

Sutcliff Consultants Ltd

Crossbush Cottage, 11 Crossbush Road
Felpham, West Sussex PO22 7LS

Site: Langoustine Cottage, 18 Kingsey Ave. Emsworth
Job: Garden Room
Job number: 2040

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EuroBeam Project Summary

Project started 17 Nov 2020 AHS
Site address: Langoustine Cottage, 18 Kingsey Ave. Emsworth
Job: Garden Room
Client: RJCA / Pearce
Job number: 2040

ITEMS:

- 1: User text page: D:\OneDrive\SDA\1-General Notes.txt
First line: GENERAL NOTES
- 2: User text page: D:\OneDrive\SDA\2-Standard loading data.txt
First line: STANDARD LOADING DATA
- 3: Beam: R1 - Common Rafters (Worst Case)
Span: 2.0 m.
Reactions (unfactored/factored): R1: 0.50/0.67 kN R2: 0.50/0.67 kN
Use 47 x 97 C16
- 4: Beam: PU1 - Purlin (Worst Case)
Span: 3.4 m.
Reactions (unfactored/factored): R1: 6.72/9.10 kN R2: 6.72/9.10 kN
Use 150 x 195 C24
- 5: Beam: PU2 - Purlins onto french doors
Span: 2.4 m.
Reactions (unfactored/factored): R1: 3.50/4.73 kN R2: 3.50/4.73 kN
Use 150 x 195 C24
- 6: Beam: PU3 - Purlin over side french doors
Span: 3.4 m.
Reactions (unfactored/factored): R1: 4.96/6.70 kN R2: 4.96/6.70 kN
Use 150 x 195 C24
- 7: Beam: PU4 - Purlins over shower room and store
Span: 2.4 m.
Reactions (unfactored/factored): R1: 4.74/6.42 kN R2: 4.74/6.42 kN
Use 150 x 195 C24
- 8: Beam: B1 - Over games room
Span: 3.9 m.
Reactions (unfactored/factored): R1: 10.62/14.31 kN R2: 9.24/12.43 kN
Use 150 x 195 D40
- 9: Beam: B2 - Central eaves beam over side veranda
Span: 3.5 m.
Reactions (unfactored/factored): R1: 3.58/4.86 kN R2: 3.58/4.86 kN
Use 150 x 150 D30

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10: Beam: B3 - End eaves beam over side verranda
Span: 2.5 m.
Reactions (unfactored/factored): R1: 2.56/3.47 kN R2: 2.56/3.47 kN
Use 150 x 150 D30

11: Beam: GB1 - Ground beam beneath cavity walls (Worst Case)
Span: 3.7 m.
Reactions (unfactored/factored): R1: 36.43/47.87 kN R2: 36.43/47.87 kN
Section: 400 x 250 mm RC, C25/30 concrete
Reinforcement: 4 No H16
H6 links at 140 mm max c/s

12: Beam: GB2 - Ground beam beneath Deck perimeters (Worst Case)
Span: 3.7 m.
Reactions (unfactored/factored): R1: 9.91/12.96 kN R2: 9.91/12.96 kN
Section: 400 x 250 mm RC, C25/30 concrete
Reinforcement: 2 No H12
H8 links at 140 mm max c/s

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GENERAL NOTES

These Calculations and details are to be read in conjunction with all relevant Architect's & Engineer's drawings & specifications.

The following calculations have been prepared for Building Regulations Approval purposes only and are not considered to constitute working details.

Any Work undertaken prior to Building Regulations Approval is done so entirely at the Client/Contractors own risk.

Provide Double Joists under Dormer/Return studwork and Partitions parallel to spans.

Provide Triple Joists under Dormer cheeks

Solid blocking to be fitted tight between all floor joists where spans exceed 3m & at not more than 2.5m intervals.

Unless stated otherwise, it has been assumed that solid internal walls are load-bearing.

Foundations to Existing Walls assumed to be load bearing are to be exposed on site and suitability to be agreed upon with the building inspector. All new foundations are to be founded off natural virgin ground.

Existing Walls, Beams, Lintels Etc. to be exposed on site where required by the building inspector and suitability agreed upon.

If found to be unsatisfactory, replacements are to be designed & agreed with the building inspector before works are carried out.

It is assumed that any roof trusses and their bracing members on the project will be supplied by a Specialist Sub-Contractor, who will prepare all relevant calculations and details for submission to the inspecting Authority.

Trusses should be fixed to wall plates using an appropriate truss clip and not by skew nailing.

Any precast concrete floors are to be by Specialist Sub-Contractor, who will provide all relevant calculations and details for Inspectors approval.

Any balustrade and metal staircases are to be by Specialist Sub-Contractor, who will provide all relevant calculations and details for inspecting Authority approval.

30 x 5.0mm thick galvanised mild steel lateral restraint straps, 900mm long with 150mm upstand, are to be provided between external walls and floors at 1200mm max. centres. Where straps are perpendicular to the span of the floor, they shall be taken across 3 No. joists, with solid timber noggins between. The strap's upstand shall be fixed to the solid masonry wall using 5 No. x No. 12 screws in plugged holes.

Timber wall plates are to be held down using 30 x 2.5mm thick galvanised mild steel vertical restraint straps, 900mm long with 100mm flat return, at 2000mm max. Centres (but either side of wide windows). The strap should be fixed to the solid masonry wall using 5 No. x No. 12 screws in plugged holes.

Movement joints are to be provided in accordance with masonry manufacturer's recommendations (approximately 6m centres in blockwork and 12m centres in brickwork)

Existing Construction

Any deviation from the design assumptions is to be reported to the Architect/Engineer prior to commencement of work. The existing building is assumed to be in good structural order and any defects are to be reported to the Architect/Engineer. The temporary stability of the structure during all stages of the construction work is the responsibility of the Contractor.

New Works

All new steelwork is to be Grade S355, unless noted otherwise in the calculations. Normal paint finish to be thoroughly wire brushed and painted with two coats of Zinc Phosphate primer. All damaged areas of paintwork are to be touched up after erection of the steelwork. All bolts are to be Grade 8.8, unless noted otherwise, and all welds are to be 6mm continuous fillet type, unless noted otherwise.

All welds to cranked beams to be full-thickness, full strength welds.

If connection is steel beam to column below, then stitch weld column to underside of beam above.

For steel bearing onto timber post, then angle cleat to be either welded or bolted to underside of steel beam & bolted/screwed to post.

Timber bearing onto timber posts to be secured with either butt straps to side or screwed steel angle cleat beneath. All posts / columns to be located centrally beneath the beam above.

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Concrete grades are to be as noted within the calculations and placed, compacted and protected in accordance with BS 8110.

Any dormers are to be clad externally with 12.5mm thick exterior quality ply, fixed at 150mm centres with No. 8 galvanised screws, centres closed to 75mm around all external openings.

All new masonry piers, walls, etc. are to be built as noted within the calculations or to the Architect's/ Engineer's specifications.

It is brought to the contractors attention that to comply with the current Health and Safety Regulations for both temporary and permanent works specialized plant / equipment /lifting gear may be required to install/construct some elements of the structure.

The property owner is reminded of the requirements to serve a Party Structure Notice on any owners of any buildings that might be affected under the requirements of the Part Wall Etc. Act 1996.

NB! When undertaking alteration works to a trussed rafter roof structure, all new beams, rafters, floor joists etc are to be installed prior to the removal of any of the trussed rafter members unless an alternative method of support to the roof is provided.

NB! The Contractor is responsible for maintaining the stability of the structure through out the course of the works. The Contractor is to satisfy himself as to the condition of the structure prior to commencing any works, and to employ a suitably qualified person to advise accordingly. The Contractor is responsible for the design and installation of all temporary works. The Contractor is to employ a temporary works Engineer to design temporary works and bracing. The existing structure has been assumed to be in a sound stable condition in a good state of repair, this is to be confirmed prior to commencing any works.

**IMPORTANT NOTICE FOR THE INFORMATION OF ALL DUTY HOLDERS UNDER THE CDM 2015 REGULATIONS
STRUCTURAL ENGINEER / DESIGNER FORESEABLE RISKS AND HAZARDS NOTED AT THE REAR
OF THIS DOCUMENT**

**CLIENT DUTY AND RESPONSIBILITY INFORMATION CAN BE FOUND AT THE CITB WEBSITE
PRINCIPAL CONTRACTORS MAY WISH TO DOWNLOAD AND USE CDM WIZARD APP TO ASSIST
THEM WITH THEIR DUTIES**

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Standard Loadings used (BS 6399)(kN/m ²)	STANDARD LOADING DATA	
	Separate	Combined value
Floor dead load (with ceiling below):	0.50	
Floor dead load (with NO ceiling below):	0.40	
Floor dead load (precast concrete (with ceiling below):	2.50	
Floor live load (Domestic):	1.50	2.0
Floor live load (Office):	2.50	3.0
Floor live load (Commercial / Filing):	4.50	5.0
Ceiling dead load	0.40	
Ceiling live load:	0.75	1.15
Ceiling live load (No access):	0.25	0.65
Pitched roof dead load (Tiles) (along line of slope)	0.76	
Pitched roof dead load (Slates) (along line of slope)	0.60	
Pitched roof live load (allowance for pitch to be made)	0.75	1.50
Flat roof dead load	0.75	
Flat roof live load	0.75	1.50
If site greater than 100m AOD then live load	1.0	1.75
N.B. Joisted flat roofs also need to be checked to ensure that an individual joist can take a 900N (0.9kN) point load at mid-span with no other live load.		
Dormer cheeks & faces	1.0	
Double-glazed window units	0.50	
Double-glazed Bifold Doors	0.50	
Stud partitions	0.40	
Stud partitions (Lath & Plaster)	0.60	
Cavity walls:		
Inner leaf (dense)	2.20	
Inner leaf (lightweight)	0.75	
Render & set	0.30	
Outer leaf	2.20	3.25 - 4.70
Solid brick walls for every 100mm thickness:	2.20	
External handrail lateral load	0.74	
Internal handrail lateral load	0.36	

Dimensions given for beams are clear span & extra length needs to be added to allow for bearings of minimum 100mm at each end or to mid-depth of padstones if greater.

Rafter & hip spans are plan lengths (run of rafter)

Always check dimensions on site before ordering materials.

Unless otherwise stated R1 is taken as the reaction at the outer wall or failing that the LEFT hand end of the beam when viewed from the front of the property

Formulae in the loading column of the calculations are entered in the sequence

Loading (kN/m²) x effective span or joist spacing if required (m) x Height of load (m)

Calculation of masonry strength under bearing plates using BS5628 Part 1

Normal standards of manufacturing and construction control; $\gamma_m = 3.5$ (Table 4)

100mm wide bearing plates under beams built into party walls fall within

Class 2 limits where plate is less than 800mm (8x) long

Sizes given for members in these calculations are minimum sizes. Consideration should be given to standardising member sizes to the deepest section in a group of elements in order to give even floor / ceiling levels for instance. Unless specifically mentioned otherwise, all timber is to be grade C24. All structural timber is to be suitably treated against decay and insect attack.

Where timber elements are to be left exposed the dimension of the element is to be increased by a minimum of 25mm for each exposed face to allow for charring in the event of a fire. Alternatively an approved fire-retardant finish can be applied in agreement with the building inspector.

Padstone Sizes are minimum dimensions & should, where necessary, be increased to the nearest standard size available.

Minimum bearing onto 330mm long padstone where parallel to span = 180mm.

Minimum bearing onto 450mm long padstone where parallel to span = 225mm.

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Minimum padstone thickness 225mm long = 150mm.
Minimum padstone thickness 330mm long = 150mm.
Minimum padstone thickness 450mm long = 220mm.
Small Padstone alternative 225x100x150 = 2 x engineering brick.

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Beam: R1 - Common Rafters (Worst Case)

Span: 2.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	Roof_Dead	0.75(1/cos25)*0.6	0		L	0.50	0.50
Total load (unfactored): 0.99 kN						0.50	0.50
Dead/Permanent (unfactored): 0.99 kN						0.50	0.50
Live/Variable (unfactored): 0.00 kN						0.00	0.00
(6.10a): 1.34 kN						0.67	0.67
(6.10b): 1.24 kN						0.62	0.62

Load types: U:UDL; Load positions are measured in m. from R1; Load durations: G: Dead

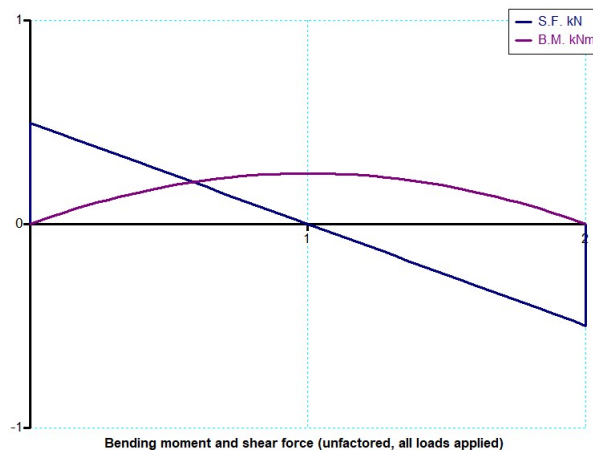
Maximum B.M. = 0.336 kNm (6.10a) at 1.00 m. from R1

Maximum S.F. = 0.671 kN (6.10a) at R1

Mid-span deflections: Dead: $0.10 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $0.00 \times 10^8/EI$

Total: $0.10 \times 10^8/EI$



Timber beam calculation to BS EN1995-1-1 using C16 timber

Use 47 x 97 C16 1.7 kg/m approx

$W_{el,y} = 73.70 \text{ cm}^3$ $I_y = 357 \text{ cm}^4$

Timber grade: C16

Grade bending strength, $f_{m,k} = 16.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 3.2 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 5,400 \text{ N/mm}^2$; $E_{0,mean} = 8,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $\gamma_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/97)^{0.2} = 1.09$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / \gamma_M = 16.0 \times 0.80 \times 1.09 \times 1.1 / 1.30 = 11.82 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 0.34 \times 1000 / 73.7 = 4.55 \text{ N/mm}^2$ OK

Bending resistance = $11.82 \times 73.7 / 1000 = 0.871 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 47 = 31.5 \text{ mm}$. [A1:2008 (6.13a)]

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Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 3.20 \times 0.80 \times 1.1 / 1.30 = 2.17 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 0.671 \times 1000 \times (3/2) / (31.5 \times 97) = 0.33 \text{ N/mm}^2$ OK

Shear resistance = $2.17 \times 31.5 \times 97 \times (2/3) / 1000 = 4.41 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 8.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 0.249 \times 10^6 / ((8,000/16) \times 47 \times 97) = 0.13 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	0.10	3.62	1.00	0.80	6.52	
Shear deflection:		0.13			0.24	
Total mm.		<u>3.75</u>			<u>6.76</u>	OK

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Beam: PU1 - Purlin (Worst Case)

Span: 3.4 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.26	0.26
U G	Roof_Dead	$0.75(1/\cos 25) \cdot 2.2$	0		L	3.10	3.10
U G	Vaulted Clg ins. & P.B.	$0.15 \cdot 2.2$	0		L	0.56	0.56
U QS	Roof_Live	$0.75 \cdot 2.2$	0		L	2.81	2.81
Total load (unfactored): 13.44 kN						6.72	6.72
Dead/Permanent (unfactored): 7.83 kN						3.92	3.92
Live/Variable (unfactored): 5.61 kN						2.81	2.81
(6.10a): 14.78 kN						7.39	7.39
(6.10b): 18.19 kN						9.10	9.10

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed:

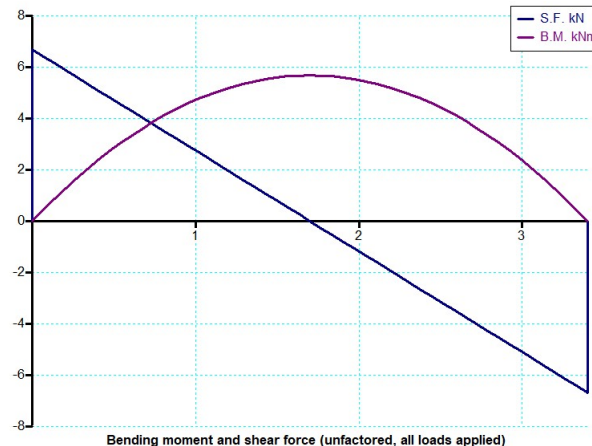
Maximum B.M. = 7.73 kNm (6.10b) at 1.70 m. from R1

Maximum S.F. = 9.10 kN (6.10b) at R1

Mid-span deflections: Dead: $4.01 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $2.87 \times 10^8 / EI$

Total: $6.88 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using C24 timber

Use **150 x 195 C24** 12.3 kg/m approx

$W_{el,y} = 950.6 \text{ cm}^3$ $I_y = 9,269 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 14.77 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 7.73 \times 1000 / 951 = 8.13 \text{ N/mm}^2$ OK

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Bending resistance = $14.77 \times 951/1000 = 14.04$ kNm

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5$ mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46$ N/mm²

Design shear stress, $s_{v,y,d} = 9.10 \times 1000 \times (3/2)/(100.5 \times 195) = 0.70$ N/mm² OK

Shear resistance = $2.46 \times 100.5 \times 195 \times (2/3)/1000 = 32.16$ kN

Deflection

Final deflection limit = $L/250 = 13.60$ mm

Final deflection, $u_{fin} = S u_{inst}(1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$$u_{inst,v} = 1.2 \times 5.71 \times 10^6 / ((11,000/16) \times 150 \times 195) = 0.34 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	4.01	3.93	1.00	0.80	7.07	
Live QS:	2.87	2.82	0.00	0.80	2.82	
Shear deflection:		0.34			0.50	
Total mm.		<u>7.09</u>			<u>10.39</u>	OK

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Beam: PU2 - Purlins onto french doors

Span: 2.4 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.18	0.18
U G	Roof_Dead	$0.75(1/\cos 25) \cdot 1.6$	0		L	1.59	1.59
U G	Vaulted Clg ins. & P.B.	$0.15 \cdot 1.6$	0		L	0.29	0.29
U QS	Roof_Live	$0.75 \cdot 1.6$	0		L	1.44	1.44
Total load (unfactored): 7.00 kN						3.50	3.50
Dead/Permanent (unfactored): 4.12 kN						2.06	2.06
Live/Variable (unfactored): 2.88 kN						1.44	1.44
(6.10a): 7.72 kN						3.86	3.86
(6.10b): 9.46 kN						4.73	4.73

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed:

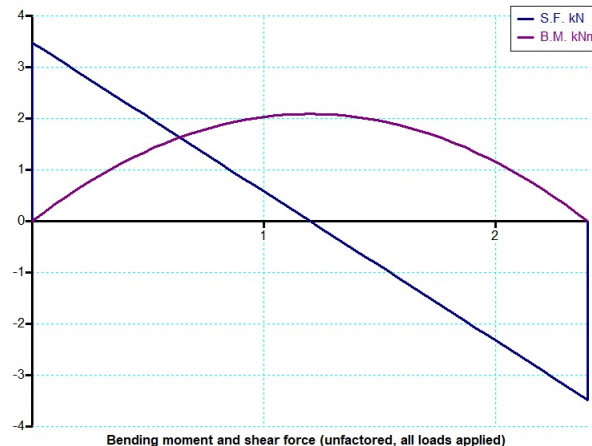
Maximum B.M. = 2.84 kNm (6.10b) at 1.20 m. from R1

Maximum S.F. = 4.73 kN (6.10b) at R1

Mid-span deflections: Dead: $0.74 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.52 \times 10^8 / EI$

Total: $1.26 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using C24 timber

Use **150 x 195 C24** 12.3 kg/m approx

$W_{el,y} = 950.6 \text{ cm}^3$ $I_y = 9,269 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 14.77 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.84 \times 1000 / 951 = 2.99 \text{ N/mm}^2$ OK

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Bending resistance = $14.77 \times 951/1000 = 14.04$ kNm

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5$ mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46$ N/mm²

Design shear stress, $s_{v,y,d} = 4.73 \times 1000 \times (3/2)/(100.5 \times 195) = 0.36$ N/mm² OK

Shear resistance = $2.46 \times 100.5 \times 195 \times (2/3)/1000 = 32.16$ kN

Deflection

Final deflection limit = $L/250 = 9.60$ mm

Final deflection, $u_{fin} = S u_{inst}(1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$$u_{inst,v} = 1.2 \times 2.10 \times 10^6 / ((11,000/16) \times 150 \times 195) = 0.13 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	0.74	0.73	1.00	0.80	1.31	
Live QS:	0.52	0.51	0.00	0.80	0.51	
Shear deflection:		0.13			0.18	
Total mm.		<u>1.36</u>			<u>2.00</u>	OK

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Beam: PU3 - Purlin over side french doors

Span: 3.4 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.26	0.26
U G	Roof_Dead	$0.75(1/\cos 25) \times 1.6$	0		L	2.25	2.25
U G	Vaulted Clg ins. & P.B.	0.15×1.6	0		L	0.41	0.41
U QS	Roof_Live	0.75×1.6	0		L	2.04	2.04
Total load (unfactored): 9.91 kN						4.96	4.96
Dead/Permanent (unfactored): 5.83 kN						2.92	2.92
Live/Variable (unfactored): 4.08 kN						2.04	2.04
(6.10a): 10.94 kN						5.47	5.47
(6.10b): 13.40 kN						6.70	6.70

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed:

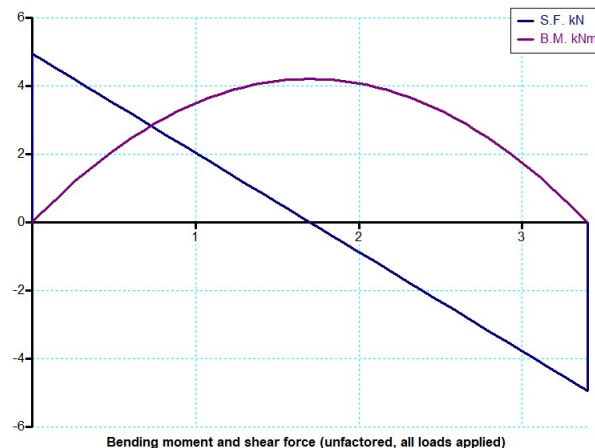
Maximum B.M. = 5.70 kNm (6.10b) at 1.70 m. from R1

Maximum S.F. = 6.70 kN (6.10b) at R1

Mid-span deflections: Dead: $2.99 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $2.09 \times 10^8 / EI$

Total: $5.07 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using C24 timber

Use **150 x 195 C24** 12.3 kg/m approx

$W_{el,y} = 950.6 \text{ cm}^3$ $I_y = 9,269 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 14.77 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 5.70 \times 1000 / 951 = 5.99 \text{ N/mm}^2$ OK

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Bending resistance = $14.77 \times 951/1000 = 14.04$ kNm

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5$ mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46$ N/mm²

Design shear stress, $s_{v,y,d} = 6.70 \times 1000 \times (3/2)/(100.5 \times 195) = 0.51$ N/mm² OK

Shear resistance = $2.46 \times 100.5 \times 195 \times (2/3)/1000 = 32.16$ kN

Deflection

Final deflection limit = $L/250 = 13.60$ mm

Final deflection, $u_{fin} = S u_{inst}(1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 4.21 \times 10^6 / ((11,000/16) \times 150 \times 195) = 0.25$ mm

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	2.99	2.93	1.00	0.80	5.27	
Live QS:	2.09	2.05	0.00	0.80	2.05	
Shear deflection:		0.25			0.37	
Total mm.		<u>5.23</u>			<u>7.69</u>	OK

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Beam: PU4 - Purlins over shower room and store

Span: 2.4 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.18	0.18
U G	Roof_Dead	$0.75(1/\cos 25) \times 2.2$	0		L	2.19	2.19
U G	Vaulted Clg ins. & P.B.	0.15×2.2	0		L	0.40	0.40
U QS	Roof_Live	0.75×2.2	0		L	1.98	1.98
Total load (unfactored): 9.49 kN						4.74	4.74
Dead/Permanent (unfactored): 5.53 kN						2.76	2.76
Live/Variable (unfactored): 3.96 kN						1.98	1.98
(6.10a): 10.43 kN						5.22	5.22
(6.10b): 12.84 kN						6.42	6.42

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed:

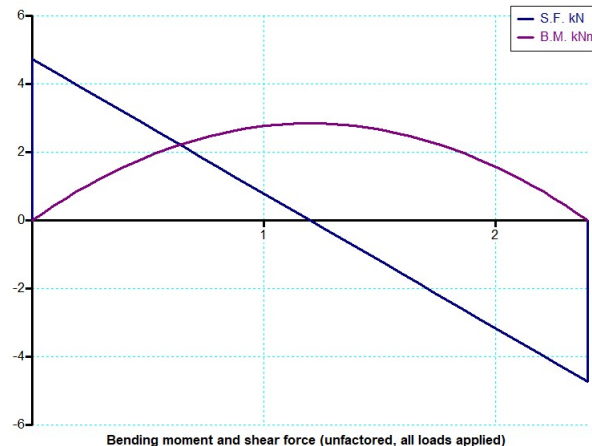
Maximum B.M. = 3.85 kNm (6.10b) at 1.20 m. from R1

Maximum S.F. = 6.42 kN (6.10b) at R1

Mid-span deflections: Dead: $0.99 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.71 \times 10^8 / EI$

Total: $1.71 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using C24 timber

Use **150 x 195 C24** 12.3 kg/m approx

$W_{el,y} = 950.6 \text{ cm}^3$ $I_y = 9,269 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 14.77 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 3.85 \times 1000 / 951 = 4.05 \text{ N/mm}^2$ OK

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Bending resistance = $14.77 \times 951/1000 = 14.04$ kNm

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5$ mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46$ N/mm²

Design shear stress, $s_{v,y,d} = 6.42 \times 1000 \times (3/2)/(100.5 \times 195) = 0.49$ N/mm² OK

Shear resistance = $2.46 \times 100.5 \times 195 \times (2/3)/1000 = 32.16$ kN

Deflection

Final deflection limit = $L/250 = 9.60$ mm

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$$u_{inst,v} = 1.2 \times 2.85 \times 10^6 / ((11,000/16) \times 150 \times 195) = 0.17 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	0.99	0.98	1.00	0.80	1.76	
Live QS:	0.71	0.70	0.00	0.80	0.70	
Shear deflection:		0.17			0.25	
Total mm.		<u>1.84</u>			<u>2.70</u>	OK

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Beam: B1 - Over games room

Span: 3.9 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.3	0		L	0.59	0.59
P G	Beam: PU1 - Purlin : R1	3.92	0.9			3.02	0.90
P QS	Beam: PU1 - Purlin : R1	2.81	0.9			2.16	0.65
P G	Beam: PU2 - Purlins : R1	2.06	0.9			1.58	0.48
P QS	Beam: PU2 - Purlins : R1	1.44	0.9			1.11	0.33
P G	Beam: PU2 - Purlins : R1	2.06	2.9			0.53	1.53
P QS	Beam: PU2 - Purlins : R1	1.44	2.9			0.37	1.07
P G	Beam: PU3 - Purlin : R1	2.92	2.9			0.75	2.17
P QS	Beam: PU3 - Purlin : R1	2.04	2.9			0.52	1.52
Total load (unfactored): 19.86 kN						10.62	9.24
Dead/Permanent (unfactored): 12.13 kN						6.46	5.67
Live/Variable (unfactored): 7.73 kN						4.16	3.57
(6.10a): 22.17 kN						11.84	10.33
(6.10b): 26.74 kN						14.31	12.43

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed:

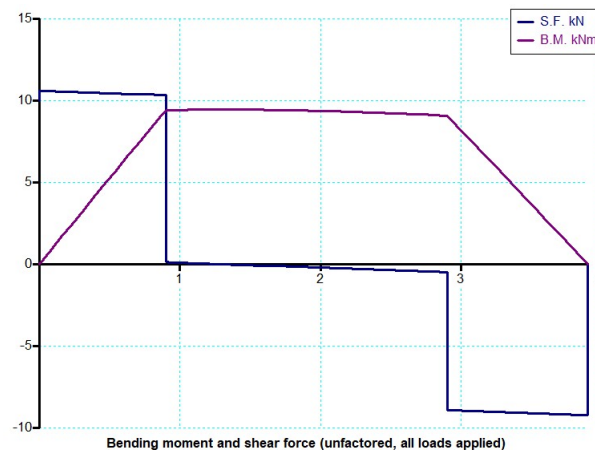
Maximum B.M. = 12.75 kNm (6.10b) at 1.26 m. from R1

Maximum S.F. = 14.31 kN (6.10b) at R1

Mid-span deflections: Dead: $9.98 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $6.40 \times 10^8/EI$

Total: $16.37 \times 10^8/EI$



Timber beam calculation to BS EN1995-1-1 using D40 timber

Use **150 x 195 D40** 19.3 kg/m approx

$W_{el,y} = 950.6 \text{ cm}^3$ $I_y = 9,269 \text{ cm}^4$

Timber grade: D40

Grade bending strength, $f_{m,k} = 40.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 10,900 \text{ N/mm}^2$; $E_{0,mean} = 13,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $\gamma_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / \gamma_m = 40.0 \times 0.80 \times 1.00 \times 1.0/1.30 = 24.62 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 12.75 \times 1000/951 = 13.41 \text{ N/mm}^2$ OK

Bending resistance = $24.62 \times 951/1000 = 23.40 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 14.31 \times 1000 \times (3/2)/(100.5 \times 195) = 1.10 \text{ N/mm}^2$ OK

Shear resistance = $2.46 \times 100.5 \times 195 \times (2/3)/1000 = 32.16 \text{ kN}$

Deflection

Final deflection limit = $L/150 = 26.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst}(1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$$u_{inst,v} = 1.2 \times 9.40 \times 10^6 / ((13,000/16) \times 150 \times 195) = 0.47 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8/EI$	Inst. mm	y_2	k_{def}	Fin. mm	
Dead:	9.98	8.28	1.00	0.80	14.90	
Live QS:	6.40	5.31	0.00	0.80	5.31	
Shear deflection:		0.47			0.71	
Total mm.		<u>14.06</u>			<u>20.92</u>	OK

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Beam: B2 - Central eaves beam over side veranda

Span: 3.5 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.26	0.26
U G	Roof_Dead	$0.75(1/\cos 25) \times 1.2$	0		L	1.74	1.74
U QS	Roof_Live	0.75×1.2	0		L	1.57	1.57
Total load (unfactored): 7.16 kN						3.58	3.58
Dead/Permanent (unfactored): 4.01 kN						2.00	2.00
Live/Variable (unfactored): 3.15 kN						1.57	1.57
(6.10a): 7.77 kN						3.88	3.88
(6.10b): 9.73 kN						4.86	4.86

Load types: U:UDL; Load positions are measured in m. from R1
Load durations: G: Dead; Qx: Imposed:

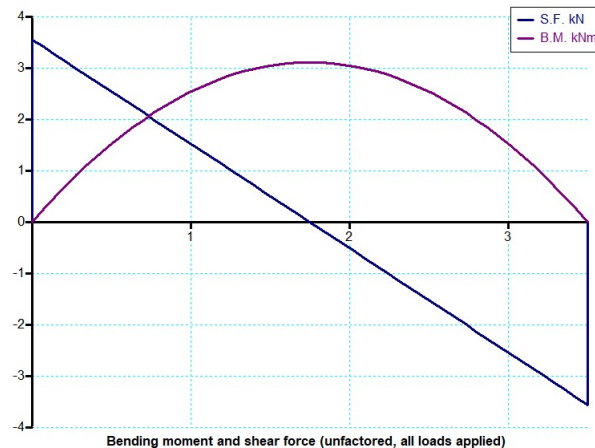
Maximum B.M. = 4.26 kNm (6.10b) at 1.75 m. from R1

Maximum S.F. = 4.86 kN (6.10b) at R1

Mid-span deflections: Dead: $2.24 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $1.76 \times 10^8 / EI$

Total: $3.99 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using D30 timber

Use **150 x 150 D30** 14.4 kg/m approx

$W_{el,y} = 562.5 \text{ cm}^3$ $I_y = 4,219 \text{ cm}^4$

Timber grade: D30

Grade bending strength, $f_{m,k} = 30.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 9,200 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 30.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 18.46 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 4.26 \times 1000 / 563 = 7.57 \text{ N/mm}^2$ OK

Bending resistance = $18.46 \times 563 / 1000 = 10.38 \text{ kNm}$

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Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5$ mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46$ N/mm²

Design shear stress, $s_{v,y,d} = 4.86 \times 1000 \times (3/2)/(100.5 \times 150) = 0.48$ N/mm² OK

Shear resistance = $2.46 \times 100.5 \times 150 \times (2/3)/1000 = 24.74$ kN

Deflection

Final deflection limit = $L/250 = 14.00$ mm

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 3.13 \times 10^6 / ((11,000/16) \times 150 \times 150) = 0.24$ mm

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	2.24	4.82	1.00	0.80	8.67	
Live QS:	1.76	3.79	0.00	0.80	3.79	
Shear deflection:		0.24			0.35	
Total mm.		<u>8.85</u>			<u>12.81</u>	OK

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Beam: B3 - End eaves beam over side veranda

Span: 2.5 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.19	0.19
U G	Roof_Dead	$0.75(1/\cos 25) \times 1.2$	0		L	1.24	1.24
U QS	Roof_Live	0.75×1.2	0		L	1.13	1.13
Total load (unfactored): 5.11 kN						2.56	2.56
Dead/Permanent (unfactored): 2.86 kN						1.43	1.43
Live/Variable (unfactored): 2.25 kN						1.13	1.13
(6.10a): 5.55 kN						2.77	2.77
(6.10b): 6.95 kN						3.47	3.47

Load types: U:UDL; Load positions are measured in m. from R1
Load durations: G: Dead; Qx: Imposed:

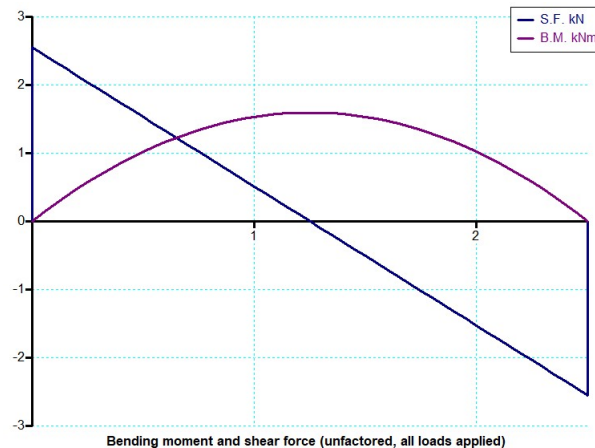
Maximum B.M. = 2.17 kNm (6.10b) at 1.25 m. from R1

Maximum S.F. = 3.47 kN (6.10b) at R1

Mid-span deflections: Dead: $0.58 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.46 \times 10^8 / EI$

Total: $1.04 \times 10^8 / EI$



Timber beam calculation to BS EN1995-1-1 using D30 timber

Use **150 x 150 D30** 14.4 kg/m approx

$W_{el,y} = 562.5 \text{ cm}^3$ $I_y = 4,219 \text{ cm}^4$

Timber grade: D30

Grade bending strength, $f_{m,k} = 30.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 9,200 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.8$ (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.80$ (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 30.0 \times 0.80 \times 1.00 \times 1.0 / 1.30 = 18.46 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.17 \times 1000 / 563 = 3.86 \text{ N/mm}^2$ OK

Bending resistance = $18.46 \times 563 / 1000 = 10.38 \text{ kNm}$

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Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 150 = 100.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / \gamma_m = 4.00 \times 0.80 \times 1.0/1.30 = 2.46 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 3.47 \times 1000 \times (3/2)/(100.5 \times 150) = 0.35 \text{ N/mm}^2$ OK

Shear resistance = $2.46 \times 100.5 \times 150 \times (2/3)/1000 = 24.74 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 10.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 1.60 \times 10^6 / ((11,000/16) \times 150 \times 150) = 0.12 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8/EI$	Inst. mm	γ_2	k_{def}	Fin. mm	
Dead:	0.58	1.25	1.00	0.80	2.26	
Live QS:	0.46	0.99	0.00	0.80	0.99	
Shear deflection:		<u>0.12</u>			<u>0.18</u>	
Total mm.		<u>2.36</u>			<u>3.42</u>	OK

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Beam: GB1 - Ground beam beneath cavity walls (Worst Case)

Span: 3.7 m.

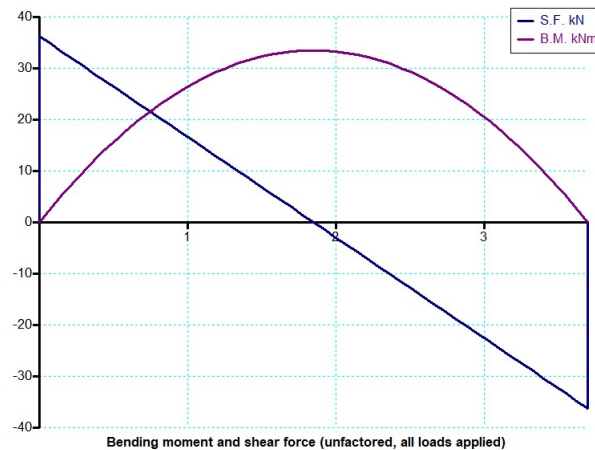
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	3.0	0		L	5.55	5.55
U G	Roof_Dead	$0.75(1/\cos 25) \times 1.2$	0		L	1.84	1.84
U QS	Roof_Live	0.75×1.2	0		L	1.66	1.66
U G	Wall_Cavity_Lightweight	3.25×2.4	0		L	14.43	14.43
U G	Floor_Dead_Concrete	1.0×2.8	0		L	5.18	5.18
U QA	Floor_Live	1.5×2.8	0		L	7.77	7.77
Total load (unfactored): 72.87 kN						36.43	36.43
Dead/Permanent (unfactored): 54.00 kN						27.00	27.00
Live/Variable (unfactored): 18.87 kN						9.43	9.43
(6.10a): 91.71 kN						45.86	45.86
(6.10b): 95.74 kN						47.87	47.87

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

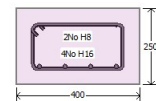
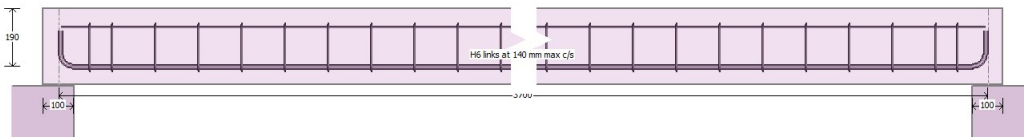
Maximum B.M. = 44.28 kNm (6.10b) at 1.85 m. from R1

Maximum S.F. = 47.87 kN (6.10b) at R1



RC beam calculation to BS EN1992.1.1

Member size: **400 x 250 C25/30**



Effective depth of beam, $d = 190$ mm

Bending

Design bending moment, $M = 44.28$ kNm

$$K = M / (bd^2 f_{ck}) = 44.28 \times 10^6 / (400 \times 190^2 \times 25) = 0.123$$

As $K \leq K'$, 0.168, no compression reinforcement required

$$\text{Lever arm, } z = d(0.5 + \sqrt{(0.25 - 0.881K)}) = 190 \times (0.5 + \sqrt{(0.25 - 0.881 \times 0.123)}) = 167 \text{ mm}$$

$$\text{Tension steel: area required, } A_s = M / (0.87 f_{yk} z) = 44.28 \times 10^6 / (0.87 \times 500 \times 167) = 611 \text{ mm}^2$$

Use **4 No H16**, area = $4 \times 201 = 804 \text{ mm}^2$

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Steel area limits: $\geq 0.0156f_{ck}^{2/3}\%$ = 0.13% = 133 mm² and $\leq 4\%$ = 4,000 mm² OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed} , at face of supports = 46.57 kN (support width 100 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100r_i f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$$k = 1.0$$

$$r_i = A_{st}/(bd) = 804/(400 \times 190) = 0.011 \text{ [6.2.2]}$$

$$V_{Rd,c} = 0.12 \times 1.00(100 \times 0.011 \times 25)^{1/3} \times 400 \times 190/1000 = 27.2 \text{ kN}$$

As $V_{Ed} \geq V_{Rd,c}$, shear reinforcement is required

Calculate concrete strut strength = $v_1 \cdot b \cdot z \cdot f_{cd} / (\cot \phi + \tan(\phi))$

$$v_1 = 0.6(1 - f_{ck}/250) = 0.540; \quad z = 0.9d; \quad f_{cd} = f_{ck}/\gamma_m$$

$$V_{Rd,max,1} (\cot \phi = 2.5) = 0.124 \cdot b \cdot d (1 - f_{ck}/250) \cdot f_{ck} = 212.0 \text{ kN}$$

$$V_{Rd,max,2} (\cot \phi = 1.0) = 0.18 \cdot b \cdot d (1 - f_{ck}/250) \cdot f_{ck} = 308 \text{ kN}$$

As $V_{Ed} \leq V_{Rd,max,1}$ take ϕ as 21.8° ($\cot \phi = 2.5$)

Design links for shear force, V_{Ed} , at d from face of supports = 41.66 kN

Use H6 links: link area, A_{sw} , 2 H6 = 56.5 mm²

Link area required, $A_{sw}/s = V_{Ed}/(0.9 \cdot d \cdot f_{yk} \cdot \cot \phi) = 41.66 \times 1000/(0.9 \times 190 \times (500/1.15) \times 2.50) = 0.224$

Maximum link spacing, $s = 56.5/0.224 = 252 \text{ mm}$, reduced to 0.75d, 143 mm

Use H6 links at 140 mm c/s

Shear resistance, $V_{Rd,s} = (A_{sw}/s) \cdot z \cdot f_{ywd} \cdot \cot \phi = (57/140) \times 0.9 \times 190 \times (500/1.15) \times 2.50/1000 = 75.1 \text{ kN}$ [EC2 Eq.(6.8)]

Tensile force from shear in tension steel, $D F_{td} = 0.5 V_{Ed} \cot \phi = 0.5 \times 41.66 \times 2.50 = 52 \text{ kN}$ [EC2 Eq.(6.18)]

BM at d from face of support = 10.74 kNm: resultant tensile force = 56.5 kN

Total tensile force = 52.1 + 56.5 = 108.6 kN

Tensile resistance = $804 \times (500/1.15)/1000 = 350 \text{ kN}$ OK

Deflection

Basic span/effective depth limits: $r_1 = 0.35\%$: 30; $r_1 = 0.50\%$: 20; $r_1 = 1.5\%$: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b \cdot d = 611 \times 100/(400 \times 190) = 0.804\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 18.2 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but ≤ 1.5) = $804/611 = 1.32$

Adjusted span/depth limit = $18.2 \times 1.32 = 23.9$; Actual = $3.70 \times 1000/190 = 19.5$ OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$$c_{min,b} = f = 16 \text{ mm}$$

$$c_{min,dur} = 45 \text{ mm (exposure class: XC4, cyclic wet or dry, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]}$$

Required cover, $c_{nom} = 45 + 10 = 55 \text{ mm}$

Cover to beam soffit = $250 - (190 + 16/2 + 6) = 46 \text{ mm}$. OK

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Beam: GB2 - Ground beam beneath Deck perimeters (Worst Case)

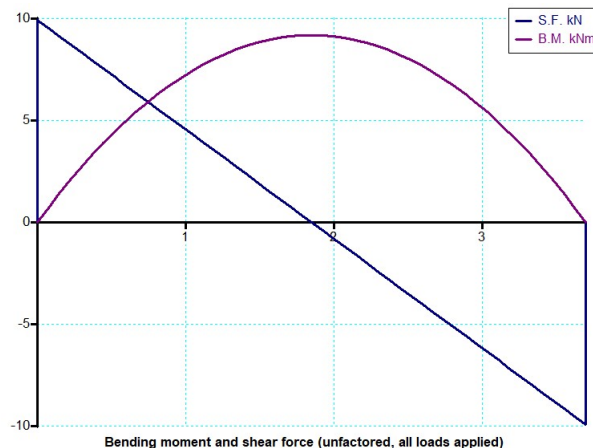
Span: 3.7 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	3.0	0		L	5.55	5.55
U G	Roof_Dead	$0.75(1/\cos 25) \times 0.7$	0		L	1.07	1.07
U QS	Roof_Live	0.75×0.7	0		L	0.97	0.97
U G	Floor_Dead_Concrete	1.0×0.5	0		L	0.93	0.93
U QA	Floor_Live	1.5×0.5	0		L	1.39	1.39
Total load (unfactored): 19.81 kN						9.91	9.91
Dead/Permanent (unfactored): 15.10 kN						7.55	7.55
Live/Variable (unfactored): 4.72 kN						2.36	2.36
(6.10a): 24.75 kN						12.38	12.38
(6.10b): 25.93 kN						12.96	12.96

Load types: U:UDL; Load positions are measured in m. from R1
Load durations: G: Dead; Qx: Imposed; QA: Residential

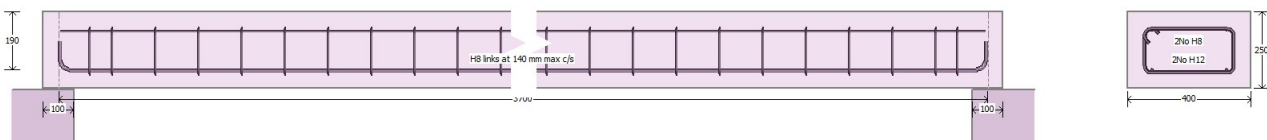
Maximum B.M. = 11.99 kNm (6.10b) at 1.85 m. from R1

Maximum S.F. = 12.96 kN (6.10b) at R1



RC beam calculation to BS EN1992.1.1

Member size: **400 x 250 C25/30**



Effective depth of beam, $d = 190$ mm

Bending

Design bending moment, $M = 11.99$ kNm

$$K = M / (bd^2 f_{ck}) = 11.99 \times 10^6 / (400 \times 190^2 \times 25) = 0.033$$

As $K \leq K'$, 0.168, no compression reinforcement required

$$\text{Lever arm, } z = d(0.5 + \sqrt{0.25 - 0.881K}) = 190 \times (0.5 + \sqrt{0.25 - 0.881 \times 0.033}) = 184 \text{ mm}$$

$$\text{Tension steel: area required, } A_s = M / (0.87 f_{yk} z) = 11.99 \times 10^6 / (0.87 \times 500 \times 184) = 150 \text{ mm}^2$$

$$\text{Use 2 No H12, area} = 2 \times 113 = 226 \text{ mm}^2$$

Steel area limits: $\geq 0.0156 f_{ck}^{2/3} \% = 0.13 \% = 133 \text{ mm}^2$ and $\leq 4 \% = 4,000 \text{ mm}^2$ OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed} , at face of supports = 12.61 kN (support width 100 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100r_i f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$$k = 1.0$$

$$r_i = A_{st}/(bd) = 226/(400 \times 190) = 0.003 \text{ [6.2.2]}$$

$$V_{Rd,c} = 0.12 \times 1.00(100 \times 0.003 \times 25)^{1/3} \times 400 \times 190/1000 = 17.8 \text{ kN}$$

As $V_{Rd,c} > V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw} , 2 H8 = 101 mm²

$$A_{sw}/s = 0.08b \sqrt{f_{ck}/f_{yk}} = 0.08 \times 400 \times \sqrt{25/(1000 \times 500)} = 0.320$$

Maximum link spacing, s , = 101/0.320 = 314 mm, reduced to 0.75d, 143 mm

Provide H8 links at 140 mm max c/s

Deflection

Basic span/effective depth limits: $r_1=0.35\%$: 30; $r_1=0.50\%$: 20; $r_1=1.5\%$: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 150 \times 100/(400 \times 190) = 0.197\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but ≤ 1.5) = 226/150 = 1.51 so 1.5

Adjusted span/depth limit = 30.0 x 1.50 = 45.0; Actual = 3.70 x 1000/190 = 19.5 OK

Check detailing

Minimum bar spacing needs to be greatest of f (12mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$$c_{min,b} = f = 12 \text{ mm}$$

$$c_{min,dur} = 45 \text{ mm (exposure class: XC4, cyclic wet or dry, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]}$$

Required cover, $c_{nom} = 45 + 10 = 55 \text{ mm}$

Cover to beam soffit = 250 - (190 + 12/2 + 8) = 46 mm. OK