



Title: **Lacock Abbey**
 Dining Room Roof

Job No: **22289**

Date: **September 2025**

Client: **The National Trust**

Copyright: Hockley and Dawson Consulting Engineers

This Report has been prepared for the use of the above client.

This document shall not be reproduced in whole or in part or relied upon by third parties for any use whatsoever without the express written authority of Hockley and Dawson Consulting Engineers.

Hockley and Dawson Consulting Engineers shall have no liability for any use of the document other than the purpose for which it was originally prepared.

1.0 Introduction and Brief

Following on from the initial inspection of the cracking across the dining room ceiling and the ceiling structure, a follow up survey was carried out of the roof structure over. The preliminary analysis indicated that the overall loading and deflection of the roof trusses could have been contributing to the ceiling issues, and a measured survey was required to further develop these calculations.

2.0 Description

The In general the principal roof truss members appeared to be in a reasonable condition. There were indications of historic alterations to the structure, and possibly that there had originally been different floor / ceiling levels within the area, with vacant pockets in the external masonry walls. It was unclear whether the roof timbers had been reused from the original scheme and integrated into the later trussed valley system.

The existing truss is set out as shown on the elevation below, with all timber elements in red, and the steel tension rods in blue. The roof loads are transferred to the truss through the three principal rafters as shown. The bottom tie beam supports this roof structure through the braces, and also the ceiling structure directly. Measurements were taken for all the elements shown which were then inputted into the structural analysis model.

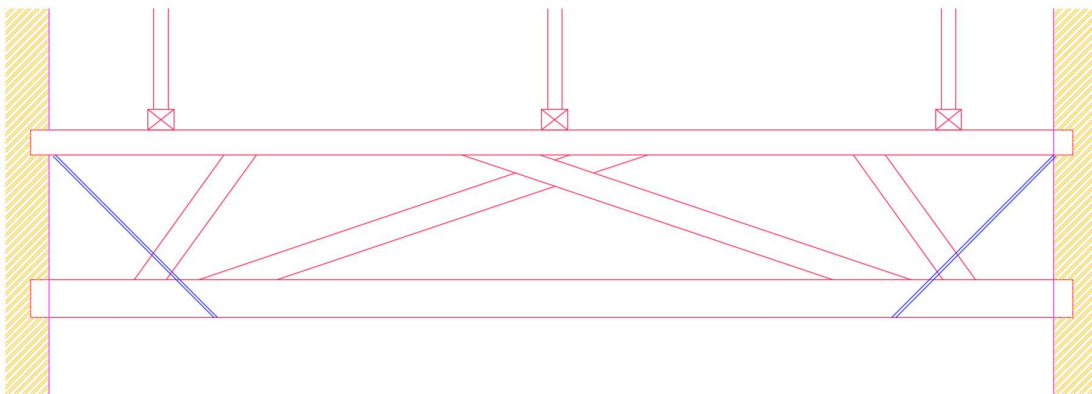


Figure 1 - Existing elevation of girder truss

Whilst there were three sets of struts, ties, or braces, it was noted that these primarily provided support to the top beam member, focussing the stresses towards the end of the bottom tie beam. There was no intermediate support to the lower tie beams.

3.0 Structural Analysis

Diagram and table extracts from the structural analyses can be found in Appendix A. The timbers were not regularised, and had varying section sizes. For the purposes of the analysis, typical dimensions were used, and notches or defects ignored.

The existing structure was modelled, and found to have excessive deflection of the lower tie beam. The upper beam was also slightly over the allowable limits. These recommended limits are based on $\text{span} / 360$ for all loading, or $\text{span} / 250$ for imposed load only. The stresses and worst case forces from the girder analysis were reviewed for the principal beam members, and both the upper and lower tie beams passed these checks. It should be noted that the lower tie beam was found to be running at 98% capacity in this analysis.

No imposed load was applied at ceiling level, which should be considered to allow for maintenance access. It should also be noted that an imposed load of 0.75kN/m^2 was applied to the roof, which may not be sufficient for snow loading given the profile of the roof and risk of build up in the valleys. Should increased imposed loads be applied to reflect these variations then it is likely that the lower tie beam would fail the combined stress checks.

Given that the individual timbers passed the initial checks, the issue under consideration was the overall deflection of the truss system. The proposed model introduced a pair of tension rods or wires to support the bottom tie, hanging it from the top member and transferring the stresses back through the existing system of struts. The results of this proposed variation can be seen in detail in Appendix A, but the results of the two systems has been summarised in the tables below. This snapshot is based on one set of equivalent models, and pin-pin base supports. The timber grade has conservatively been taken as a hardwood grade D30, and typical section sizes for the members were used. The survey found the range of section sizes to generally be within 20mm across similar members.

Upper Tie Beam

Truss	Deflection	Bending Moment	Shear Force	Axial
Existing	22mm	9.8kNm	11.9kN	22.0kN
Proposed	6.3mm	6.8kNm	13.0kN	35.8kN
	-15.7mm	-3kNm	+1.1kN	+13.8kN

Lower Tie Beam

Truss	Deflection	Bending Moment	Shear Force	Axial
Existing	44mm	30.4kNm	26.3kN	43.5kN
Proposed	5.1mm	7.5kNm	12.3kN	63.9kN
	-38.9mm	-22.9kNm	-14kN	+20.4kN

Updated member analyses were carried out using the results above. Whilst the axial forces in the members increases, the combined tension and bending check for the tie beam reduces from 99% capacity to just 37%. As shown in the tables, the overall deflection of the bottom beam also significantly reduces.

4.0 Proposed Works

The proposed additional tension members could be introduced by way of steel brackets and stainless steel ropes. This would ensure the minimal disturbance to the historic fabric, and that (in general) the works would be reversible. Due to the assumed direct fixing of the lathe and plaster ceiling finishes to the underside of the tie beam, it may not be suitable to introduce shoe brackets to the tie beams. If this is the case then the supporting brackets could be face fixed – similar to the “bottom bracket type 3” in Appendix C2. This option would have the advantage of minimal disturbance of the ceiling finishes, however it would rely on multiple bolts through the beam. Provisional sizing would suggest 4No. M20 bolts would be required.

The first page of details included in Appendix C1 are extracts from a previous similar stabilisation scheme that was carried out where there were concerns of roof truss connections and overall deflections. The profile of the Lacock girder truss is much simpler, and the inverted shoe brackets could be used for all of the top fixings.

If the general methodology of brackets and tension ropes is approved then the specifics of the brackets and fixings can be refined to suit the exact stresses and section sizes of the Lacock trusses.

5.0 Conclusions and Recommendations

The existing roof structure fails in deflection, and the theoretical deflection of the bottom tie beam is likely to be the primary cause of cracking across the dining room ceiling. Where the cracking is noticeably worse the secondary ceiling beam has failed at the end bearing. All members analysed passed the typical stress checks, however some elements are running at very high capacity for combined tension and bending.

To increase the stability of the ceiling and the overall roof structure, two sets of tension ropes could be added to each girder truss as shown on the proposed elevation. For the installation of these brackets, either local sections of the ceiling finishes would need to be cut back and reinstated, or the bolts would need to be introduced through the bottom tie beam. Please note that the bracket type 2 option (patch repair of ceiling) is only suitable if there is sufficient depth of finishes to allow for the bracket – estimated to be 15mm thick.

The proposed system avoids the use of any long rigid sections such as full length steel flitch plates. Due to access restrictions for the roof space, typical repairs such as these would necessitate the removal of roof tiles and rafters to allow for the installation. The brackets and steel ropes could instead be installed via the existing access hatch.

For both options a crash deck or other form of temporary works would be required in the dining room to allow for the safe installation of the system.

6.0 Standard Conditions

This report is the property of and is confidential to the client designated in the report. Whilst it may be shown to the client's professional advisors, the contents are not to be disclosed to, or made use of, by any third party, without our express written consent. Without such consent, we can accept no responsibility to any third party.

Whilst every effort will be made to fully inspect those parts of the building requested of us to inspect, no permanent or secured fixtures and fittings will have been removed. We will not have inspected woodwork or other parts of the structure which were covered, unexposed or inaccessible and we are unable to report that any such part of the property is free from defect.

Hockley and Dawson certify that they have carried out the works contained herein with due care and diligence to their best belief and knowledge based on the time and information available.

This report is made on behalf of Hockley and Dawson. By receiving it and acting on it, the client – or any third party relying on it – accepts that no individual is personally liable in contract, tort or breach of statutory duty (including negligence).

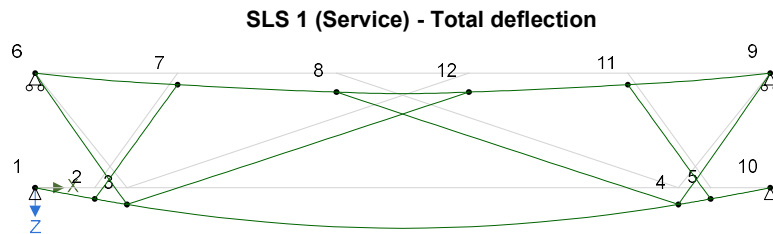
7.1 Appendix A – Calculations

ANALYSIS

Tedds calculation version 1.0.38

Results

Total deflection



Node deflections

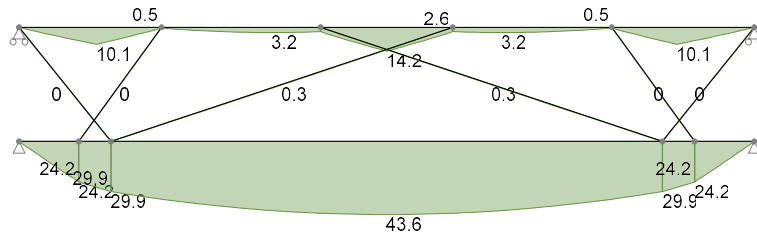
Load combination: SLS 1 (Service)

Node	Deflection		Rotation (°)	Co- ordinate system
	X (mm)	Z (mm)		
1	0	0	1.05879	
2	0	12.3	0.99685	
3	-0.1	18.2	0.92285	
4	0.1	18.2	-0.92285	
5	0	12.3	-0.99685	
6	0.1	0	0.58054	
7	0	12.4	0.32563	
8	0	20.5	0.19015	
9	-0.1	0	-0.58054	
10	0	0	-1.05879	
11	0	12.4	-0.32563	
12	0	20.5	-0.19015	

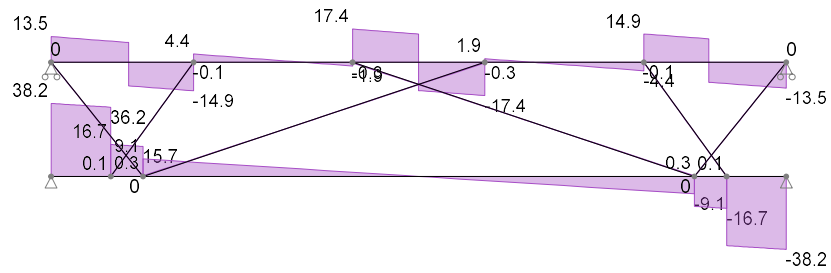
Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
SLS 1 (Service)	0	89.7
ULS 1 (Strength)	0	130.2

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



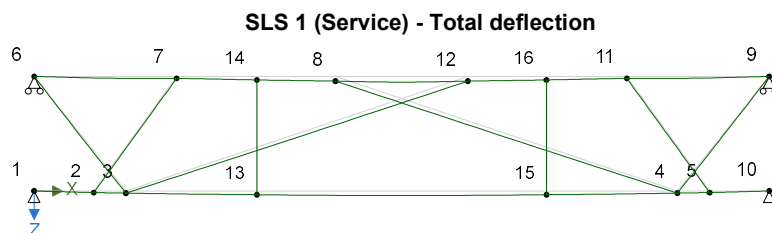
;
;

ANALYSIS

Tedds calculation version 1.0.38

Results

Total deflection



Node deflections

Load combination: SLS 1 (Service)

Node	Deflection		Rotation (°)	Co- ordinate system
	X (mm)	Z (mm)		
1	0	0	0.15244	
2	-0.1	1.8	0.12411	
3	-0.1	2.4	0.10113	
4	0.1	2.4	-0.10113	
5	0.1	1.8	-0.12411	
6	0.2	0	0.15528	
7	0.1	2.1	0.05726	
8	0	5.2	0.09484	
9	-0.2	0	-0.15528	
10	0	0	-0.15244	
11	-0.1	2.1	-0.05726	
12	0	5.2	-0.09484	
13	0	4.2	0.05327	
14	0	3.9	0.10523	
15	0	4.2	-0.05327	
16	0	3.9	-0.10523	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
SLS 1 (Service)	0	89.8
ULS 1 (Strength)	0	130.3

7.2 Appendix B – Photographs



Dining room ceiling, with visible cracking along the midline. Worst cracking located above the scaffold tower position.



Girder truss beneath valley with roof structure over



End of girder truss showing arrangement of struts and tension rod



Tension rod through the strut



Internal face of eastern wall showing redundant pockets in wall plate – suggesting possible previous ceiling level.

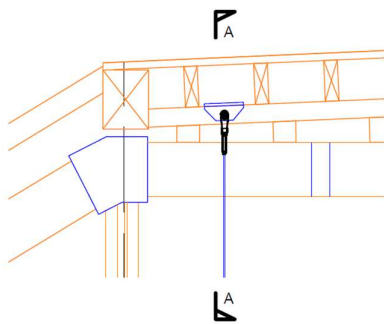
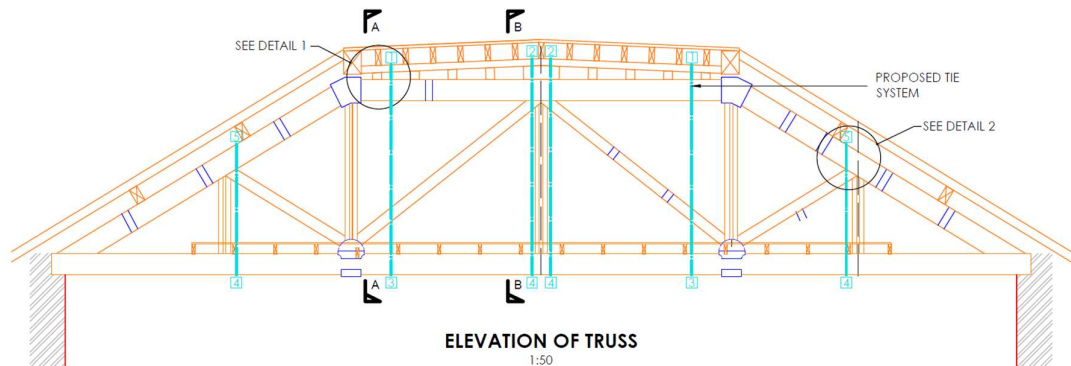


Northern wall with cracking beneath bearing of girder truss.

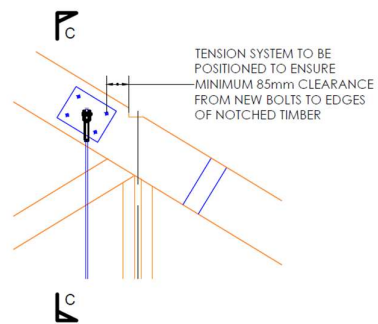


Historic strapping repair to failed secondary ceiling beam – see previous report for details.

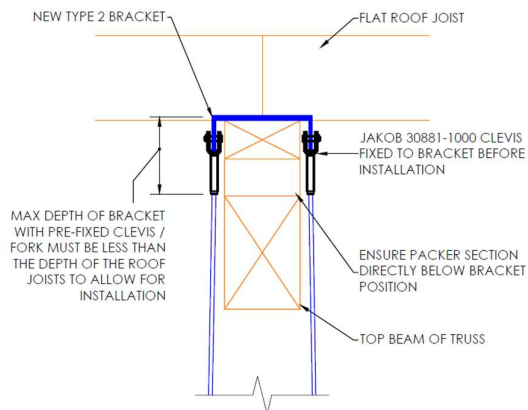
7.3 Appendix C1 – Example Tension System with Details



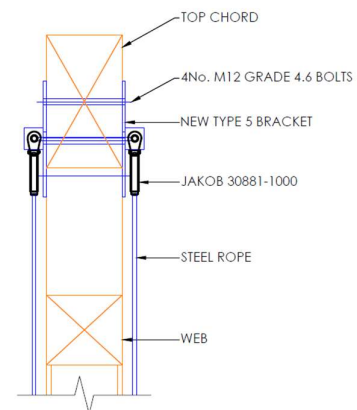
DETAIL 1
1:20



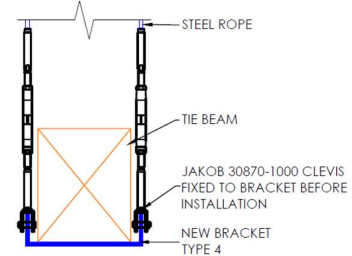
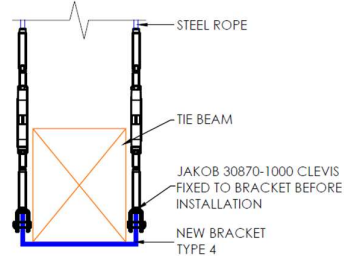
DETAIL 2
1:20



SECTION B-B
1:10



SECTION C-C
1:10



7.4 Appendix C2 – Proposed Elevation with Example Brackets

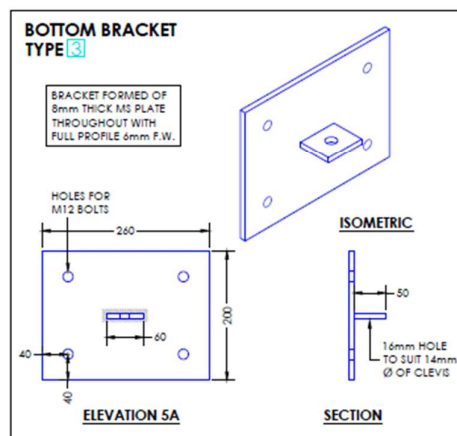
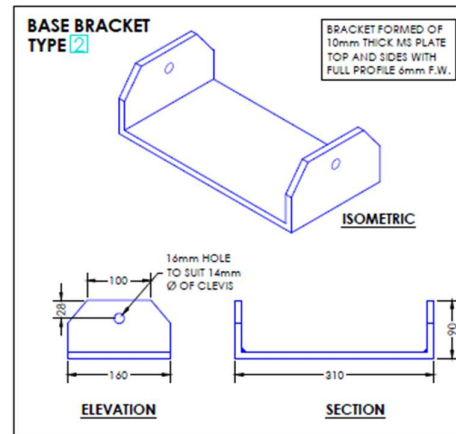
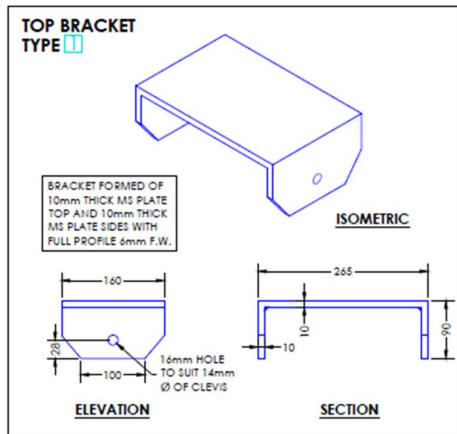
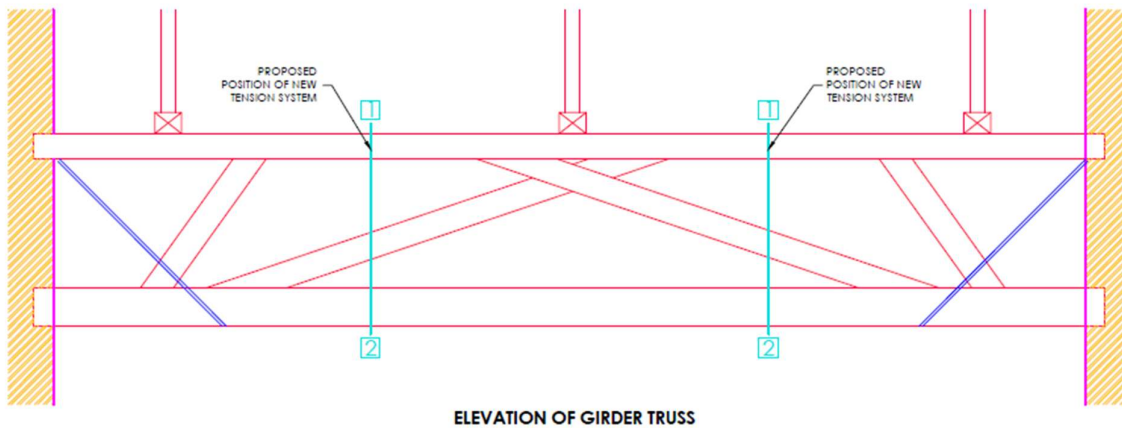


PLATE DIMENSIONS AND FIXING DETAILS SUBJECT TO CALCULATED LOADINGS FOR SPECIFIC TRUSS REQUIREMENTS. SYSTEMS SHOWN HAVE BEEN INCLUDED FOR INDICATIVE PURPOSES AND TO DEMONSTRATE THE ALTERNATIVE METHODS OF SUPPORT.

FOR MINIMAL IMPACT ON HISTORIC TIMBERS, BASE BRACKET TYPE 2 IS RECOMMENDED, HOWEVER THIS WOULD REQUIRE LOCAL CUTTING OUT OF THE PLASTER FINISHES. IF THERE IS INSUFFICIENT SPACE BETWEEN THE UNDERSIDE OF THE BEAM TO ALLOW FOR PLATE AND FINISHES, THEN LOCAL NOTCHING OF THE BEAM WOULD BE NECESSARY.

THE ALTERNATIVE BASE SUPPORT WOULD BE SIMILAR TO BRACKET TYPE 3, FIXED TO THE BOTTOM TIE BEAM WITH A MINIMUM OF 4x16 BOLTS.

FOR BOTH OPTIONS, THE TOP BRACKET WOULD BE AS TYPE 1, WHICH COULD FIT BETWEEN RAFTERS AND BENEATH THE VALLEY BOXING. THIS WOULD NOT REQUIRE ANY MECHANICAL FIXINGS INTO THE TOP TIE BEAM.