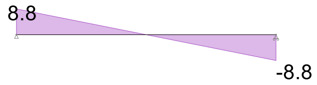
Results

Forces

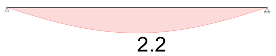
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1**
Resistance of members to instability	γ_{M1} = **1**
Resistance of tensile members to fracture	γ_{M2} = **1.1**

Beam design

Section details

Section type	UKB 203x102x23 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **9.3** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

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

Width of section — $c = d = \mathbf{169.4}$ mm

$c / t_w = 31.4 = 38.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{40.6}$ mm

$c / t_f = 4.4 = 5.4 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

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

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{184.6}$ mm — $\eta = \mathbf{1.000}$

$h_w / t_w = 34.2 = 42 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} = \mathbf{8.8}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1238}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \mathbf{253.7}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.035}$

PASS - Design shear resistance exceeds design shear force

<u>Check design 1950 mm along span</u>

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} = \mathbf{8.6}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{83.1}$ kNm

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.103}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **3900** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **47.1** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.328**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.319**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.509**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.987**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.516**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **42.9** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.2**

PASS - Design buckling resistance moment exceeds design bending moment

<u>Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)</u>

<u>Check design 1950 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection δ_y = **2.2** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 360$ = **10.8** mm

$\delta_y / \delta_{y,Allowable}$ = **0.207**

PASS - Allowable deflection exceeds design deflection

STEEL MEMBER ANALYSIS & DESIGN (EN1993)

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

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

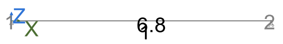
Tedds calculation version 4.4.06

ANALYSIS

Tedds calculation version 1.0.35

Geometry

Geometry (m) - Steel (EC3) - UKB 356x127x33



Span	Length (m)	Section	Start Support	End Support
1	6.8	UKB 356x127x33	Pinned	Roller Pin X
UKB 356x127x33: Area 42 cm^2, Inertia Major 8249 cm^4, Inertia Minor 280 cm^4, Shear area parallel to Minor 21 cm^2, Shear area parallel to Major = 19 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

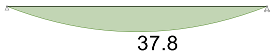
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	2.3 kN/m
Beam	Imposed	UDL	GlobalZ	2 kN/m

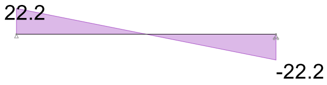
Results

Forces

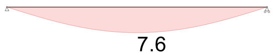
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Beam design

Section details

Section type UKB 356x127x33 (Tata Steel Advance)

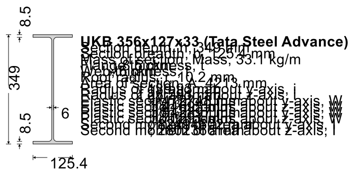
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **8.5** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

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

Width of section — $c = d = \mathbf{311.6}$ mm

$c / t_w = 51.9 = 63.8 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{49.5}$ mm

$c / t_f = 5.8 = 7.2 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

__**Check design at start of span**__

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{332}$ mm — $\eta = \mathbf{1.000}$

$h_w / t_w = 55.3 = 68 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} = \mathbf{22.2}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2306}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \mathbf{472.6}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.047}$

PASS - Design shear resistance exceeds design shear force

__**Check design 3400 mm along span**__

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} = \mathbf{37.8}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{192.7}$ kNm

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.196}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **6800** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **41.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **2.153**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 c

Imperfection factor - Table 6.3 α_{LT} = **0.49**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **2.668**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.216**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.216**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **41.6** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.91**

PASS - Design buckling resistance moment exceeds design bending moment

<u>Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)</u>

<u>Check design 3400 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection δ_y = **7.6** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 360$ = **18.9** mm

$\delta_y / \delta_{y,Allowable}$ = **0.402**

PASS - Allowable deflection exceeds design deflection