

Project Title:	Existing Barn, 58 Spalding Road, HOLBEACH, Lincolnshire PE12 7HG
Client:	Mr T Penney
Report Type:	Structural Inspection Report Final Version
Date:	January 2023
Project No.	PW-22-11-03

PW
STRUCTURAL SOLUTIONS LTD

19 Coleridge Place
Bourne
Lincolnshire
PE10 9WA
T:01778 420269
M:07902 538309
E:peter.wood@pwstructural.co.uk

Tel: 01778 420269
Mob: 07902 538309
E-mail: peter.wood@pwstructural.co.uk

19 Coleridge Place
Bourne
Lincolnshire
PE10 9WA

Mr T Penney
10 Abbots Crescent,
SPALDING,
Lincolnshire.
PE11 1NB

Date:15 January 2022

Dear Mr Penney

Re Existing Barn Conversion 58 Spalding Road, Holbeach – Structural Report: Final

The purpose of this report is to address a concern raised in connection with planning permission for the above mentioned conversion. The concern is in respect of the structural integrity of the conversion once complete and how such structural integrity can be maintained, and where necessary, made good.

The report is based upon photographs, drawings and details made available by yourself and additional site observations made by PW Structural Solutions Ltd. Those site observations were made on the morning of 9th December 2022 when the ambient conditions were dry and bright.

1.0 Building Description

The existing barn is a brick-built structure with a slate roof supported on timber rafters and purlins. It is approximately 12.79m long x 4.79m wide and approximately 4.73 m to eaves level above ground floor level. The roof is a symmetrical duo-pitch roof with a rise of approximately 2.03m. The long walls and the roof ridge line are orientated approximately east/west. There is no visible insulation to the structure. The structure is understood to be at least 75 years old.

The external brick walls are 215mm (8.5”) thick, as are the two internal partition walls at ground floor level. The internal partition wall at first floor level is 102.5mm (4”) thick. The first floor, accessed via a companionway type timber ladder is timber boards supported on joists which span between the long walls. The roof structure, which is only visible in the eastern half of the property, comprises timber rafters supported on the wall, on purlins and a ridge board. The purlins span from the gable end walls onto the internal partition wall. At least one collar tie has been provided within the east end of the roof. The western end has a ceiling higher than the collar tie. It is possible that there is a collar tie between each pair of rafters above the ceiling providing support for the ceiling.

The building foundations comprise two courses of corbelled brickwork sitting on the local yellow silt formation. Formation level is approximately 0.76m below ground level. The ground floor is a concrete slab which is assumed to be laid directly onto the sub-strate.

Windows and doors currently exist on the south and west elevations only. The west elevation has a central access door with windows either side at ground floor level. A central loading door exists at first floor level.

The south elevation originally had two stable doors although only one now remains. The innermost stable door has been replaced by a wider double door which is assumed to have been intended to allow the stable to be used as a garage. An additional window exists at ground floor level beyond the garage door.

At first floor level, a second loading door is located on the south elevation on an alignment roughly central to the two original stable doors. There are two windows at first floor level with the innermost window being directly above the matching window at ground floor level.

Lintels to all original doors and windows are brick-arch lintels externally and timber beam lintels internally. There is no visible lintel above the "garage" doors. The frame appears to be acting as a lintel. However, it does not appear to be doing so successfully as the doors do not open easily.

2.0 Proposed Modifications

The proposal is to convert the barn into a 3-bedroom dwelling. This involves the removal of one of the existing ground floor walls and the introduction of window and door openings into the existing plain rear wall of the property. An extension within the footprint of the existing "lean-to" covered storage area is proposed to include a ground floor cloakroom and a utility room. The main entrance to the dwelling is proposed to be through the utility room.

3.0 0bservations – Existing Building

When the innermost stable door was replaced, no attempt appears to have been made to provide a new lintel above the wider garage doors. The original brick arch lintel over the erstwhile stable door is still extant, and the space below this and the garage door frame has been infilled with additional brickwork to match the existing. The brickwork below the original lintel has fallen away from it leaving a gap that is up to 5mm wide.

A diagonal crack extends from the top east corner of the garage door to the centre of the loading door above where the stone has split vertically. Testing with a spirit level indicated a bow in the horizontal plane exists at this point.

The partition wall between the two original stables includes an inter-connecting door located approximately 440mm behind the external wall. A significant crack exists in the wall above the door. The crack width is approximately 25mm. Marks on the first-floor joists adjacent to the partition wall indicate a similar reduction in bearing length of the joists. The partition wall is on the same line as the apex of the bow observed on the external wall. It is probable then that the bow, the significant crack, and the replacement of the original stable door are all related to the installation of the garage doors.

It is worth noting that a slight sag in the roof ridge line was also visible on the same alignment.

These observations can be seen in the pictures below.






J. PENNEY & SON (1956)
L.A. & J. PENNEY & SON (1956)

Minor cracking also exists within the brickwork above the remaining stable door, though this entirely within the coursing.



As is typical of buildings of this age which are generally less well maintained than dwellings some weathering of bricks was observed on the south and west elevations. Loss of the “glazed face” of the bricks could be seen on the south elevation and some corner spalling was observed.





At first floor level cracks exist at the junction between the external walls and the partition wall. What can also be seen is that the coursing between the internal wall and the external walls is slightly mis-aligned. This does suggest that the walls may not be properly tied together either with masonry ties or by “toothing-in”.

The lintels above the first-floor windows in the south elevation are timber internally. These exhibit signs of deflection under load (vertical cracks at quarter points along the span) and thermal expansion (diagonal cracks at the ends).

4.0 Observations – Proposed Conversion

At first floor level the existing partition wall remains as do the window and door openings on the south and west elevations. Even with the addition of insulation above the new ceiling, these walls should remain capable of carrying the loads.

The north wall has two additional penetrations which will require lintels to be provided. Wall stability checks may lead to wind posts being required to ensure the stability of the wall panel.

At ground floor level, the existing door and window openings on the south and west elevations will remain. The lintel above the existing garage door will require replacement to ensure that it is consistent with the width of the opening below.

New openings are to be formed in the north elevation. Each of these will require properly designed and specified lintels. These openings may compromise the stability of the existing wall panels. The residual panels should be checked and remediated as necessary with wind posts etc. The stability of wall panels on both north and south elevations may also be compromised by the removal of the wall between the original stables. Whilst this wall does not support any load from the floor above it does act as a “buttressing” wall as defined in Part A1/2 of the Building Regulations. The removal of the wall may result in additional restraints against horizontal (i.e. wind) loads being required.

The existing floor joists are 2” x 9” sawn timber at 15” centres. The effective span is 14’9” (4.5m) At stress grade C16 initial calculations show that these are adequate to support domestic floor loading. Strengthening (e.g. flitching) may be required allow individual joists to support stud-partition walls, landing and stairs.

The existing rafters are 2” x 4” sawn timber also at 15” centres (52 x 104 @381 centres). These are supported on 3” x 5” purlins at mid-span. An initial calculation shows that with the addition of a ceiling and insulation etc. the rafters will remain adequate at stress grade C16.

The span of the visible purlins is 18’ (5.49m). Initial calculations show that this will not be adequate under the increased roof loading. It may be possible to flitch these purlins to achieve the required capacity. Reducing the span by converting some of the intermediate rafters to trusses is another means of increasing the capacity of the purlin to acceptable levels.

In respect of the foundations, the load applied to the existing barn as it currently stands, and as it will be following conversion, are broadly identical over the footprint of the building. There will be some re-distribution of load onto the foundation on the north elevation as a result of new openings. However, the ground below the existing footings has been consolidated for at least the last 70 years so any “recovery” due to load removal, or additional “settlement” due to increased loads, is expected to be insignificant. The load on the barn floor (storage of chitting potatoes) is several times higher than domestic loading. Even with an insulated and raised floor, the loading on the existing slab will not exceed that maximum loading that has been applied to it so far.

The extension will require a new foundation within ground that has been used for external covered storage. This will have given the ground below the proposed extension some consolidation, but it is unlikely to match that of the existing building. This means that there is a potential for differential settlement between the extension and the existing building. This can be mitigated by constructing the extension on a reinforced concrete raft foundation which can also be detailed to avoid adding any load directly to the existing footings.

5.0 Structural Feasibility

Converting the existing barn to a dwelling will require some structural engineering work to be carried out in order to maintain the structural integrity of the building and to ensure that integrity endures in the long term. None of this work is unusual or challenging. It is typical of much of the work that is seen in domestic projects and similar conversions.

A reinforced concrete raft foundation is a straightforward design, detailing and construction exercise. Lintels are proprietary items specified to suit the applied loading. The possible lack of ties between the external wall and the partition wall at first floor level can be addressed by the installation of “L” shaped “Helifix” type bars across the junctions between them.

Existing rafters can be converted to trusses with the installation of bottom chord ties and appropriate internal members. First floor joists carrying additional loads from walls and stairs etc can be strengthened by flitching with steel plates as necessary.

The one item that will require careful consideration is the remediation measures for the bow in the south wall and the 25mm crack in the wall between the stables. It should be noted here that the wall itself will be removed, but the defect will still require remediation if long-term stability of the overall structure is to be ensured.

Tie rods across the building would be one method, but the size and position of the new door and window combination in the north wall makes such an approach difficult. A steel frame in three dimensions could be used to tie the north and south walls together and re-instate the buttressing action lost by the removal of the wall. This would incorporate wind posts on the north elevation, a beam along the line of the removed wall, and a wind post on the south wall also on the line of the removed wall.

6.0 Summary

The existing barn is a 70+ year old building in remarkably good condition. It has seen some modification during its life and it is probable that one of those modifications has caused the only significant defect observed on the existing property.

Remediation of that defect is the most difficult structural engineering task required for the proposed conversion; but it is, nevertheless, straightforward. All other tasks required to ensure the successful conversion of the property are simple by comparison, both in design and in execution.

There is no structural engineering reason why this project should not progress.

If you have any questions, please call me.

Yours Sincerely,

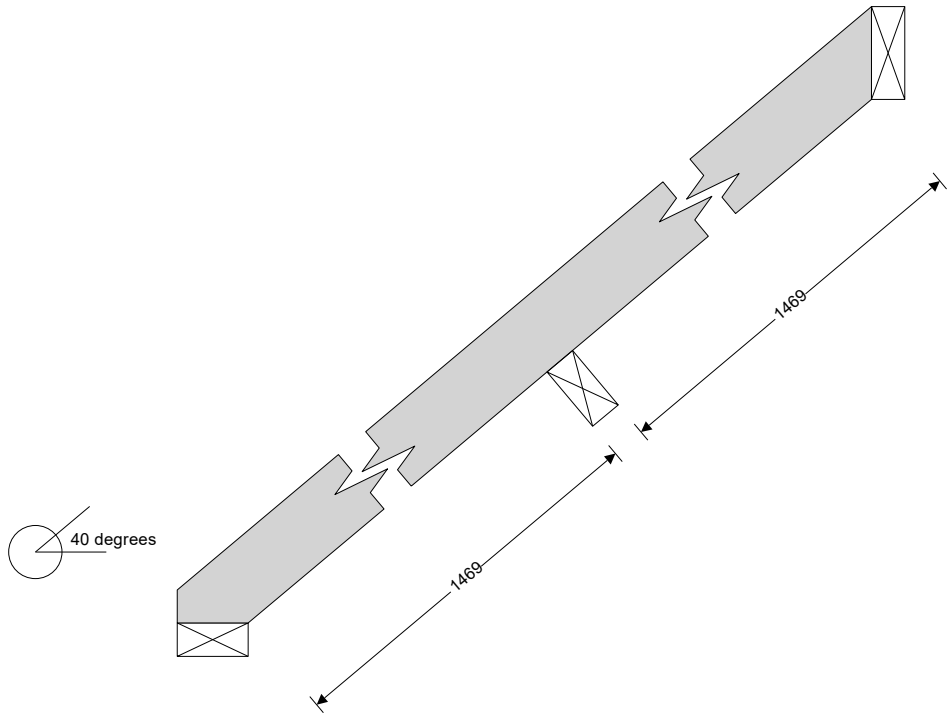


Peter Wood, Director
BSc CEng MICE MIStructE

Tekla Tedds PW Structural Solutions Ltd 19 Coleridge Place, Bourne Lincolnshire PE10 9WA	Project Existing Barn - 58 Spalding Road, Holbeach			Job no. PW-22-11-03	
	Calcs for Rafters			Start page no./Revision A/ 1	
	Calcs by P D Wood	Calcs date 14/12/2022	Checked by	Checked date	Approved by	Approved date

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections	b = **47** mm
Depth of timber sections	h = **100** mm
Rafter spacing	s = **381** mm
Rafter slope	α = **40.0** deg
Clear span of rafter on horizontal	L_{clh} = **1125** mm
Clear span of rafter on slope	$L_{cl} = L_{clh} / \cos(\alpha)$ = **1469** mm
Rafter span	**Continuous**
Timber strength class	**C16**

Section properties

Cross sectional area of rafter	$A = b \times h$ = **4700** mm^2
Section modulus	$Z = b \times h^2 / 6$ = **78333** mm^3
Second moment of area	$I = b \times h^3 / 12$ = **3916667** mm^4
Radius of gyration	$r = \sqrt{(I / A)}$ = **28.9** mm

Loading details

Rafter self weight	$F_j = b \times h \times \rho_{char} \times g_{acc}$ = **0.01** kN/m
Dead load on slope	F_d = **0.53** kN/m^2
Imposed load on plan	F_u = **0.75** kN/m^2
Imposed point load	F_p = **0.90** kN

Modification factors

Section depth factor	$K_7 = (300\ mm / h)^{0.11}$ = **1.13**
Load sharing factor	K_8 = **1.10**

Consider long term load condition

Load duration factor K_3 = **1.00**

Total UDL perpendicular to rafter $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.166** kN/m

Notional bearing length $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **1** mm

Effective span $L_{eff} = L_{cl} + L_b$ = **1470** mm

Check bending stress at purlin

Bending stress parallel to grain σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.579** N/mm^2

Applied bending stress $\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z)$ = **0.571** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain σ_c = **6.800** N/mm^2

Minimum modulus of elasticity E_{min} = **5800** N/mm^2

Compression member factor K_{12} = **0.72**

Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.411** N/mm^2

Applied compressive stress $\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.067** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **22.086** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e)$ = **0.997**

Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm}$ = **0.099 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.579** N/mm^2

Applied bending stress $\sigma_{m_max} = 9 \times F \times L_{eff}^2 / (128 \times Z)$ = **0.321** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain σ_c = **6.800** N/mm^2

Minimum modulus of elasticity E_{min} = **5800** N/mm^2

Compression member factor K_{12} = **0.72**

Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.411** N/mm^2

Applied compressive stress $\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 13 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.094** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **22.086** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e)$ = **0.995**

Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm}$ = **0.066 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.737** N/mm^2

Applied shear stress $\tau_{max} = 15 \times F \times L_{eff} / (16 \times A)$ = **0.049** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} =$ **4.409** mm

Bending deflection $\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I) =$ **0.121** mm

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) =$ **0.021** mm

Total deflection $\delta_{max} = \delta_b + \delta_s =$ **0.142** mm

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor $K_3 =$ **1.25**

Total UDL perpendicular to rafter $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) =$ **0.333** kN/m

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

Effective span $L_{eff} = L_{cl} + L_b =$ **1471** mm

Check bending stress at purlin

Bending stress parallel to grain $\sigma_m =$ **5.300** N/mm^2

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.224** N/mm^2

Applied bending stress $\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) =$ **1.151** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain $\sigma_c =$ **6.800** N/mm^2

Minimum modulus of elasticity $E_{min} =$ **5800** N/mm^2

Compression member factor $K_{12} =$ **0.70**

Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} =$ **6.547** N/mm^2

Applied compressive stress $\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A) =$ **0.134** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **22.053** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) =$ **0.994**

Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} =$ **0.161 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain $\sigma_m =$ **5.300** N/mm^2

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.224** N/mm^2

Applied bending stress $\sigma_{m_max} = 9 \times F \times L_{eff}^2 / (128 \times Z) =$ **0.647** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain $\sigma_c =$ **6.800** N/mm^2

Minimum modulus of elasticity $E_{min} =$ **5800** N/mm^2

Compression member factor $K_{12} =$ **0.70**

Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} =$ **6.547** N/mm^2

Applied compressive stress $\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 13 \times \tan(\alpha) / 3) / (8 \times A) =$ **0.189** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **22.053** N/mm^2

Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) =$ **0.991**
Combined axial compression and bending check	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} =$ **0.108 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain	$\tau =$ **0.670** N/mm^2
Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.921** N/mm^2
Applied shear stress	$\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) =$ **0.098** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff} =$ **4.412** mm
Bending deflection	$\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I) =$ **0.245** mm
Shear deflection	$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) =$ **0.042** mm
Total deflection	$\delta_{max} = \delta_b + \delta_s =$ **0.286** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor	$K_3 =$ **1.50**
Total UDL perpendicular to rafter	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) =$ **0.166** kN/m
Notional bearing length	$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] =$ **4** mm
Effective span	$L_{eff} = L_{cl} + L_b =$ **1473** mm

Check bending stress at purlin

Bending stress parallel to grain	$\sigma_m =$ **5.300** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **9.868** N/mm^2
Applied bending stress	$\sigma_{m_max} = F \times L_{eff}^2/(8 \times Z) + 3 \times F_p \times \cos(\alpha) \times L_{eff}/(32 \times Z) =$ **1.788** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain	$\sigma_c =$ **6.800** N/mm^2
Minimum modulus of elasticity	$E_{min} =$ **5800** N/mm^2
Compression member factor	$K_{12} =$ **0.68**
Permissible compressive stress	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} =$ **7.576** N/mm^2
Applied compressive stress	$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha)/3)/(8 \times A) + F_p \times \sin(\alpha)/A =$ **0.190** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress	$\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **21.995** N/mm^2
Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) =$ **0.991**
Combined axial compression and bending check	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} =$ **0.208 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain	$\sigma_m =$ **5.300** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **9.868** N/mm^2
Applied bending stress	$\sigma_{m_max} = F \times L_{eff}^2/(16 \times Z) + 13 \times F_p \times \cos(\alpha) \times L_{eff}/(64 \times Z) =$ **2.919** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain	$\sigma_c =$ **6.800** N/mm^2

Minimum modulus of elasticity	E_{min} = **5800** N/mm^2
Compression member factor	K_{12} = **0.68**
Permissible compressive stress	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **7.576** N/mm^2
Applied compressive stress	$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha)+4 \times \tan(\alpha))/(8 \times A)+F_p \times \sin(\alpha)/A$ = **0.212** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress	$\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **21.995** N/mm^2
Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e)$ = **0.990**
Combined axial compression and bending check	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm}$ = **0.327 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain	τ = **0.670** N/mm^2
Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8$ = **1.106** N/mm^2
Applied shear stress	$\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.269** N/mm^2

PASS - Applied shear stress within permissible limits

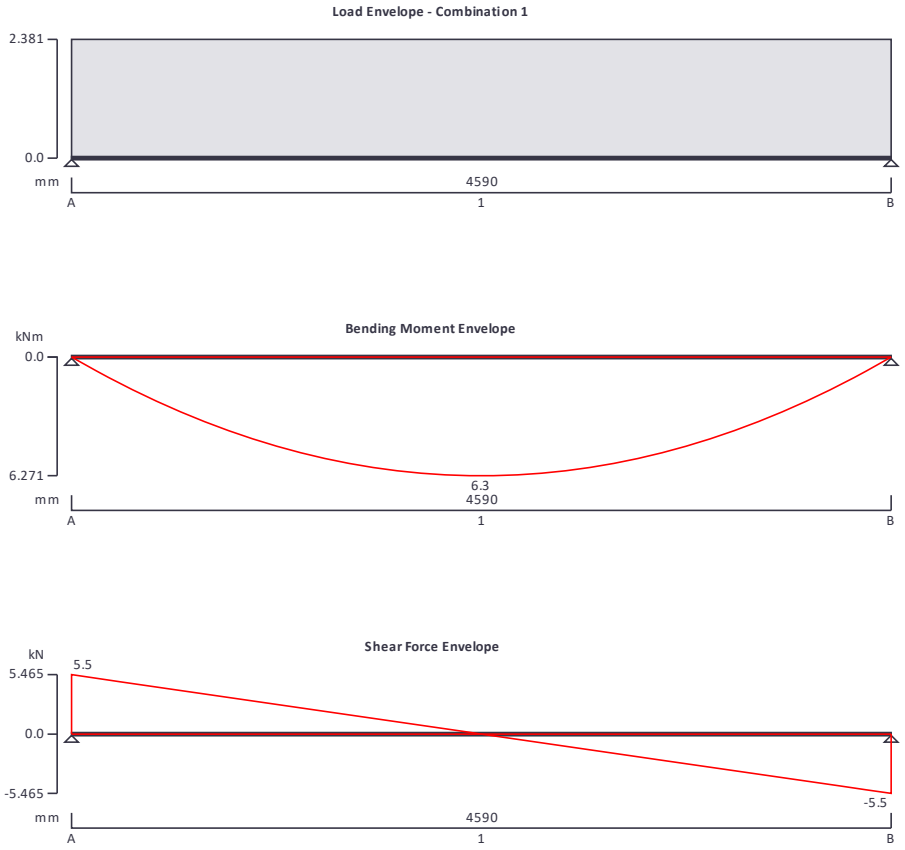
Check deflection

Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff}$ = **4.418** mm
Bending deflection	$\delta_b = L_{eff}^3 \times (F \times L_{eff} /185 + 0.015 \times F_p \times \cos(\alpha)) / (E_{mean} \times I)$ = **1.081** mm
Shear deflection	$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} +2 \times F_p \times \cos(\alpha))/(5 \times E_{mean} \times A)$ = **0.139** mm
Total deflection	$\delta_{max} = \delta_b + \delta_s$ = **1.219** mm

PASS - Total deflection within permissible limits

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.980 kN/m
Imposed full UDL 1.380 kN/m

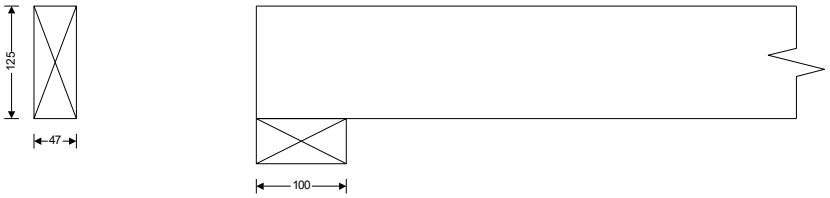
Load combinations

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

Analysis results

Maximum moment	M_{max} = **6.271** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **6.271** kNm	
Maximum shear	F_{max} = **5.465** kN	F_{min} = **-5.465** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **5.465** kN	
Total load on beam	W_{tot} = **10.930** kN	

Reactions at support A	R_{A_max} = **5.465** kN	R_{A_min} = **5.465** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **2.298** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **3.167** kN	
Reactions at support B	R_{B_max} = **5.465** kN	R_{B_min} = **5.465** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.298** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **3.167** kN	



Timber section details

Breadth of sections	b = **47** mm
Depth of sections	h = **125** mm
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **47** mm
Inclination of section	θ = **40.0** deg
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **4590** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **5875** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **122396** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **46021** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **7649740** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **1081490** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **36.1** mm
	$i_y = \sqrt{(I_y / A)}$ = **13.6** mm
Second moment of area of rotated section	$I_\theta = (I_x + I_y) / 2 + (I_x - I_y) \times \cos(2 \times \theta) / 2$ = **4935897** mm^4

Modification factors

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

Lateral support - cl.2.10.8

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

Permissible depth-to-breadth ratio - Table 19	**5.00**
Actual depth-to-breadth ratio	$h / (N \times b)$ = **2.66**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.200** N/mm^2

Applied bearing stress

$\sigma_{c_a} = R_{B_max} / (N \times b \times L_b) =$ **1.163** N/mm^2

$\sigma_{c_a} / \sigma_{c_adm} =$ **0.529**

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

Bending parallel to grain

Permissible bending stress

$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **5.836** N/mm^2

Applied bending stress

$\sigma_{m_a} = M \times \cos(\theta) / Z_x + M \times \sin(\theta) / Z_y =$ **126.842** N/mm^2

$\sigma_{m_a} / \sigma_{m_adm} =$ **21.735**

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress

$\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.670** N/mm^2

Applied shear stress

$\tau_a = 3 \times F / (2 \times A) =$ **1.395** N/mm^2

$\tau_a / \tau_{adm} =$ **2.083**

FAIL - Applied shear stress exceeds permissible shear stress

Deflection

Modulus of elasticity for deflection

$E = E_{min} =$ **5800** N/mm^2

Permissible deflection

$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **13.770** mm

Bending deflection

$\delta_{b_s1} =$ **480.743** mm

Shear deflection

$\delta_{v_s1} =$ **3.534** mm

Total deflection

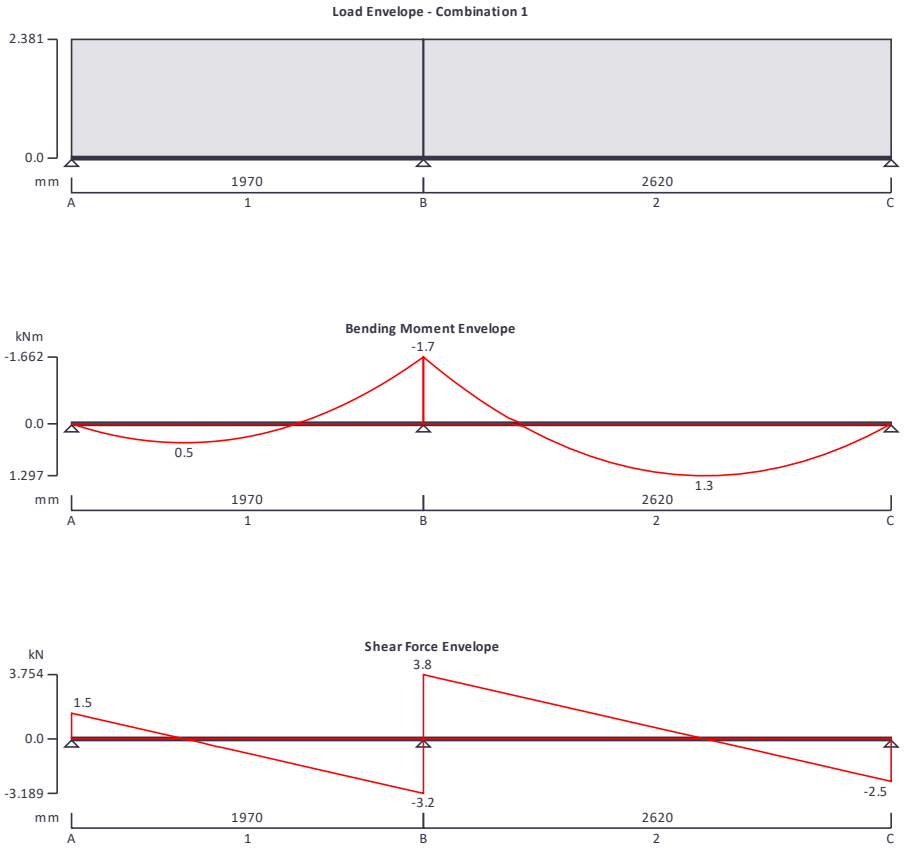
$\delta_a = \delta_{b_s1} + \delta_{v_s1} =$ **484.276** mm

$\delta_a / \delta_{adm} =$ **35.169**

FAIL - Total deflection exceeds permissible deflection

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



Applied loading

Beam loads

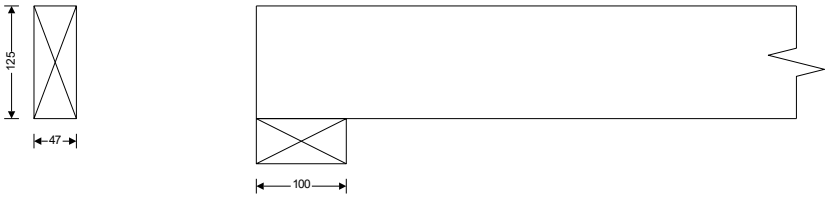
Dead self weight of beam × 1
Dead full UDL 0.980 kN/m
Imposed full UDL 1.380 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00
	Span 2	Dead × 1.00
		Imposed × 1.00
	Support C	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **1.297** kNm	M_{min} = **-1.662** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.662** kNm	
Maximum shear	F_{max} = **3.754** kN	F_{min} = **-3.189** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **3.754** kN	
Total load on beam	W_{tot} = **10.930** kN	
Reactions at support A	R_{A_max} = **1.502** kN	R_{A_min} = **1.502** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.632** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **0.870** kN	
Reactions at support B	R_{B_max} = **6.943** kN	R_{B_min} = **6.943** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.920** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **4.024** kN	
Reactions at support C	R_{C_max} = **2.485** kN	R_{C_min} = **2.485** kN
Unfactored dead load reaction at support C	R_{C_Dead} = **1.045** kN	
Unfactored imposed load reaction at support C	$R_{C_Imposed}$ = **1.440** kN	



Timber section details

Breadth of sections	b = **47** mm
Depth of sections	h = **125** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **47** mm
Inclination of section	θ = **40.0** deg
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span 1	L_{s1} = **1970** mm
Length of span 2	L_{s2} = **2620** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **5875** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **122396** mm^3
	Z_y = h × (N × b)^2 / 6 = **46021** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **7649740** mm^4
	I_y = h × (N × b)^3 / 12 = **1081490** mm^4
Radius of gyration	i_x = √(I_x / A) = **36.1** mm
	i_y = √(I_y / A) = **13.6** mm
Second moment of area of rotated section	$I_θ$ = (I_x + I_y) / 2 + (I_x - I_y) × cos(2 × θ) / 2 = **4935897** mm^4

Modification factors

Duration of loading - Table 17	K_3 = **1.00**

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

Lateral support - cl.2.10.8

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

Permissible depth-to-breadth ratio - Table 19	**5.00**
Actual depth-to-breadth ratio	$h / (N \times b)$ = **2.66**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2
Applied bearing stress	$\sigma_{c_a} = R_{B_max} / (N \times b \times L_b)$ = **1.477** N/mm^2
	$\sigma_{c_a} / \sigma_{c_adm}$ = **0.610**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **5.836** N/mm^2
Applied bending stress	$\sigma_{m_a} = M \times \cos(\theta) / Z_x + M \times \sin(\theta) / Z_y$ = **33.618** N/mm^2
	$\sigma_{m_a} / \sigma_{m_adm}$ = **5.761**

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8$ = **0.670** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A)$ = **0.958** N/mm^2
	τ_a / τ_{adm} = **1.431**

FAIL - Applied shear stress exceeds permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min}$ = **5800** N/mm^2
Permissible deflection	$\delta_{adm} = \min(0.551\ in, 0.003 \times L_{s2})$ = **7.860** mm
Bending deflection	δ_{b_s2} = **26.590** mm
Shear deflection	δ_{v_s2} = **0.731** mm
Total deflection	$\delta_a = \delta_{b_s2} + \delta_{v_s2}$ = **27.321** mm
	δ_a / δ_{adm} = **3.476**

FAIL - Total deflection exceeds permissible deflection

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

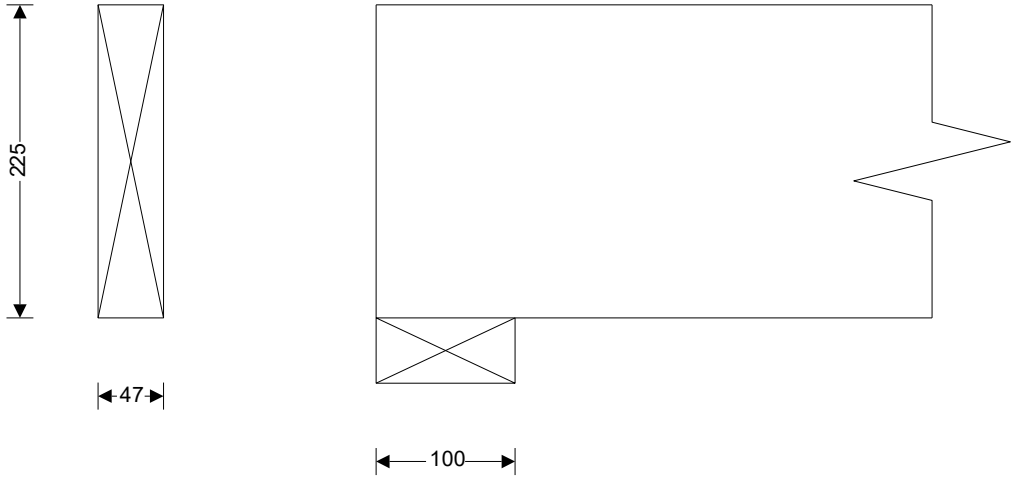
Joist details

Joist breadth	b = **47** mm
Joist depth	h = **225** mm
Joist spacing	s = **381** mm
Timber strength class	**C16**
Service class of timber	**1**



Span details

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



Section properties

Second moment of area	$I = b \times h^3 / 12$ = **44613281** mm^4
Section modulus	$Z = b \times h^2 / 6$ = **396563** mm^3

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m
Dead load	F_{d_udl} = **0.50** kN/m^2
Imposed UDL(Long term)	F_{i_udl} = **1.50** kN/m^2
Imposed point load (Medium term)	F_{i_pt} = **1.40** kN

Modification factors

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

Service class for shear parallel to grain	K_{2s} = **1.00**
Service class for modulus of elasticity	K_{2e} = **1.00**
Section depth factor	K_7 = **1.03**
Load sharing factor	K_8 = **1.10**

Consider long term loads

Load duration factor	K_3 = **1.00**
Maximum bending moment	M = **2.010** kNm
Maximum shear force	V = **1.787** kN
Maximum support reaction	R = **1.787** kN
Maximum deflection	δ = **11.215** mm

Check bending stress

Bending stress	σ_m = **5.300** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **6.017** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **5.069** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	τ = **0.670** N/mm^2
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.737** N/mm^2
Applied shear stress	τ_{max} = 3 × V / (2 × b × h) = **0.253** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)	σ_{cp1} = **2.200** N/mm^2
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.420** N/mm^2
Applied bearing stress	σ_{c_max} = R / (b × L_b) = **0.380** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	δ_{adm} = min(L_{s1} × 0.003, 14 mm) = **13.500** mm
Bending deflection (based on E_{mean})	$\delta_{bending}$ = **10.800** mm
Shear deflection	δ_{shear} = **0.415** mm
Total deflection	$\delta = \delta_{bending} + \delta_{shear}$ = **11.215** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **2.139** kNm
Maximum shear force	V = **1.901** kN
Maximum support reaction	R = **1.901** kN
Maximum deflection	δ = **10.239** mm

Check bending stress

Bending stress	σ_m = **5.300** N/mm^2
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.522** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **5.393** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	τ = **0.670** N/mm^2

Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 =$ **0.921** N/mm^2
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h) =$ **0.270** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)	$\sigma_{cp1} =$ **2.200** N/mm^2
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 =$ **3.025** N/mm^2
Applied bearing stress	$\sigma_{c_max} = R / (b \times L_b) =$ **0.404** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm}) =$ **13.500** mm
Bending deflection (based on E_{mean})	$\delta_{bending} =$ **9.798** mm
Shear deflection	$\delta_{shear} =$ **0.441** mm
Total deflection	$\delta = \delta_{bending} + \delta_{shear} =$ **10.239** mm

PASS - Actual deflection within permissible limits