

Issued by email:

Ref: 9540/NL/MJ

5th July, 2019

Issued by email to:

Paul Temblett (p.temblett@talk21.com)

Dear Mr Temblett,

RE: Structural Inspection - Barn Conversion - Nant y Gaseg Uchaf Farm, Rhydyfro, Swansea. SA8 4SY

Following our appointment, we undertook an inspection of the above property, which was performed on Tuesday 18th of June, 2019. The inspection was instructed to provide a full structural survey of the barn and to address pre-commencement condition no.3 set out within the Planning Approval documentation.

Our findings are restricted to structural matters only. The inspection and report is based on a visual inspection of all exposed surfaces from ground level. The purpose of this report is to make general structural assessment of the building in respect of its present and potential structural defects and to provide feasible remedial recommendations for the purpose of converting the barn into habitable structures in accordance with the current standards and Building Regulation Part A.

This report shall be for the confidential use of our client, and the report shall not be reproduced in whole, or in part, or relied upon by a third party without the express written authorisation of Vale Consultancy.

Description

The barn is located within Nant y Gaseg Uchaf Farm, just off the A474 to the north of Pontardawe. The building is rectangular in plan approximately 24m in length and 6m in width. The building is divided into three different segments, with large solid random rubble masonry walls dividing each segment and concurrently providing internal support for the timber purlins.

The solid random rubble masonry walls are approximately 500mm thick and are built directly off the firm clay ground. The barn consists of a duo pitch roof, with gable ends facing east and west, with the front elevation facing north (**photograph 1**). The duo pitch roof is supported by a series of timber A-frames spaced at varying centres, subsequently supporting timber purlins and rafters. The roof is covered in what appears to be relatively recent blue slates. A sketch of the roof members can be found on **Sketch SK01** in **Appendix B**.

The external ground level adjacent to the eastern gable is currently higher than the internal ground level by approximately 150mm, whereas the ground level adjacent to the western gable is lower than the internal floor level. However, it is anticipated to lower the external levels of the ground to accommodate a patio area.

The lintels over the window and door openings are all timbers.

Observation and Recommendations

By looking at the ridgeline from outside the building, there is no evidence of excessive sagging, which is a common sign of roof splaying. However, the ridgeline does appear to vary in level and deflect naturally from one internal load bearing masonry wall to the other (**photograph 1**). Internally, the roof structure appears relatively in good condition, excluding some of the purlins and localised areas of the A-frame timbers. As seen in **Photograph 2**, evidence of slight deflection was noticed in some of the timber purlins. This deflection is evidence under current permanent loading of the roof, which only entails slates, battens, felt and the self-weight of the timber structure. However, to enable transforming the building into a habitable space and to conform with current building regulations, insulation and plasterboard will have to be added to the roof profile which evidentially increases the permanent loading on the roof by 0.2kN/m^2 (20kg/m^2).

Structural calculations have been carried out on the existing purlins and sloping rafters which can be found in **Appendix B**. As seen in the Calculations Package, the existing timber purlins are inadequate for additional loading to conform to current standards. Therefore, we have carried out a design for strengthening works of the purlins. We suggest strengthening the purlins by either adding a 125mm deep x 50mm wide C24 timber purlin to the existing member to provide extra support for the sloping timber rafters or replacing the rafters with 200mm deep x 75mm wide C24 timber purlins. If additional timber sections are to be added to the existing, then the new timber members should be connected by bolting them through using M12 bolts at 450mm centres, staggered. The sloping rafters were checked for bending moment capacity, deflection and shear stress capacity. It was found that the current timber rafters are adequate to support the extra loading. From inspection, the collar ties for the timber A-frames appears too high to avoid splaying of the truss chords. It is industry standard to limit the height of the collar tie to $\frac{1}{3}$ total height of the truss, therefore we recommend installing a new-collar tie across the A-frames at location directly underneath the bottom purlins. It is recommended that the new collar ties should be a minimum 175 x 75mm in cross section. Furthermore, some localised defects were found with some of the existing collar ties (**photograph 3**). Where dry-rotting and woodworm damage is found within the A-frame trusses, we recommend replacing the timbers with the same section sizes, grade C24 or hardwood.

The solid masonry walls generally appeared in good condition with localised areas of cracking and bulging.

To the eastern gable, a vertical crack was noticed approximately in line with internal face of the side elevation, which suggests that the side return along the south elevation has moved away slightly from the building (**photograph 4**). Further disorientation of the masonry stonework, and large open

joists were noticed on the southern elevation wall, adjacent to the south -east corner (**photograph 5**). This defect is likely associated with the crack to the eastern gable.

Above the window opening on the northern elevation, labelled as 'X' on **SK01, Appendix B**, the masonry appears defective with excessive open joints, large internal crack and slight leaning (**photograph 6**). It is recommended that this wall panel is taken down carefully and rebuilt.

Another masonry panel in the same room as the defective panel above was noticed. The wall panel labelled as 'B' in Sketch SK01 is bulging inwards excessively (**photograph 7**). It is recommended that this wall panel is also taken down and rebuilt.

The internal large masonry dividing walls are not keyed-in properly to the external shell. Large vertical separation gaps, approximately 25mm wide in places, can be seen between each of the internal masonry walls and the north and south side elevation, confirming the lack of tying in detail (**photograph 8**). It is recommended that the internal masonry dividing walls are physically tied in to the side elevations, by installing Helifix Helibars at 300mm vertical centres across the corner of the two walls. Please see the Product Data Sheet and Sketch SK02 within **Appendix C** for installation guidance. The same product should be installed over vertical cracks in the masonry walls, to stitch the cracks and avoid further movements.

It is recommended that all of the original pointing is raked out and repointing taken place over the whole extent of the external surfaces, using Natural Hydraulic Lime 3.5, which will increase the buildability characteristics of the walls.

It is evident that the external and internal walls have been constructed directly off the firm clay ground. It's safe to assume that the ground is adequate to support the weight of the large masonry walls, as they have been standing adequately for so long. However, in some areas the external ground levels have been lowered past the level of the base of the walls, resulting in the foundation stones and clay bearing material being exposed (**photograph 9**). It is necessary to underpin or haunch these sections of walls, as per detailed in SK03 of **Appendix D**. To improve the watertight characteristics of the building, it is also recommended to install a land drain around the external perimeter of the property to ensure free draining of ground water away from the building. We recommend installing a 100mm diameter perforated pipe in a French drain (clean stone trench) around the perimeter of the building and discharging into a soakaway a minimum distance of 5.0m away from any building.

It is recommended to construct a new timber load bearing studwork wall on the internal face of the barn in location where a first floor is to be added. The internal load bearing studwork wall would be used to support the permanent and imposed loading of the additional floor. This load bearing stud wall should be fixed to the existing masonry walls to provide additional lateral stability. The load bearing walls should consist of 140mm deep x 38mm wide C16 studs at 400mm centres and be bearing on adequate concrete slab with mesh reinforcement.

Widespread evidence of woodworm infestation was noticed in all timber lintels. Some timber lintels also suffered from dry rotting. Due to the age and condition of the timber lintels, it will be impossible to sufficiently analyse the lintels by calculations, therefore we recommend that all lintels are

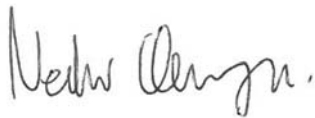
replaced with either Pre cast concrete lintels for shorter spans and Catnic/IG box lintels for longer spans.

Conclusions

Generally, as it stands from a structural perspective, the present outbuildings appear to be in adequate condition, without significant structural rectification works required, to be converted into habitable spaces. If all of our recommendations and observations above are followed in accordance with the current standards and building regulations it is anticipated that converting the barns will improve their overall lifespan.

If you have any queries regarding this report or require any further information please contact our office at the above address.

Yours sincerely



Nedw Llewelyn BEng (Hons) MSc CEng MICE

On behalf of Vale Consultancy

Enclosed:

Appendix A – Photographs

Appendix B – Calculations Package and Sketch SK01

Appendix C – Helifix Helibar Detail and Product Data Sheet

Appendix D – Underpinning Detail

Appendix A – Photographs



Photograph 1



Photograph 2



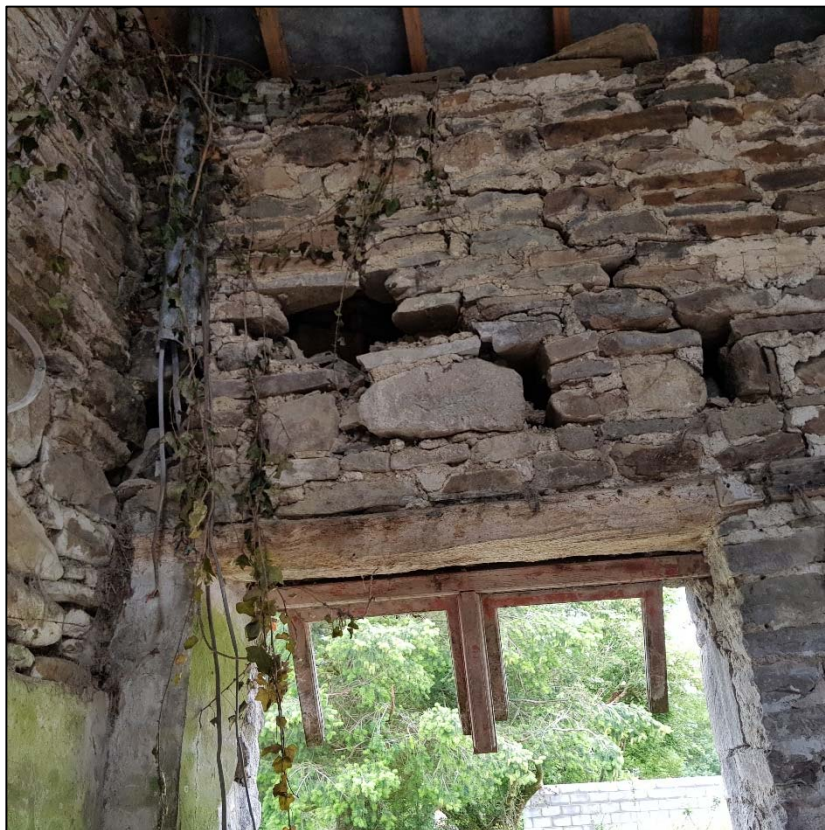
Photograph 3



Photograph 4



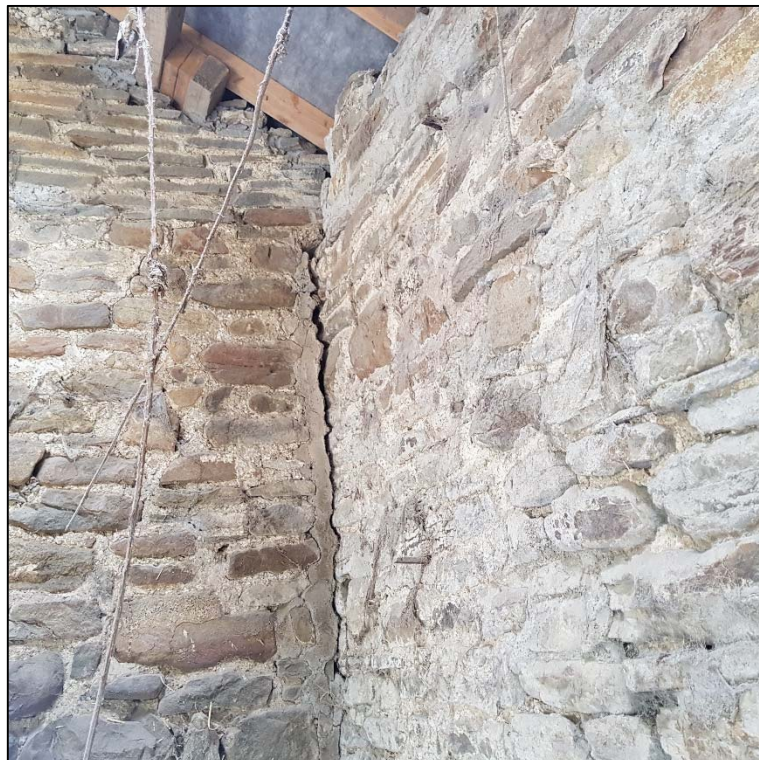
Photograph 5



Photograph 6



Photograph 7



Photograph 8



Photograph 9

Appendix B – Calculations Package and Sketch SK01



Vale Consultancy
CONSULTING CIVIL & STRUCTURAL ENGINEERS

Title: Structural Calculations for Existing Purlin
and Rafters

Project: Nant y Gaseg Uchaf Farm, Rhyd y Fro,
Pontardawe

Client: Paul Temblett

Document No: 001
Project No: 9540
Prepared By: NL
Checked By: MJ

Revision: -

Date: 08/07/2019



Timber Purlin Check.

Roof-dead

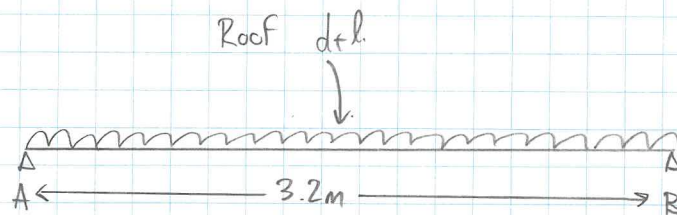
Slates	- 0.35
Battens + felt	- 0.05
Rafters	- 0.15
Insulation	- 0.05
Plasterboard + skim	- 0.15

$$0.75 / \cos 30^\circ = 0.87 \text{ kN/m}^2$$

Roof-live.

Snow. - 0.6 kN/m^2

Purlin Check.



$$\text{Roof-dead} = 0.87 \times \frac{1.63}{2} = 0.71 \text{ kN/m}$$

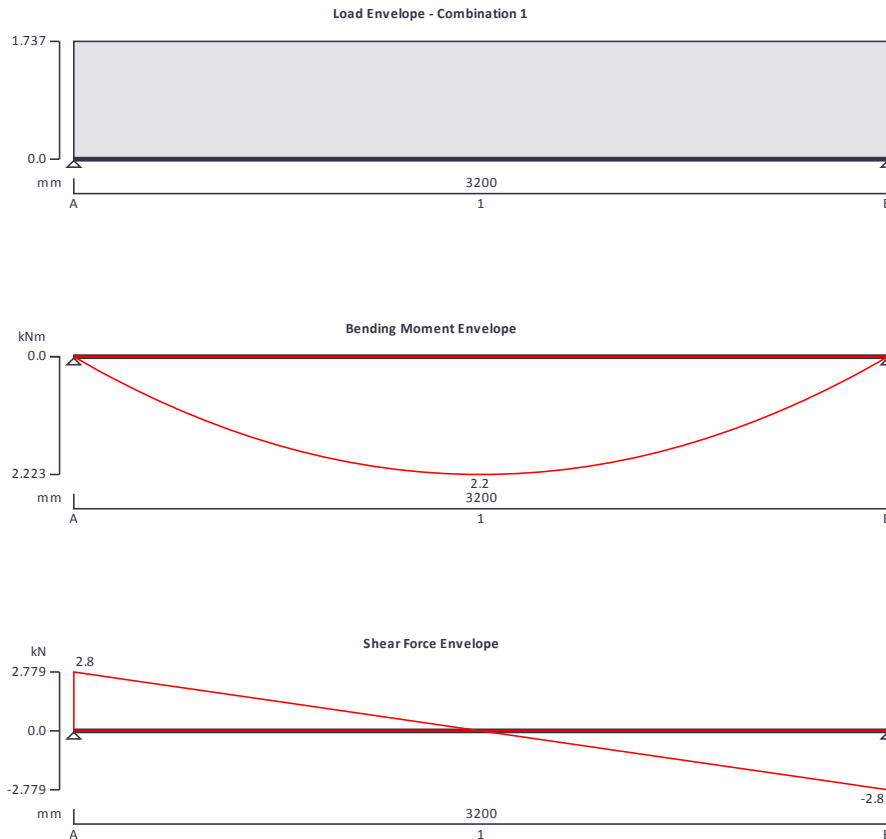
$$\text{Roof-live} = 0.6 \times \frac{1.63}{2} = 0.49 \text{ kN/m}$$

Refer to analysis = Existing purlins inadequate.

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

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.02



Applied loading

Beam loads

Roof

Permanent full UDL 0.710 kN/m

Roof

Variable full UDL 0.490 kN/m

Permanent self weight of beam $\times 1$

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Span 1

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$

Analysis results

Maximum moment

$M_{\max} = 2.223$ kNm

$M_{\min} = 0.000$ kNm

Design moment

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.223$ kNm

Maximum shear

$F_{\max} = 2.779$ kN

$F_{\min} = -2.779$ kN

Design shear

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = \mathbf{2.779 \text{ kN}}$$

Total load on beam

$$W_{\text{tot}} = \mathbf{5.558 \text{ kN}}$$

Reactions at support A

$$R_{A_{\max}} = \mathbf{2.779 \text{ kN}}$$

$$R_{A_{\min}} = \mathbf{2.779 \text{ kN}}$$

Unfactored permanent load reaction at support A

$$R_{A_{\text{Permanent}}} = \mathbf{1.187 \text{ kN}}$$

Unfactored variable load reaction at support A

$$R_{A_{\text{Variable}}} = \mathbf{0.784 \text{ kN}}$$

Reactions at support B

$$R_{B_{\max}} = \mathbf{2.779 \text{ kN}}$$

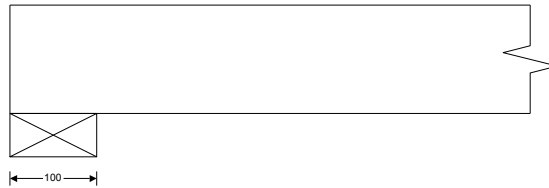
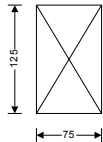
$$R_{B_{\min}} = \mathbf{2.779 \text{ kN}}$$

Unfactored permanent load reaction at support B

$$R_{B_{\text{Permanent}}} = \mathbf{1.187 \text{ kN}}$$

Unfactored variable load reaction at support B

$$R_{B_{\text{Variable}}} = \mathbf{0.784 \text{ kN}}$$



Timber section details

Breadth of timber sections

$$b = \mathbf{75 \text{ mm}}$$

Depth of timber sections

$$h = \mathbf{125 \text{ mm}}$$

Number of timber sections in member

$$N = \mathbf{1}$$

Overall breadth of timber member

$$b_b = N \times b = \mathbf{75 \text{ mm}}$$

Inclination of section

$$\theta = \mathbf{30.0 \text{ deg}}$$

Timber strength class - EN 338:2016 Table 1

$$\mathbf{C24}$$

Member details

Load duration - cl.2.3.1.2

Long-term

Service class of timber - cl.2.3.1.3

1

Length of span

$$L_{s1} = \mathbf{3200 \text{ mm}}$$

Length of bearing

$$L_b = \mathbf{100 \text{ mm}}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = \mathbf{9375 \text{ mm}^2}$$

Section modulus

$$W_y = N \times b \times h^2 / 6 = \mathbf{195312 \text{ mm}^3}$$

$$W_z = h \times (N \times b)^2 / 6 = \mathbf{117187 \text{ mm}^3}$$

Second moment of area

$$I_y = N \times b \times h^3 / 12 = \mathbf{12207031 \text{ mm}^4}$$

$$I_z = h \times (N \times b)^3 / 12 = \mathbf{4394531 \text{ mm}^4}$$

Radius of gyration

$$r_y = \sqrt{I_y / A} = \mathbf{36.1 \text{ mm}}$$

$$r_z = \sqrt{I_z / A} = \mathbf{21.7 \text{ mm}}$$

Second moment of area of rotated section

$$I_\theta = (I_y + I_z) / 2 + (I_y - I_z) \times \cos(2 \times \theta) / 2 = \mathbf{10253906 \text{ mm}^4}$$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3

$$\gamma_M = \mathbf{1.300}$$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$$k_{\text{mod}} = \mathbf{0.700}$$

Deformation factor for service classes - Table 3.2

$$k_{\text{def}} = \mathbf{0.600}$$

Depth factor for bending - exp.3.1

$$k_{h,m} = \min((150 \text{ mm} / h)^{0.2}, 1.3) = \mathbf{1.037}$$

Depth factor for tension - exp.3.1

$$k_{h,t} = \min((150 \text{ mm} / \max(b, h))^{0.2}, 1.3) = \mathbf{1.037}$$

Bending stress re-distribution factor - cl.6.1.6(2)

$$k_m = \mathbf{0.700}$$

Crack factor for shear resistance - cl.6.1.7(2)

$$k_{cr} = \mathbf{0.670}$$

Load configuration factor - exp.6.4

$$k_{c,90} = \mathbf{1.000}$$

Project Nant y Gaseg Uchaf Farm, Rhyd y Fro, Pontardawe. SA8 4SY				Job no. 9540	
Calcs for Existing Purlin Check				Start page no./Revision 4	
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System strength factor - cl.6.6

$$k_{sys} = 1.000$$

Lateral buckling factor - cl.6.3.3(5)

$$k_{crit} = 1.000$$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress

$$\sigma_{c,90,d} = R_{A,max} / (N \times b \times L_b) = 0.371 \text{ N/mm}^2$$

Design compressive strength

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.346 \text{ N/mm}^2$$

$$\sigma_{c,90,d} / f_{c,90,d} = 0.275$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Biaxial bending - cl 6.1.6

Design bending stress in major (y-y) axis

$$\sigma_{m,y,d} = M \times \cos(\theta) / W_y = 9.858 \text{ N/mm}^2$$

Design bending stress in minor (z-z) axis

$$\sigma_{m,z,d} = M \times \sin(\theta) / W_z = 9.486 \text{ N/mm}^2$$

Design bending strength

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 13.403 \text{ N/mm}^2$$

Combined bending checks - eq.6.11 & eq.6.12

$$\sigma_{m,y,d} / f_{m,d} + k_m \times \sigma_{m,z,d} / f_{m,d} = 1.231$$

$$k_m \times \sigma_{m,y,d} / f_{m,d} + \sigma_{m,z,d} / f_{m,d} = 1.223$$

FAIL - Design bending stress exceeds design bending strength

Shear - cl.6.1.7

Applied shear stress

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.664 \text{ N/mm}^2$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.154 \text{ N/mm}^2$$

$$\tau_d / f_{v,d} = 0.308$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = 12.800 \text{ mm}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = 9.160 \text{ mm}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 14.656 \text{ mm}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = 6.048 \text{ mm}$$

Factor for quasi-permanent variable action

$$\psi_2 = 0.3$$

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 7.136 \text{ mm}$$

Total final deflection

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 21.792 \text{ mm}$$

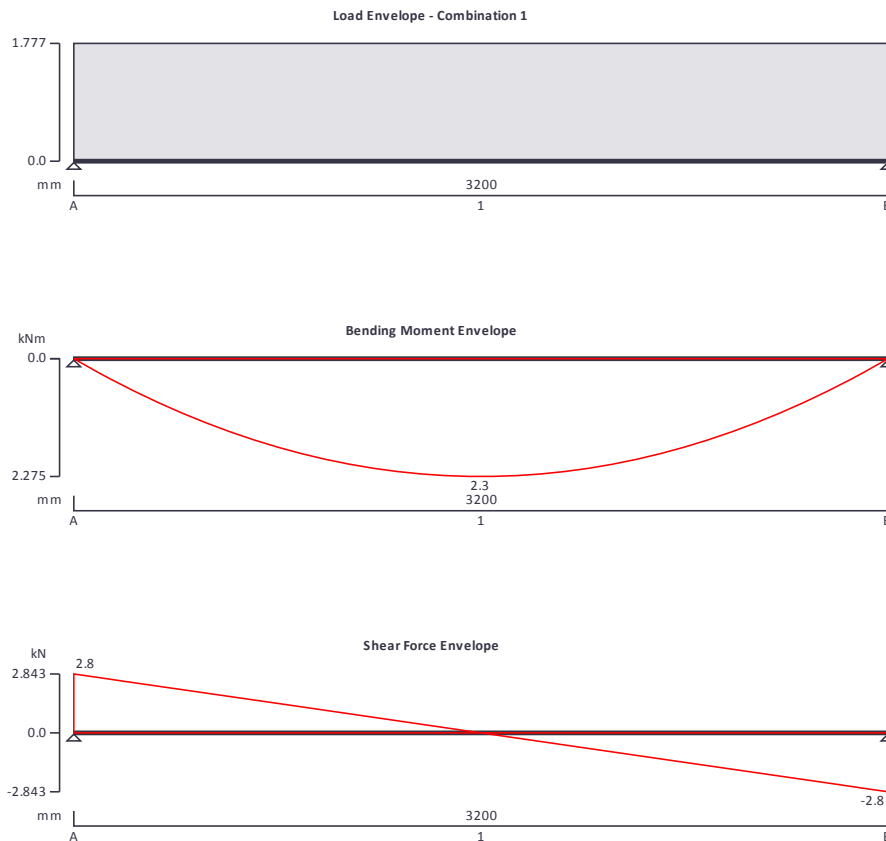
$$\delta_{fin} / \delta_{lim} = 1.703$$

FAIL - Total final deflection exceeds the deflection limit

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

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.02



Applied loading

Beam loads

Roof

Permanent full UDL 0.710 kN/m

Roof

Variable full UDL 0.490 kN/m

Permanent self weight of beam $\times 1$

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Span 1

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$

Analysis results

Maximum moment

$M_{\max} = 2.275$ kNm

$M_{\min} = 0.000$ kNm

Design moment

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.275$ kNm

Maximum shear

$F_{\max} = 2.843$ kN

$F_{\min} = -2.843$ kN

Design shear

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = \mathbf{2.843 \text{ kN}}$$

Total load on beam

$$W_{\text{tot}} = \mathbf{5.687 \text{ kN}}$$

Reactions at support A

$$R_{A_{\max}} = \mathbf{2.843 \text{ kN}}$$

$$R_{A_{\min}} = \mathbf{2.843 \text{ kN}}$$

Unfactored permanent load reaction at support A

$$R_{A_{\text{Permanent}}} = \mathbf{1.235 \text{ kN}}$$

Unfactored variable load reaction at support A

$$R_{A_{\text{Variable}}} = \mathbf{0.784 \text{ kN}}$$

Reactions at support B

$$R_{B_{\max}} = \mathbf{2.843 \text{ kN}}$$

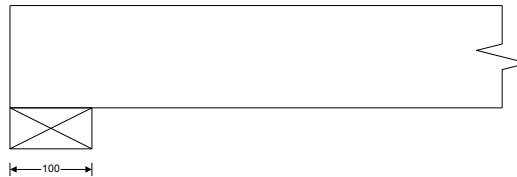
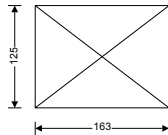
$$R_{B_{\min}} = \mathbf{2.843 \text{ kN}}$$

Unfactored permanent load reaction at support B

$$R_{B_{\text{Permanent}}} = \mathbf{1.235 \text{ kN}}$$

Unfactored variable load reaction at support B

$$R_{B_{\text{Variable}}} = \mathbf{0.784 \text{ kN}}$$



Timber section details

Breadth of timber sections

$$b = \mathbf{163 \text{ mm}}$$

Depth of timber sections

$$h = \mathbf{125 \text{ mm}}$$

Number of timber sections in member

$$N = \mathbf{1}$$

Overall breadth of timber member

$$b_b = N \times b = \mathbf{163 \text{ mm}}$$

Inclination of section

$$\theta = \mathbf{30.0 \text{ deg}}$$

Timber strength class - EN 338:2016 Table 1

$$\mathbf{C16}$$

Member details

Load duration - cl.2.3.1.2

$$\mathbf{\text{Long-term}}$$

Service class of timber - cl.2.3.1.3

$$\mathbf{1}$$

Length of span

$$L_{s1} = \mathbf{3200 \text{ mm}}$$

Length of bearing

$$L_b = \mathbf{100 \text{ mm}}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = \mathbf{20375 \text{ mm}^2}$$

Section modulus

$$W_y = N \times b \times h^2 / 6 = \mathbf{424479 \text{ mm}^3}$$

$$W_z = h \times (N \times b)^2 / 6 = \mathbf{553521 \text{ mm}^3}$$

Second moment of area

$$I_y = N \times b \times h^3 / 12 = \mathbf{26529948 \text{ mm}^4}$$

$$I_z = h \times (N \times b)^3 / 12 = \mathbf{45111948 \text{ mm}^4}$$

Radius of gyration

$$r_y = \sqrt{I_y / A} = \mathbf{36.1 \text{ mm}}$$

$$r_z = \sqrt{I_z / A} = \mathbf{47.1 \text{ mm}}$$

Second moment of area of rotated section

$$I_\theta = (I_y + I_z) / 2 + (I_y - I_z) \times \cos(2 \times \theta) / 2 = \mathbf{31175448 \text{ mm}^4}$$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3

$$\gamma_M = \mathbf{1.300}$$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$$k_{\text{mod}} = \mathbf{0.700}$$

Deformation factor for service classes - Table 3.2

$$k_{\text{def}} = \mathbf{0.600}$$

Depth factor for bending - exp.3.1

$$k_{h,m} = \min((150 \text{ mm} / h)^{0.2}, 1.3) = \mathbf{1.037}$$

Depth factor for tension - exp.3.1

$$k_{h,t} = \mathbf{1.000}$$

Bending stress re-distribution factor - cl.6.1.6(2)

$$k_m = \mathbf{0.700}$$

Crack factor for shear resistance - cl.6.1.7(2)

$$k_{cr} = \mathbf{0.670}$$

Load configuration factor - exp.6.4

$$k_{c,90} = \mathbf{1.000}$$

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Calcs for Strengthening purlin				Start page no./Revision 7	
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System strength factor - cl.6.6

$$k_{sys} = 1.000$$

Lateral buckling factor - cl.6.3.3(5)

$$k_{crit} = 1.000$$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress

$$\sigma_{c,90,d} = R_{A,max} / (N \times b \times L_b) = 0.174 \text{ N/mm}^2$$

Design compressive strength

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.185 \text{ N/mm}^2$$

$$\sigma_{c,90,d} / f_{c,90,d} = 0.147$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Biaxial bending - cl 6.1.6

Design bending stress in major (y-y) axis

$$\sigma_{m,y,d} = M \times \cos(\theta) / W_y = 4.641 \text{ N/mm}^2$$

Design bending stress in minor (z-z) axis

$$\sigma_{m,z,d} = M \times \sin(\theta) / W_z = 2.055 \text{ N/mm}^2$$

Design bending strength

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 8.935 \text{ N/mm}^2$$

Combined bending checks - eq.6.11 & eq.6.12

$$\sigma_{m,y,d} / f_{m,d} + k_m \times \sigma_{m,z,d} / f_{m,d} = 0.680$$

$$k_m \times \sigma_{m,y,d} / f_{m,d} + \sigma_{m,z,d} / f_{m,d} = 0.594$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.312 \text{ N/mm}^2$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.723 \text{ N/mm}^2$$

$$\tau_d / f_{v,d} = 0.181$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = 12.800 \text{ mm}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = 4.342 \text{ mm}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 6.948 \text{ mm}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = 2.756 \text{ mm}$$

Factor for quasi-permanent variable action

$$\psi_2 = 0.3$$

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 3.252 \text{ mm}$$

Total final deflection

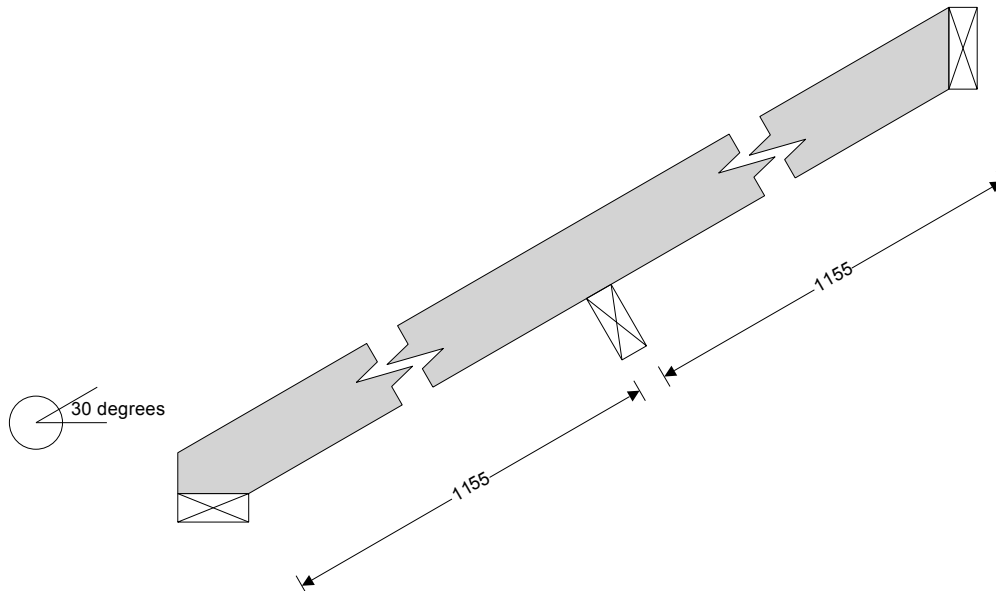
$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 10.200 \text{ mm}$$

$$\delta_{fin} / \delta_{lim} = 0.797$$

PASS - Total final deflection is less than the deflection limit

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections
Depth of timber sections
Rafter spacing
Rafter slope
Clear span of rafter on horizontal
Clear span of rafter on slope
Rafter span
Timber strength class

$b = 40 \text{ mm}$
 $h = 100 \text{ mm}$
 $s = 350 \text{ mm}$
 $\alpha = 30.0 \text{ deg}$
 $L_{clh} = 1000 \text{ mm}$
 $L_{cl} = L_{clh} / \cos(\alpha) = 1155 \text{ mm}$
Continuous
C16

Section properties

Cross sectional area of rafter
Section modulus
Second moment of area
Radius of gyration

$A = b \times h = 4000 \text{ mm}^2$
 $Z = b \times h^2 / 6 = 66667 \text{ mm}^3$
 $I = b \times h^3 / 12 = 3333333 \text{ mm}^4$
 $r = \sqrt{I / A} = 28.9 \text{ mm}$

Loading details

Rafter self weight
Dead load on slope
Imposed load on plan
Imposed point load

$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.01 \text{ kN/m}$
 $F_d = 0.80 \text{ kN/m}^2$
 $F_u = 0.60 \text{ kN/m}^2$
 $F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor
Load sharing factor

$K_7 = (300 \text{ mm} / h)^{0.11} = 1.13$
 $K_8 = 1.10$

Consider long term load condition

Load duration factor
Total UDL perpendicular to rafter
Notional bearing length

$K_3 = 1.00$
 $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.253 \text{ kN/m}$
 $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 2 \text{ mm}$

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Effective span

$$L_{eff} = L_{cl} + L_b = 1156 \text{ mm}$$

Check bending stress at purlin

Bending stress parallel to grain

$$\sigma_m = 5.300 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 6.579 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 0.634 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain

$$\sigma_c = 6.800 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 5800 \text{ N/mm}^2$$

Compression member factor

$$K_{12} = 0.79$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.941 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A) = 0.090 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 35.684 \text{ N/mm}^2$$

Euler coefficient

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

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.112 < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain

$$\sigma_m = 5.300 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 6.579 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = 9 \times F \times L_{eff}^2 / (128 \times Z) = 0.357 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain

$$\sigma_c = 6.800 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 5800 \text{ N/mm}^2$$

Compression member factor

$$K_{12} = 0.79$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.941 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 13 \times \tan(\alpha) / 3) / (8 \times A) = 0.116 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

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

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 35.684 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.996$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.074 < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) = 0.069 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 3.469 \text{ mm}$$

Bending deflection

$$\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I) = 0.083 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.023 \text{ mm}$$

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Total deflection

$$\delta_{\max} = \delta_b + \delta_s = \mathbf{0.106 \text{ mm}}$$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor

$$K_3 = \mathbf{1.25}$$

Total UDL perpendicular to rafter

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = \mathbf{0.411 \text{ kN/m}}$$

Notional bearing length

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{2 \text{ mm}}$$

Effective span

$$L_{eff} = L_{cl} + L_b = \mathbf{1157 \text{ mm}}$$

Check bending stress at purlin

Bending stress parallel to grain

$$\sigma_m = \mathbf{5.300 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.224 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{1.031 \text{ N/mm}^2}$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain

$$\sigma_c = \mathbf{6.800 \text{ N/mm}^2}$$

Minimum modulus of elasticity

$$E_{min} = \mathbf{5800 \text{ N/mm}^2}$$

Compression member factor

$$K_{12} = \mathbf{0.78}$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.314 \text{ N/mm}^2}$$

Applied compressive stress

$$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A) = \mathbf{0.146 \text{ N/mm}^2}$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{35.625 \text{ N/mm}^2}$$

Euler coefficient

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

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.146 < 1}$$

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain

$$\sigma_m = \mathbf{5.300 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.224 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_max} = 9 \times F \times L_{eff}^2 / (128 \times Z) = \mathbf{0.580 \text{ N/mm}^2}$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain

$$\sigma_c = \mathbf{6.800 \text{ N/mm}^2}$$

Minimum modulus of elasticity

$$E_{min} = \mathbf{5800 \text{ N/mm}^2}$$

Compression member factor

$$K_{12} = \mathbf{0.78}$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.314 \text{ N/mm}^2}$$

Applied compressive stress

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

PASS - Applied compressive stress within permissible limits

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

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{35.625 \text{ N/mm}^2}$$

Euler coefficient

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

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.097 < 1}$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

$$\tau = \mathbf{0.670 \text{ N/mm}^2}$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.921 \text{ N/mm}^2}$$

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Applied shear stress

$$\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) = \mathbf{0.111 \text{ N/mm}^2}$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{3.471 \text{ mm}}$$

Bending deflection

$$\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I) = \mathbf{0.136 \text{ mm}}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.037 \text{ mm}}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = \mathbf{0.173 \text{ mm}}$$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor

$$K_3 = \mathbf{1.50}$$

Total UDL perpendicular to rafter

$$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.253 \text{ kN/m}}$$

Notional bearing length

$$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{6 \text{ mm}}$$

Effective span

$$L_{eff} = L_{cl} + L_b = \mathbf{1160 \text{ mm}}$$

Check bending stress at purlin

Bending stress parallel to grain

$$\sigma_m = \mathbf{5.300 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{9.868 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + 3 \times F_p \times \cos(\alpha) \times L_{eff} / (32 \times Z) = \mathbf{1.910 \text{ N/mm}^2}$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain

$$\sigma_c = \mathbf{6.800 \text{ N/mm}^2}$$

Minimum modulus of elasticity

$$E_{min} = \mathbf{5800 \text{ N/mm}^2}$$

Compression member factor

$$K_{12} = \mathbf{0.77}$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{8.625 \text{ N/mm}^2}$$

Applied compressive stress
N/mm²

$$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A) + F_p \times \sin(\alpha) / A = \mathbf{0.203}$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{35.436 \text{ N/mm}^2}$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.993}$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.218 < 1}$$

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain

$$\sigma_m = \mathbf{5.300 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{9.868 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (16 \times Z) + 13 \times F_p \times \cos(\alpha) \times L_{eff} / (64 \times Z) = \mathbf{3.075 \text{ N/mm}^2}$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain

$$\sigma_c = \mathbf{6.800 \text{ N/mm}^2}$$

Minimum modulus of elasticity

$$E_{min} = \mathbf{5800 \text{ N/mm}^2}$$

Compression member factor

$$K_{12} = \mathbf{0.77}$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{8.625 \text{ N/mm}^2}$$

Applied compressive stress

$$\sigma_{c_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 4 \times \tan(\alpha)) / (8 \times A) + F_p \times \sin(\alpha) / A = \mathbf{0.224 \text{ N/mm}^2}$$

PASS - Applied compressive stress within permissible limits

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

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{35.436 \text{ N/mm}^2}$

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.993}$

Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.340 < 1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.670 \text{ N/mm}^2}$

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{1.106 \text{ N/mm}^2}$

Applied shear stress $\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = \mathbf{0.361 \text{ N/mm}^2}$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{3.481 \text{ mm}}$

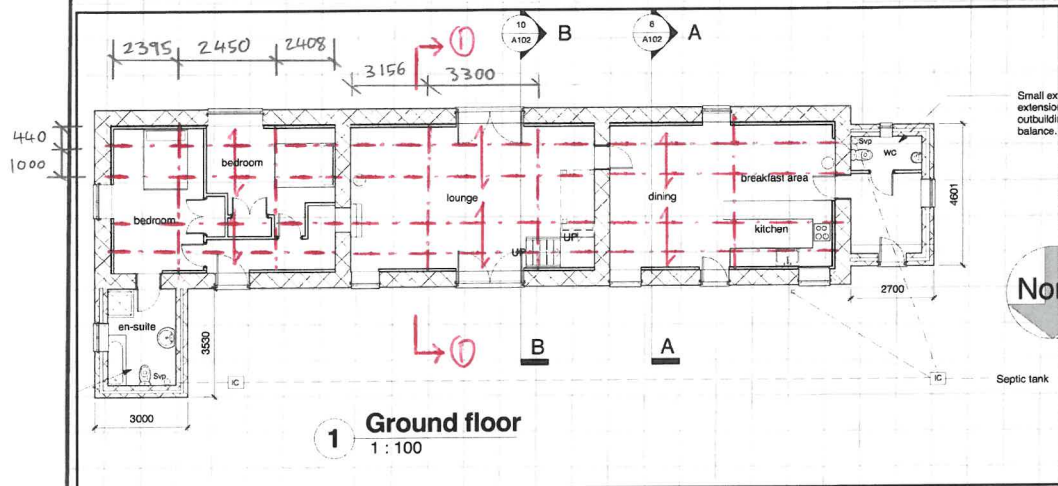
Bending deflection $\delta_b = L_{eff}^3 \times (F \times L_{eff} / 185 + 0.015 \times F_p \times \cos(\alpha)) / (E_{mean} \times I) = \mathbf{0.707 \text{ mm}}$

Shear deflection $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = \mathbf{0.147 \text{ mm}}$

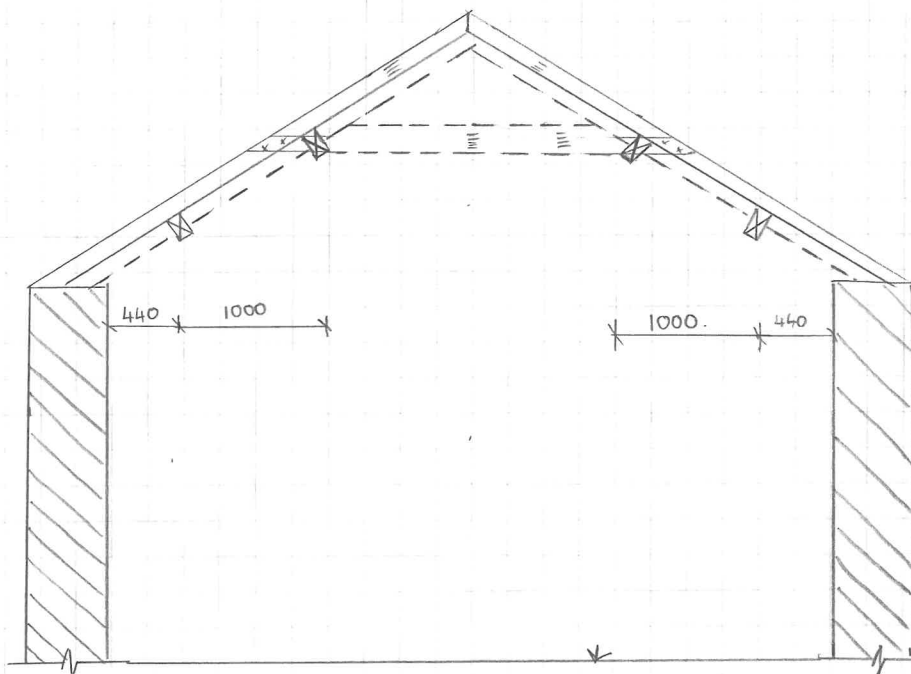
Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{0.854 \text{ mm}}$

PASS - Total deflection within permissible limits

Nant y Gaseg Uchaf Farm, Rhyd y Fro, Pontardawe.



Rafters = 100x40mm @ 350mm c/c.
Purlins = 125x75mm Timbers
A-Frame = 175x100mm Timbers.



Section 1-1

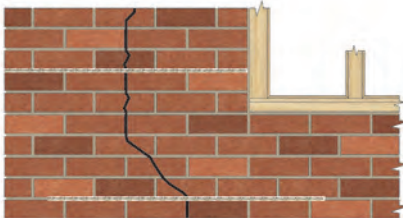
Appendix C – Helifix Helibar Detail and Product Data Sheet

HeliBar

Helical stainless steel reinforcing bar for masonry repair and strengthening in both remedial and new build situations

Applications

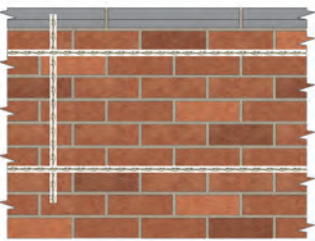
- Crack stitching
- Lintel repair and creation
- Forming deep masonry beams
- Horizontal structural restraint (when used with BowTie systems)
- Reconnecting separated walls
- Securing parapet walls
- Support existing masonry when creating new openings
- Creating movement joints
- Reinforcing new build masonry
- Seismic upgrades for existing masonry
- Repairing bridges, tunnels and arches



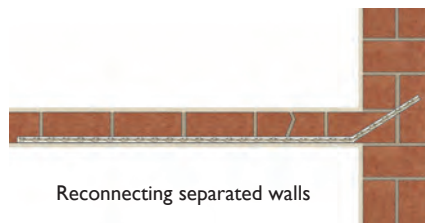
Crack stitching



Lintel reinstatement



Securing parapet walls



Reconnecting separated walls

Over 100 standard repair specifications are available online, covering all common structural faults.

Relevant Repair Details: BW01 & BW02; CS01 to CS14 (except 4, 7, 9, 11 & 12); LB01 to LB05; LR01 to LR11; MA03 to MA05; MJ01 to MJ03; PW01 to PW03; RB06; RF07 & RF09; RV01, 3 & 6;



For full Product Information, Case Studies and downloadable Repair Details go to:

www.helifix.co.uk/products/remedial-products/helibar



Features

- Austenitic stainless steel helical bars
- Combines great axial strength with flexibility
- Accommodates differential building movement
- No additional stresses introduced into structure
- Generates high tensile strength with mortar (new build only) or HeliBond grout
- Extremely economical compared with alternative methods
- May remove or reduce the need for mass underpinning
- Fully concealed once installed
- Avoids expensive taking down and rebuilding
- Minimal disruption to building's fabric or occupants
- Spreads structural loads to avoid secondary cracking
- Reduces the potential for cracking in shrinkable materials



HeliBar is inserted into HeliBond grout within a cut slot

Installation Procedures

1. HeliBar to be long enough to extend a minimum of 500mm either side of the crack or 500mm beyond the outer cracks if two or more adjacent cracks are being stitched using one rod.
2. Where a crack is less than 500mm from the end of a wall or an opening, the HeliBar is to be continued for at least 200mm around the corner and bonded into the adjoining wall or bent back and fixed into the reveal, avoiding any DPC.
3. For solid masonry in excess of 225mm thick and in a cavity wall where both leaves are cracked, the wall must be crack stitched on both sides.
4. If there is render/plaster, this thickness must be added to the depth of slot. Crack stitching must be installed in the masonry and never in the render.
5. Ensure the masonry is well wetted or primed to prevent premature drying of the HeliBond due to rapid de-watering, especially in hot conditions. Ideally additional wetting of the slot should be carried out 1 to 2 minutes prior to injecting the HeliBond grout.
6. Do not use HeliBond when the air temperature is +4°C and falling or apply over ice. In all instances the slot must be thoroughly damp or primed prior to injection of the HeliBond grout.



1. Rake out or cut slots into the horizontal mortar beds, a minimum of 500mm either side of the crack



4. Using the HeliBar Insertion Tool push one HeliBar into the grout to obtain good coverage



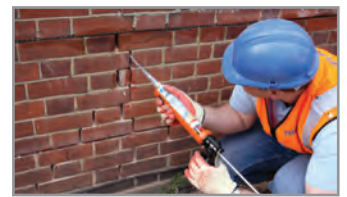
2. Clean out slots and flush with clean water and thoroughly soak the substrate within the slot



5. Insert a further bead of HeliBond over the exposed HeliBar, finishing 12mm from face and 'iron' firmly into the slot using the HeliBar Insertion Tool



3. Using the Helifix Pointing Gun, inject a bead of HeliBond along the back of the slot



6. Re-point the mortar bed and make good the vertical crack with CrackBond TE3

Slot Depth and Spacing

	Single skin/ Cavity wall	Solid Masonry		
		Up to 102.5mm	102.5mm to 225mm	Over 225mm
Depth of slot	25 – 35mm		25 – 40 mm	25 – 40mm On both sides
Vertical Spacing	Every 4 – 6 courses, 300 – 450mm			

Technical Specifications

Material:	Austenitic stainless steel Grade 304 (1.4301) or 316 (1.4401)
Diameter:	4.5mm, 6mm, 8mm and 10mm
Tensile strength (6mm HeliBar):	10kN
0.2% Proof stress (6mm HeliBar):	900 N/mm ² (304 / 1.4301) 840 N/mm ² (316 / 1.4401)
Standard lengths:	1m, 1.5m & 2m – in packs of 10
Width of slot:	Full height of bed joint (10mm in render/plaster)
Bonding agent:	HeliBond cementitious grout. 1 x 3ltr HeliBond = 10 linear metres of crack stitching
RECOMMENDED TOOLING	
For cutting slot up to 40mm deep:	Twin-bladed cutter with vacuum attachment or angle grinder or hammer and mortar chisel
For mixing HeliBond grout:	3-jaw-chuck drill with mixing paddle
For injection of HeliBond into slots:	Helifix Pointing Gun CS with mortar nozzle
For smoothing pointing:	Standard finger trowel
For inserting HeliBar:	HeliBar Insertion Tool

Appendix D – Underpinning Detail

