OVINGHAM FIRST SCHOOL

Structural Engineering Report

Structural report into the integrity and condition of the existing building, based on intrusive inspection and subsequent detailed calculations.

Jasper Kerr Consulting Engineers
info@jasperkerr.co.uk

Jasper Kerr Consulting Engineers Limited, Registered Address: 5 Turbinia Gardens, Cochrane Park, Newcastle upon Tyne. NE7 7LP. Registered Number 10544777 England.

Unit 7, Riverside Studios, Amethyst Road, Newcastle upon Tyne NE4 7YL

Phone; 07788628799 | Simon.Brent@jasperkerr.co.uk

Our ref; JK-6289-SR-02

Ovingham First School, General Condition & Structural Capacity

Intrusive Structural Inspection

Introduction

Following the initial visual inspection and subsequent report, completed in November, a mark-up was produced identifying strategic locations for intrusive opening-up works. These were completed by Nick Hogg and inspected by Simon Brent on 9th December 2020. Some additional openings were formed during the survey, and one subsequently, following assessment of the structure visible via the previously-formed openings.

The purpose of these works was to give a greater level of certainty as well as a greater depth of understanding regarding the structural integrity of the building. It is important to note that the conclusions from this report will not be exhaustive as there is still a significant amount of the structure which is not visible for inspection (eg. foundations). The inspection and subsequent calculations were targeted primarily towards elements which would cause significant and imminent risk if under-designed or over-loaded; for example, inadequate foundations may cause gradual settlement but an over-capacity roof joist could fail suddenly causing immediate risk to people beneath, and as a result this report focusses on the latter.

This report should be read in conjunction with Jasper Kerr Report JK-6289-SR-01, 'General Condition & Serviceability'. The conclusions and recommendations from this initial report are still valid, and are not repeated here.

Figure 1 - Eastern elevation of Ovingham First School, showing the Southern corner of the building which includes the timber-framed play area.

Observations

The structural form of the roof was observed in multiple locations, via openings formed through the ceiling below. In general, shallow and regularly-spaced joists span a short distance to primary members. These joists are fairly small in cross-section, but the short distance between supports results in a reasonable span:depth ratio. In most areas there is a separate layer of joists below, purely to support the suspended ceiling and MEP services such as lighting and cables. Again, these joists are small, but only spanning a relatively short distance.

The primary support for these joists is often composite timber I-Sections. Although generally designed to span longer distances than basic joists, the wider spacing of these elements observed in-situ, alongside the large clear spans, raises concerns as to the capacity of these members. These I-Sections are generally specialist elements, designed by independent manufacturers. For the purposes of this report, manufacturer's literature has been consulted as well as calculations carried out based on simplifications of the members. This is expanded upon further in the following section.

In the larger rooms of the school, for example the kitchen and main hall, glulam beams are present, splitting the span of the rooms and shortening the length of the aforementioned I-Sections. Members such as these have the capability to span significant distances, although the sizes observed appear relatively small and raise some initial concerns for potential over-loading. Again, this is expanded upon and further justified in the following section.

In general, the condition of the timber members was good. On inspection, the roof structure does not appear as old as the building itself, based on the construction dates provided by the school. It may be that the roof has been replaced at some point, or simply that it has been well-preserved through good waterproofing and ventilation. There are areas where small sections have been cut through the timbers, for cable runs or otherwise, but these are not extensive and are not considered to compromise the structural integrity of the members.

The roof was observed externally as well, from above. This established no obvious concerns regarding damage, deflection, or obvious defects to the waterproofing membrane. There were, however, general deviations in the level of the roof, which could lead to localised ponding. The survey followed a period of relatively damp weather and extensive ponding was not evident, therefore there are no significant and immediate concerns in this regard, however timber does continue to deflect over time ('creep') and is susceptible to deterioration through damp and rot, meaning the issue of ponding is likely to progress over time. This is exacerbated by the relatively shallow roof pitch and the very small gradient visible in the gutters. This inspection of the roof provided a further level of insight for calculation also, which is expanded upon below.

Calculations

Following assessment of the roof build-up, viewed from above as well as through the openings formed below, a more accurate assessment of the self-weight could be established. There are still elements of judgement, for example the quantity and density of insulation or the self-weight of the waterproof membrane, but this analysis allows a more accurate assessment of the supporting structural members. Detailed analysis was also carried out to determine the possible snow loads which may accumulate on the roof, including drift projections where lower roofs abut taller elements, guided by observation and measurement of the roof geometry. The observed deviations in the roof level also informed an allowance for ponding of water in a period of wet weather. The self-weight of the roof was calculated as $0.75kN/m^2$, which is a reasonable (if slightly high) value for a roof of this construction type. The slightly inflated value here may be a result of previous over-roofing works, but again, the load is not significantly higher than would generally be expected for an insulated timber roof. Peak snow drift has been calculated as $1.92kN/m^2$; this is a significant load, but calculations have been carried out to account for this as well as separate checks for lower, more regularly-expected loads. This allows considered comparison and pragmatic conclusions. Ponding loads have been applied equivalent to 20mm of water accumulating on the roof. Again, this is relatively conservative, and calculations are carried out both with and without this load.

Joists have been analysed based on the spans, dimensions and loadings from each of; both western classrooms, kitchen, main hall and south-east classroom. The main roof joists as well as the lower level of joists, supporting the ceilings and services, have been checked. The joists in different areas were slightly different cross sections and spans, each of which were checked independently against the observed in-situ loading; for example there was generally a lower level of joists purely for supporting the ceiling and services, however in the kitchen the suspended services were supported directly from the upper joists, resulting in a higher applied load. All such elements have been confirmed as structurally adequate, capable of supporting the loads even from a significant snow event.

As noted above, the compound I-sections have been referenced to manufacturer's literature and also calculated as equivalent, similar cross sections. Both of these checks raise concerns with the capacity of these members under even standard imposed loading. If imposed loads are limited to single-person access only then the members may be adequate in Ultimate Limit State (ULS; check against failure rather than deflection), however even this is based on the aforementioned simplification. Based on potential ponding or snow loading however, the members are at risk of failure. It should be noted that, as specialist elements, numerous assumptions have to be made in these checks (more so than with standard timber elements), but the concerns are considered to be valid even just considering the observed arrangement and spans.

The primary glulam member spanning over the kitchen, supporting all secondary joists and applied loading, is shown to be adequate even under significant snow drift loading. This member supports the joists and I-section beams for the whole extent of the roof and spans the full 6.5m width, however its size and condition results in a structurally adequate member.

Over the main hall, the single primary glulam member spans the full width, between timber columns, and supports all perpendicular I-Section joists. Under code-standard imposed loads or basic snow loading these members are at risk of failure; only if imposed loads are limited to single-person access can the member be shown to pass. For this reason, the member is not at imminent risk of collapse, but if snow were to accumulate, or ponding of rainwater to occur, then it may be susceptible to failure.

Lateral stability of the frame is evidently achieved through multiple orthogonal sheathed walls, however a basic check was carried out using only primary perimeter wall frames, considering the timber members and no restraint being taken from infill panels or sheathing, as is frequently observed to be the case for the perimeter walls here (for example on the largely-glazed elevations). Both the standard, shorter wall height, and the taller main hall wall panels, showed excessive deflection under calculation, indicating that they are inadequate in providing lateral stability for the building. As noted, the building is not considered susceptible to racking (in-plane failure) due to the frequent internal framed walls, however this adds further certainty to the earlier conclusion (from report SR-01) that perimeter walls are inadequate to resist wind loadings.

The assumptions made in the initial report have been checked and verified, and as a result the conclusions remain valid. The grade of timber is still not known, but sectional areas and structural arrangements were all confirmed as accurate where inspected.

Conclusions

The roof is generally in reasonable condition. Aside from occasional notches and openings cut to allow for passage of cables and services, the timber members appear either to have been very well protected or installed more recently than the original building construction.

The basic roof joists are adequate to support the applied loadings, and are not at risk of failure even under a significant snow event.

The primary glulam member over the main hall, as well as the compound I-sections over classrooms, are at risk of failure if ponding of rainwater or accumulation of snow were to occur. Under the roof's own self-weight, and a very small nominal applied imposed load, the members appear to be adequate.

Based on the lack of obvious and significant deflection from visual inspection, and the calculations showing sufficient capacity against nominal loading, the building is not considered to be at significant and imminent risk of structural failure or collapse. There are, however, several members which will be over-loaded and may be at risk of failure under snow or ponding loads. This is considered to be a significant risk in the medium-term, which requires attention and remedial action.

As noted earlier in this report, the previous report (JK-6289-SR-01) is still valid and relevant. The conclusions from both reports should be considered together, to give a full picture of the building's condition. The summary of that original report is that vertical posts should be strengthened or replaced in order to prevent failure of these members against wind loading. This has been further verified by the calculations carried out as part of this report.

Recommendations

Immediate evacuation of the building is not considered necessary, due to the roof structure's capacity to support its own self-weight alongside very basic imposed maintenance access. However, the susceptibility to failure of multiple members under snow loading or development of ponding suggests that remedial measures should be put in place in the short to medium term. If strengthening works were progressed, these would be very intrusive and expensive, essentially resulting in new roof and wall panel structures being installed across much of the building footprint. This does not consider the foundations, which may also be inadequate, having not been inspected in detail.

Access to the roof should be restricted to essential maintenance only, and this should be carried out in a safe and conscientious manner by a single individual only. As well as employing general safe working practices, such access should only be carried out when the room below is unoccupied. In periods of inclement weather, regular inspections should be made to ensure that snow or ponding water do not accumulate on the roof.

This report does not seek to give financial or budgeting advice, but the recommendation is that the most structurally-appropriate long-term solution will be to construct a new building. The priority of course is to ensure structural safety and stability; repair works could achieve this only to the extent of the members which have been inspected and assessed, resulting in significant construction works and also the remaining presence of uninspected elements which are also likely to be beyond their design life.

We will be more than happy to assist in the development of remedial works, or design for any new construction (including feasibility assessments and/or detailed design) if required.

We trust we have addressed your requirements correctly but if you have any comments or queries please do not hesitate to call.

Yours faithfully,

[redacted]

Simon Brent MEng CEng MIStructE– Senior Engineer, Jasper Kerr Consulting Engineers Ltd

Images

Figure 2 - Roof construction over the western classrooms, showing primary compound I-section joists with two layers of secondary joists.

Figure 3 - Roof surface, showing deviations capable of accumulating some ponding, as well as the higher level roof which may result in snow drift loading against it. The shallow roof pitch and negligible gutter gradient are also visible here.

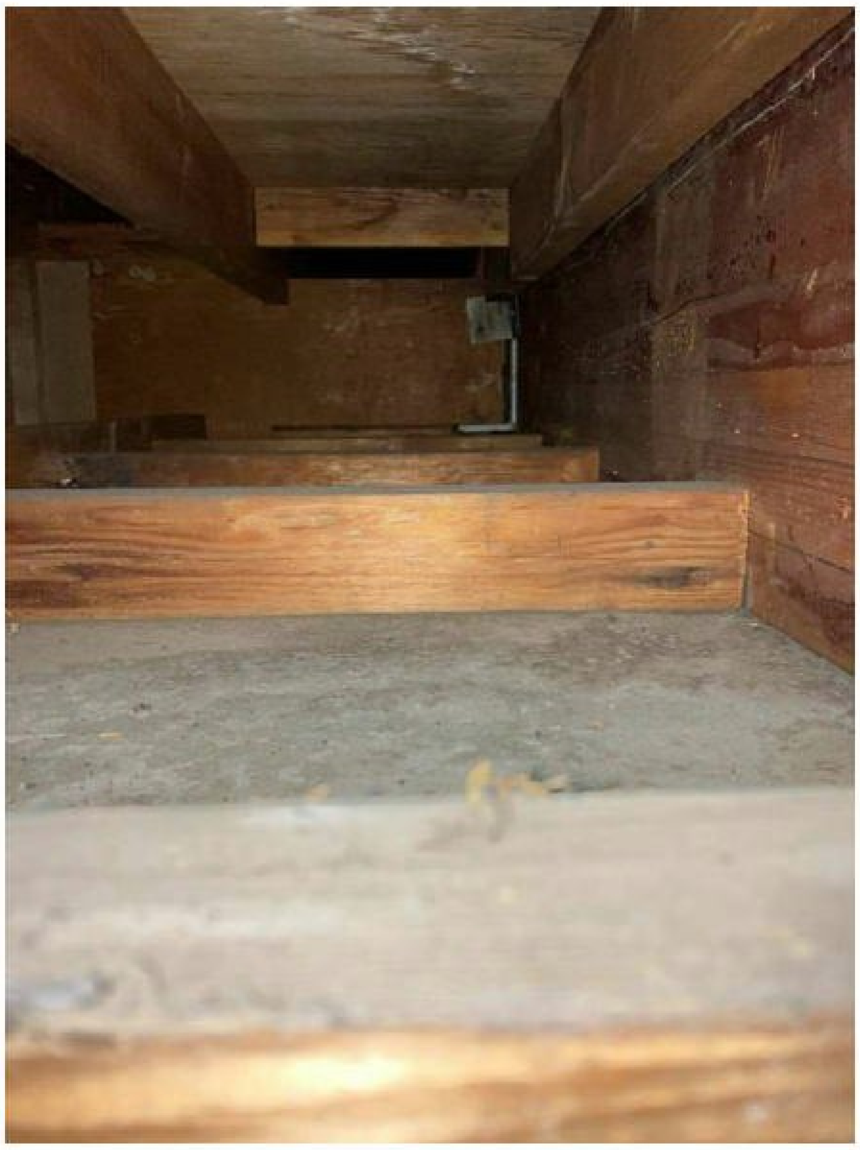


Figure 4 - Construction over the main hall, with primary Glulam members supporting two layers of timber joists, secondary composite I-sections are also visible here.

Figure 5 - Roof structure over the kitchen, with a combination of simple joists, timber I-sections and Glulam members.

JASPER KERR
CONSULTING

APPENDIX A

Structural Calculations

Structural calculations, carried out to Eurocode standard, as referenced within the Structural Report.

Jasper Kerr Consulting Engineers
info@jasperkerr.co.uk

Project				Status	
WINCHAM FIRST SCHOOL.					
Date 10/12/20 By SRB. Checked		Job No. JKC-6289.	Section	Sheet No. 1	Rev 1
Rev	Date	Details POST. INTRUSIVE SURVEY			
Part OUTLINE CALCULATIONS.					

JASPER KERR

Tel

Fax

LOADING.

ROOF WAS INSPECTED FROM ABOVE, AND MEASUREMENTS TAKEN TO DETERMINE APPROX BUILD-UP. ALTHOUGH NO CORE WAS TAKEN (IN THE INTEREST OF PRESERVING WATER-TIGHTNESS), THIS GIVES SOME FURTHER DETAIL OF ROOF SELF-WEIGHT AND GENERAL CONDITION.

MEMBRANE = 0.15 kN/m²
INSULATION = 0.1 kN/m²
COUNTER-BATTENS = 0.1 kN/m²
FIRRING STRIPS = 0.05 kN/m²
JOISTS = 0.2 kN/m²
CEILING + SERVICES = 0.15 kN/m²

$g_k = 0.75$ kN/m²

NOTE: THIS IS STILL AN ESTIMATE, BUT MORE INFORMED THAN PREVIOUSLY.

UP TO 20mm PONDING = 0.2 kN/m²
BASIC ACCESS = 0.6 kN/m²

$q_k = 0.8$ kN/m²

CONSIDER SNOW DRIFT;

NORTH EAST → $s_k = 0.6$ kN/m²

$b_1 = 8m$ (CLASSROOM)
$b_2 = 12m$ (MAIN HALL).
$h \sim 1.5m$

l_s = LEAST OF: $5h = 7.5m$
$b_1 = 8m$
15m

→ $l_s = 7.5m$.

μ_3 = LEAST OF: $2h/s_k = 5$
$2b/l_s = 3.2$
8

→ $\mu_3 = 3.2$

PEAK SNOW DRIFT = 0.6 × 3.2
= 1.92 kN/m².

Project				Status	
OVINGHAM FIRST SCHOOL.					
Date 10/12/20 By SRB. Checked	Job No. JK-6289.	Section	Sheet No. 2	Rev 1	

JASPER KERR CONSULTING

Rev	Date	Details	
		POST-INTRUSIVE SURVEY	Tel Fax

Part: OUTLINE CALCULATIONS.

ANALYSIS.

ROOF JOISTS - WEST CLASSROOMS.

SPAN ~ 2050MM.

SEPARATE JOISTS SUPPORT CEILING → g_k: 0.4 kN/m²

SPACED @ 400MM → g_k = 0.4 × 0.4
= 0.16 kN/m

q_k = 0.8 × 0.4
= 0.32 kN/m

UNDER SNOW DRIFT → q_k = 1.92 × 0.4
= 0.77 kN/m MAX.
= (1.92 × 3/4) × 0.4
= 0.58 kN/m MIN.

JOISTS ARE APPROX. 120 × 38MM.

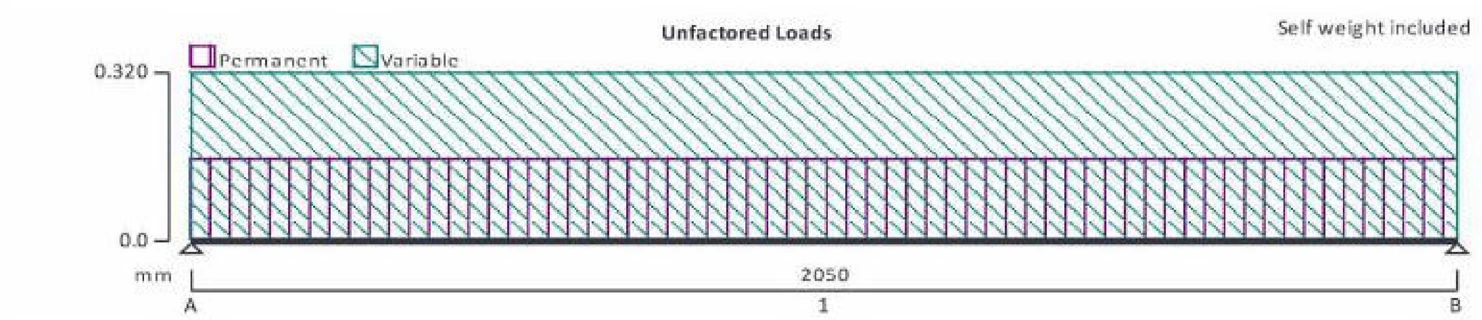
JOISTS ARE OK EVEN IN SIGNIFICANT SNOW EVENT.

BASIC CHECK - PAGES 3-4.
EXTREME SNOW CHECK PAGES 5-6.

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

Permanent self weight of beam × 1

Permanent full UDL 0.160 kN/m

Variable full UDL 0.320 kN/m

Analysis results

Design moment M = **0.376** kNm Design shear F = **0.733** kN

Total load on member W_{tot} = **1.466** kN

Reactions at support A R_{A_max} = **0.733** kN R_{A_min} = **0.733** kN

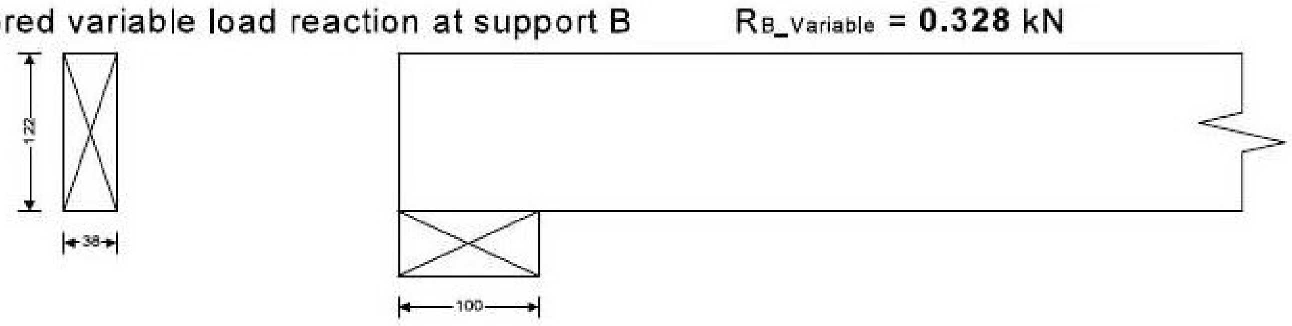
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **0.178** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.328** kN

Reactions at support B R_{B_max} = **0.733** kN R_{B_min} = **0.733** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **0.178** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.328** kN



Timber section details

Breadth of section b = **38** mm Depth of section h = **122** mm

Number of sections N = **1** Breadth of member b_b = **38** mm

Timber strength class **C16**

Member details

Service class of timber **1** Load duration **Short-term**

Length of span L_{s1} = **2050** mm

Length of bearing L_b = **100** mm

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.193** N/mm^2 Design compressive strength $f_{c.90.d}$ = **1.675** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **3.985** N/mm^2 Design bending strength $f_{m.d}$ = **12.699** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.354** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.437** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

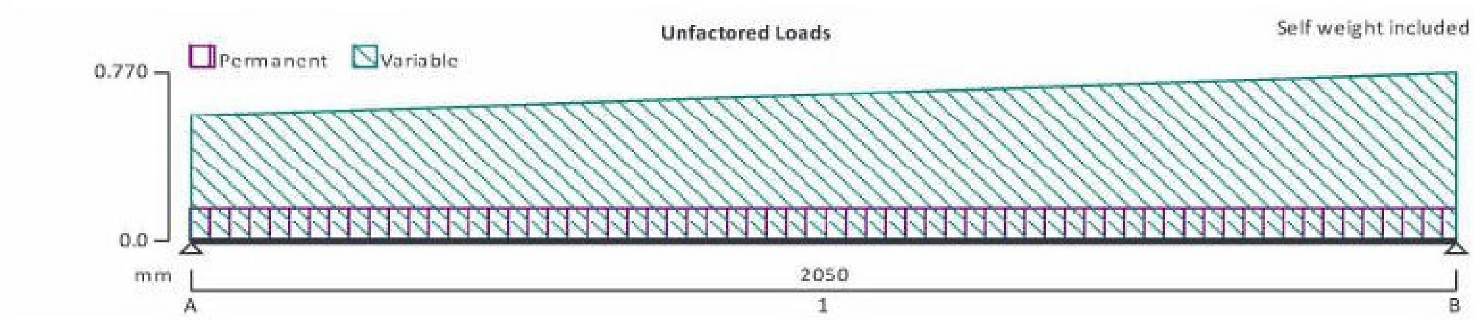
Deflection limit δ_{lim} = **8.200** mm Total final deflection δ_{fin} = **3.459** mm

PASS - Total final deflection is less than 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

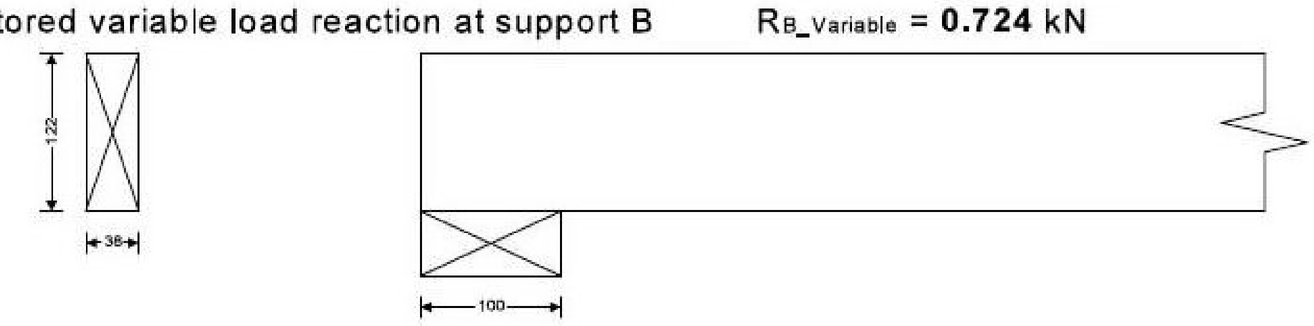
Permanent self weight of beam × 1

Permanent full UDL 0.160 kN/m

Variable full VDL 0.580 kN/m to 0.770 kN/m

Analysis results

Design moment	M = **0.656** kNm	Design shear	F = **1.327** kN
Total load on member	W_{tot} = **2.557** kN		
Reactions at support A	R_{A_max} = **1.230** kN	R_{A_min} = **1.230** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.178** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **0.659** kN		
Reactions at support B	R_{B_max} = **1.327** kN	R_{B_min} = **1.327** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.178** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **0.724** kN		



Timber section details

Breadth of section	b = **38** mm	Depth of section	h = **122** mm
Number of sections	N = **1**	Breadth of member	b_b = **38** mm
Timber strength class	**C16**		

Member details

Service class of timber	**1**	Load duration	**Short-term**
Length of span	L_{s1} = **2050** mm		
Length of bearing	L_b = **100** mm		

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.349** N/mm^2 Design compressive strength $f_{c.90.d}$ = **1.675** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **6.955** N/mm^2 Design bending strength $f_{m.d}$ = **12.699** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.641** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.437** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **8.200** mm Total final deflection δ_{fin} = **5.667** mm

PASS - Total final deflection is less than the deflection limit

Project			Status	
OVINGHAM FIRST SCHOOL.				
Date 10/12/20 By SRB. Checked	Job No. JK.6289.	Section	Sheet No. 7	Rev 1

JASPER KERR

Rev	Date	Details
		POST-INTRUSIVE SURVEY.

Tel

Fax

Part

OUTLINE CALCULATIONS.

PRIMARY TIMBERS - WEST CLASSROOMS.

COMPOUND I-SECTION. APPROX. 250MM DEEP.
SPAN ~ 7000MM

FROM MANUFACTURER'S LITERATURE, SPANS ARE ONLY ACHIEVABLE @ ≤400MM c/c.

SEE PAGE 8.

CARRY OUT EQUIVALENT CHECK AS GLULAM.

~250MM DP.
~75MM WD. → SAY 90MM TO ALLOW FOR FLANGES.

SPACING = 2.05M → $g_k = 0.75 \times 2.05 = 1.54\,kN/m$
$q_k = 0.8 \times 2.05 = 1.64\,kN/m$

IN EXTREME SNOW EVENT, $q_k = (\frac{3}{4} \times 1.92) \times 2.05 = 2.95\,kN/m$

BEAM IS SUSCEPTIBLE TO FAILURE UNDER STANDARD IMPOSED LOAD. AT SIGNIFICANT RISK UNDER SIGNIFICANT SNOW EVENT.

SATISFACTORY (TO ULS) IF IMPOSED ACTIONS LIMITED TO SINGLE-PERSON ACCESS (NO PONDING OR SNOW).

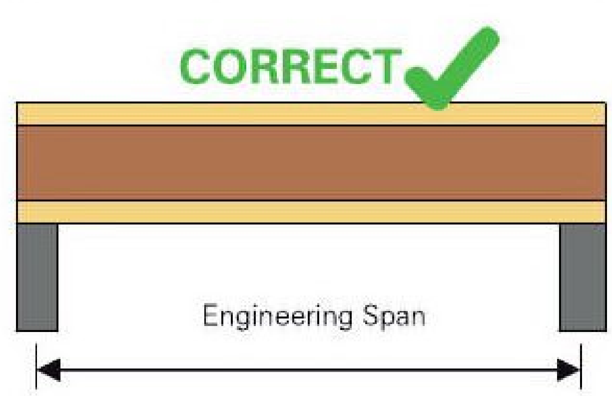
BASIC CHECK - PAGES 8-9.
MIN. LOAD CHECK - PAGES 10-11.

JJI-JOIST FLAT ROOFS

- Dead Load should take account of all components of the roof makeup including ceiling linings
- Imposed Load 0.75kN/m² (Snow)
- Deflection limit 0.003 x span
- Joist design includes a 0.9kN Man Load check at mid span
- The calculation of the spans includes a load sharing factor
- Adequate lateral restraint to the top flange of the joists is assumed to be provided by the roof deck
- The joists are designed using the principles of BS 5268-2
- Where the load conditions are different to those described above, refer to the JJI-Joist supplier for further assistance

Joist Type	Dead load up to 0.75kN/m²				Dead load up to 1.00kN/m²			
	Joist Centres (mm)				Joist Centres (mm)			
	300	400	480	600	300	400	480	600
JJI 145 A+	3242	3071	2956	2808	3071	2887	2765	2611
JJI 195 A+	4681	4407	4216	3874	4407	4121	3936	3650
JJI 195 B+	5339	5006	4673	4288	5007	4664	4404	4036
JJI 195 C	5718	5286	4931	4521	5351	4974	4645	4253
JJI 195 D	6627	5948	5542	5072	6174	5603	5214	4764
JJI 220 A+	5320	4992	4667	4291	4998	4664	4405	4046
JJI 220 B+	6149	5606	5235	4808	5752	5291	4937	4528
JJI 220 C	6456	5831	5443	4997	6031	5502	5132	4704
JJI 220 D	7230	6491	6053	5548	6826	6119	5701	5217
JJI 235 A+	5675	5257	4916	4522	5325	4965	4641	4265
JJI 235 B+	6537	5893	5505	5058	6110	5564	5193	4766
JJI 235 C	6823	6136	5730	5263	6412	5792	5404	4957
JJI 235 D	7607	6833	6375	5847	7184	6444	6006	5501
JJI 245 A+	5905	5428	5077	4672	5537	5130	4794	4407
JJI 245 B+	6743	6069	5671	5213	6330	5731	5351	4913
JJI 245 C	7041	6334	5917	5436	6655	5980	5581	5121
JJI 245 D	7861	7063	6591	6047	7425	6662	6212	5691
JJI 300 A+	7042	6351	5944	5475	6664	6005	5617	5169
JJI 300 B+	7799	7027	6573	6049	7377	6641	6207	5707
JJI 300 C	8214	7398	6917	6363	7768	6990	6530	6001
JJI 300 D	9225	8299	7752	7122	8719	7835	7313	6710
*JJI 350 A+	7832	7068	6618	6101	7414	6686	6257	5763
*JJI 350 B+	8592	7749	7252	6680	8131	7327	6853	6307
JJI 350 C	9302	8384	7842	7219	8800	7924	7407	6811
JJI 350 D	10345	9314	8706	8005	9781	8798	8217	7548
*JJI 400 A+	8707	7861	7363	6790	8244	7438	6963	6416
*JJI 400 B+	9546	8613	8063	7431	9036	8146	7622	7018
JJI 400 C	10443	9416	8810	8113	9881	8901	8324	7658
JJI 400 D	11310	10191	9530	8770	10698	9630	9000	8273
*JJI 450 A+	9550	8625	8080	7454	9044	8162	7643	7046
*JJI 450 B+	10463	9444	8843	8153	9906	8934	8361	7702
*JJI 450 C	10910	9844	9216	8494	10327	9311	8712	8022
JJI 450 D	11910L	10917	10214	9405	11457	10320	9650	8877

Table 18. Maximum Engineering Span For JJI-Joist Flat Roofs



Notes for Table 18:

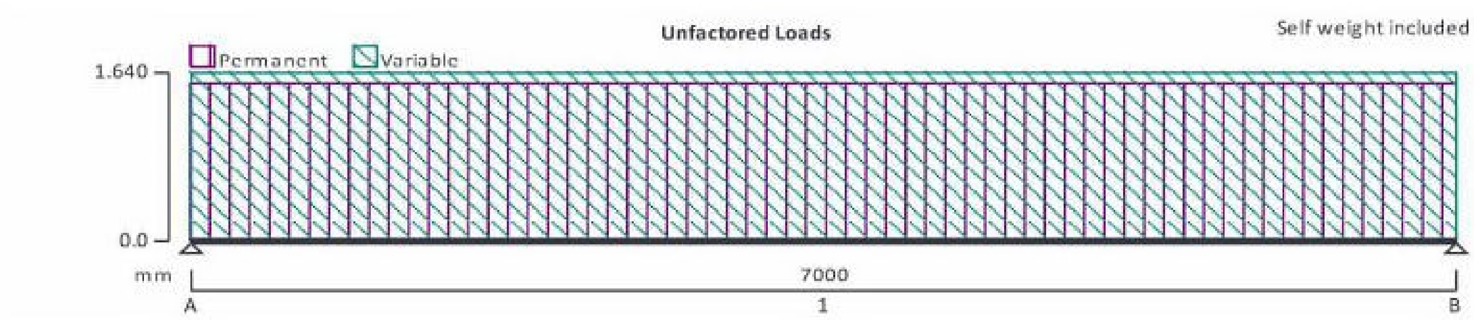
1. These tables serve as guidance only. For a more detailed JJI-Joist appraisal contact a JJI-Joist Distributor
2. To achieve the stated spans, adequate JJI-Joist bearing will be required. Web stiffeners may be necessary
3. Designs based on a dead load as shown + 0.75kN/m² imposed snow load
4. Permissible web holes to be drilled in accordance with JoistMaster software
5. Design in accordance with BS 6399-3–4.3.1 (Minimum imposed load on roof with no access)
6. No allowance for rafter overhangs within tables
7. Figures followed by L denote engineering spans limited by the maximum manufactured JJI-Joist length of 12m
8. *The top and bottom flanges of these sizes require continuous lateral restraint at a maximum of 300mm centres unless a lateral buckling check has been performed
9. No wind allowance has been considered in this table

Tekla Tedds Jasper Kerr Consulting Unit 7, Riverside Studios Amethyst Road, Newcastle Business Park Newcastle upon Tyne, NE4 7YL	Project Ovingham First School				Job no. JK-6289	
	Calcs for Primary Roof Joists				Start page no./Revision 8 1	
	Calcs by SRB	Calcs date 10/12/2020	Checked by	Checked date	Approved by	Approved date

GLULAM 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

Permanent self weight of beam × 1
Permanent full UDL 1.540 kN/m
Variable full UDL 1.640 kN/m

Analysis results

Design moment M = **28.639** kNm — Design shear F = **16.365** kN

Total load on member W_{tot} = **32.730** kN

Reactions at support A R_{A_max} = **16.365** kN — R_{A_min} = **16.365** kN

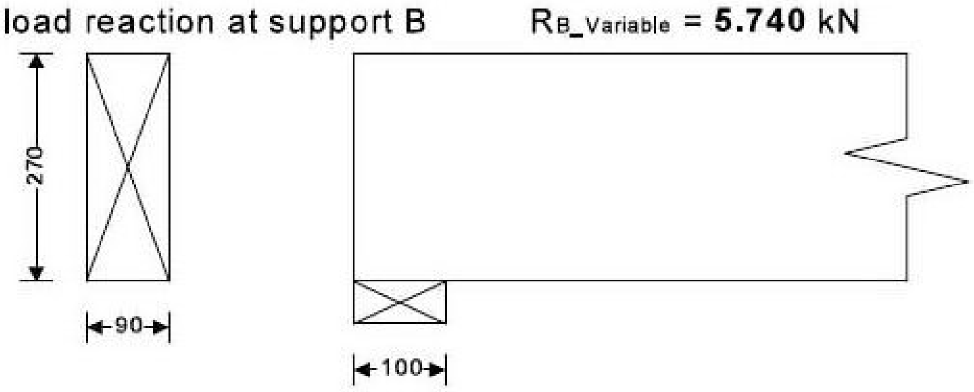
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **5.744** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **5.740** kN

Reactions at support B R_{B_max} = **16.365** kN — R_{B_min} = **16.365** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **5.744** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **5.740** kN



Glulam section details

Breadth of section b = **90** mm — Depth of section h = **270** mm

Number of sections N = **1** — Breadth of member b_b = **90** mm

Glulam strength class **GL28h**

Member details

Service class of timber **1** — Load duration **Short-term**

Length of span L_{s1} = **7000** mm

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **1.818** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.800** N/mm^2

FAIL - Design compressive stress exceeds design compressive strength at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **26.190** N/mm^2 Design bending strength $f_{m.d}$ = **21.836** N/mm^2

FAIL - Design bending stress exceeds design bending strength

Shear - cl.6.1.7

Applied shear stress τ_d = **1.508** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

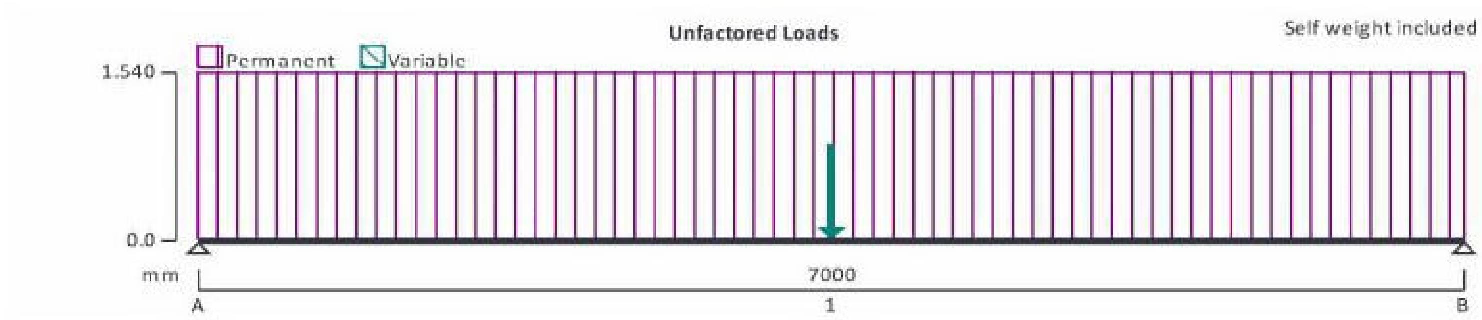
Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **78.786** mm

FAIL - Total final deflection exceeds the deflection limit

GLULAM 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

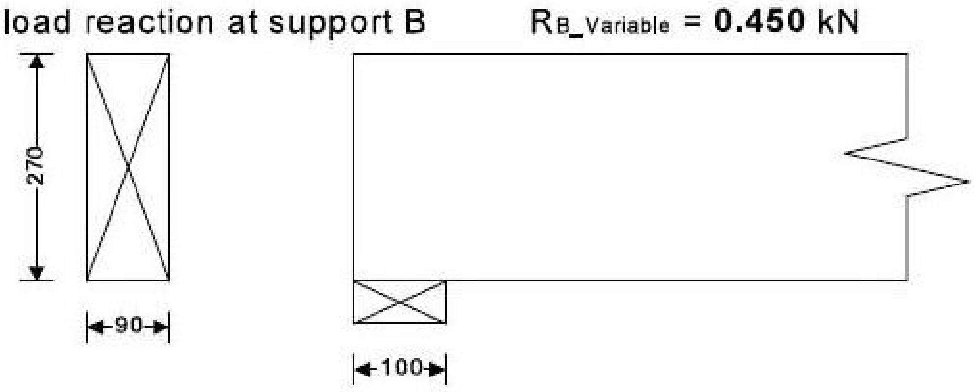
Permanent self weight of beam × 1

Permanent full UDL 1.540 kN/m

Variable point load 0.900 kN at 3500 mm

Analysis results

Design moment	M = **15.934** kNm	Design shear	F = **8.430** kN
Total load on member	W_{tot} = **16.860** kN		
Reactions at support A	R_{A_max} = **8.430** kN	R_{A_min} = **8.430** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **5.744** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **0.450** kN		
Reactions at support B	R_{B_max} = **8.430** kN	R_{B_min} = **8.430** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **5.744** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **0.450** kN		



Glulam section details

Breadth of section	b = **90** mm	Depth of section	h = **270** mm
Number of sections	N = **1**	Breadth of member	b_b = **90** mm
Glulam strength class	**GL28h**		

Member details

Service class of timber	**1**	Load duration	**Short-term**
Length of span	L_{s1} = **7000** mm		
Length of bearing	L_b = **100** mm		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c.90.d}$ = **0.937** N/mm^2	Design compressive strength	$f_{c.90.d}$ = **1.800** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **14.571** N/mm^2 Design bending strength $f_{m.d}$ = **21.836** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.777** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **49.581** mm

FAIL - Total final deflection exceeds the deflection limit

Project			Status	
WINGHAM FIRST SCHOOL.				
Date 10/12/20 By SRB. Checked	Job No. JK-6289.	Section	Sheet No. 12	Rev 1

JASPER KERR CONSULTING

Rev	Date	Details	Tel
		POST-INTRUSIVE SURVEY	Fax

Part
OUTLINE CALCULATIONS.

ROOF JOISTS - KITCHEN

SPAN ~ 2050MM, 120 x 38MM SECTIONS AS CLASSROOMS
BUT SPACED @ 600MM c/c AND NO SECONDARY
JOISTS SUPPORTING SUSP. CEILING.

$\rightarrow g_k = 0.55 \times 0.6$
$= 0.33 kN/m$
$q_k = 0.8 \times 0.6$
$= 0.48 kN/m$

JOISTS ARE OK EVEN IN
SIGNIFICANT SNOW EVENT.

BASIC CHECK - PAGES 13 - 14
SNOW DRIFT - PAGES 15 - 16.

PRIMARY GLULAM - KITCHEN.

SINGLE GLULAM MEMBER SPANS 6.5M WIDTH OF KITCHEN.

SECTION ~ 400 x 175 M.

SUPPORTS 1/2 OF KITCHEN'S 6M WIDTH.

$\rightarrow g_k = 0.75 \times (6/2)$
$= 2.25 kN/m$
$q_k = 0.8 \times (6/2)$
$= 2.4 kN/m$

SNOW DRIFT $\rightarrow q_k = (1.92/2) \times (6/2)$
$= 2.88 kN/m$

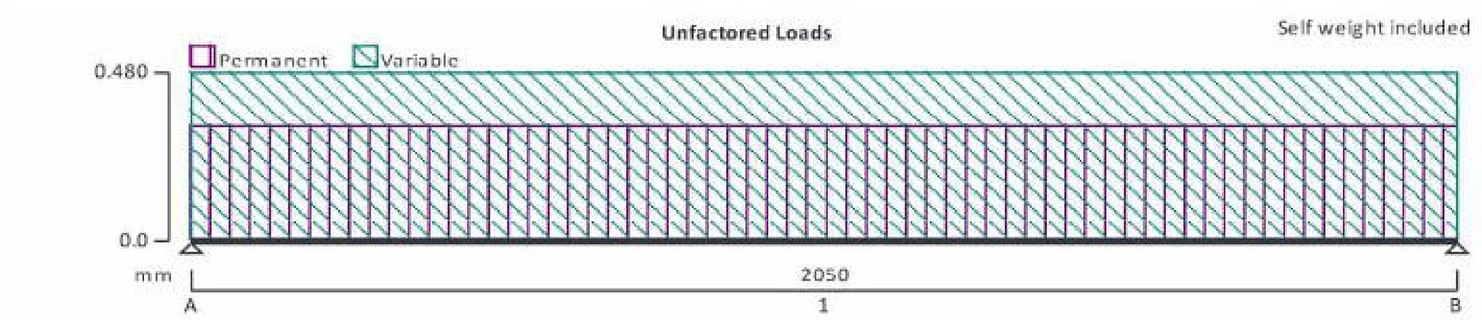
BEAM IS OK EVEN IN
SIGNIFICANT SNOW EVENT.

BASIC CHECK PAGES 17 - 18
SNOW DRIFT - PAGES 19 - 20.

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



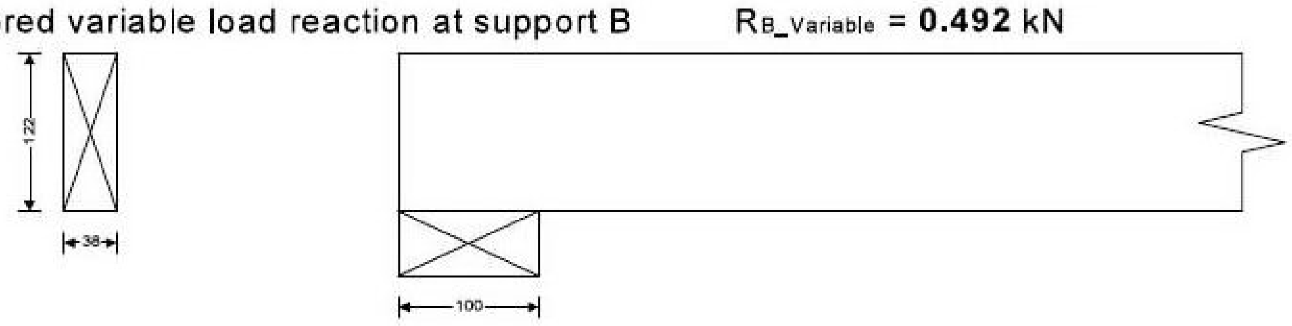
Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 0.330 kN/m
Variable full UDL 0.480 kN/m

Analysis results

Design moment M = **0.622** kNm — Design shear F = **1.214** kN
Total load on member W_{tot} = **2.428** kN
Reactions at support A R_{A_max} = **1.214** kN — R_{A_min} = **1.214** kN
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **0.353** kN
Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.492** kN
Reactions at support B R_{B_max} = **1.214** kN — R_{B_min} = **1.214** kN
Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **0.353** kN
Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.492** kN



Timber section details

Breadth of section b = **38** mm — Depth of section h = **122** mm
Number of sections N = **1** — Breadth of member b_b = **38** mm
Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Short-term**
Length of span L_{s1} = **2050** mm
Length of bearing L_b = **100** mm

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.320** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.675** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **6.601** N/mm^2 Design bending strength $f_{m.d}$ = **12.699** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.586** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.437** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **8.200** mm Total final deflection δ_{fin} = **5.887** mm

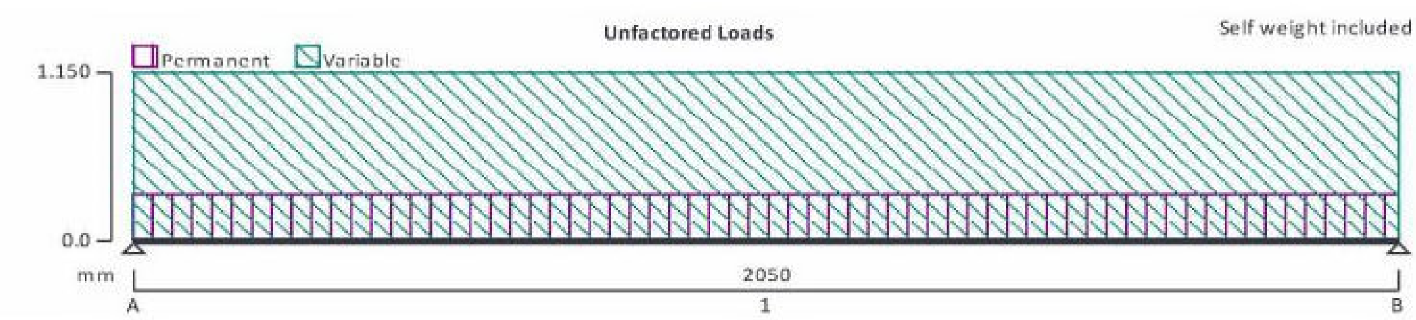
PASS - Total final deflection is less than the deflection limit

Tekla Tedds Jasper Kerr Consulting Unit 7, Riverside Studios Amethyst Road, Newcastle Business Park Newcastle upon Tyne, NE4 7YL	Project Ovingham First School			Job no. JK-6289	
	Calcs for Roof Joists - Kitchen, Snow Drift			Start page no./Revision 15 1	
	Calcs by SRB	Calcs date 10/12/2020	Checked by	Checked date	Approved by	Approved date

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



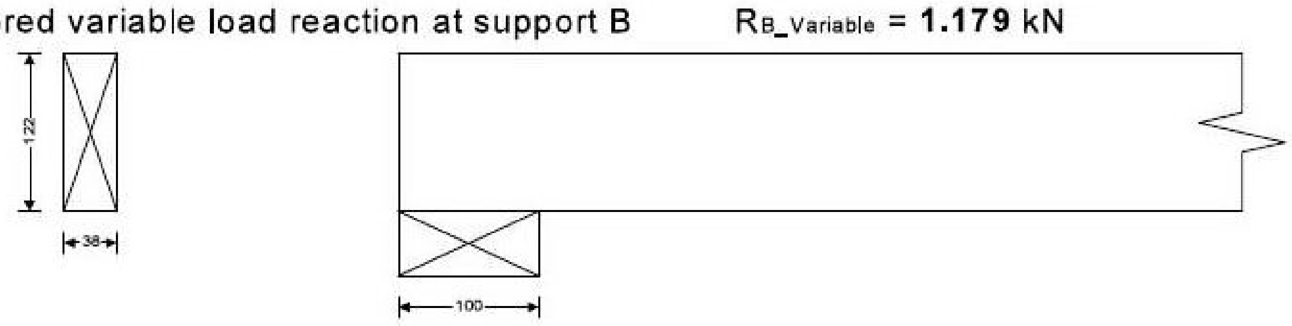
Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 0.330 kN/m
Variable full UDL 1.150 kN/m

Analysis results

Design moment M = **1.150** kNm — Design shear F = **2.244** kN
Total load on member W_{tot} = **4.489** kN
Reactions at support A R_{A_max} = **2.244** kN — R_{A_min} = **2.244** kN
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **0.353** kN
Unfactored variable load reaction at support A $R_{A\_Variable}$ = **1.179** kN
Reactions at support B R_{B_max} = **2.244** kN — R_{B_min} = **2.244** kN
Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **0.353** kN
Unfactored variable load reaction at support B $R_{B\_Variable}$ = **1.179** kN



Timber section details

Breadth of section b = **38** mm — Depth of section h = **122** mm
Number of sections N = **1** — Breadth of member b_b = **38** mm
Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Short-term**
Length of span L_{s1} = **2050** mm
Length of bearing L_b = **100** mm

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.591** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.675** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **12.202** N/mm^2 Design bending strength $f_{m.d}$ = **12.699** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **1.084** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.437** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

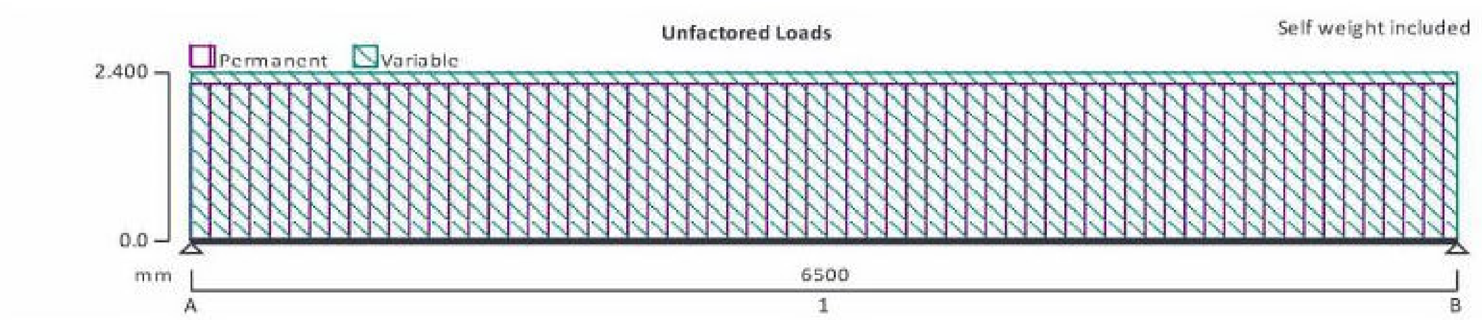
Deflection limit δ_{lim} = **8.200** mm Total final deflection δ_{fin} = **10.055** mm

FAIL - Total final deflection exceeds the deflection limit

GLULAM 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



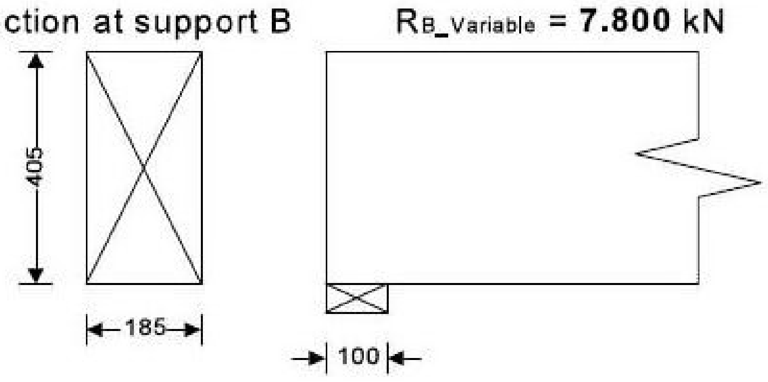
Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 2.250 kN/m
Variable full UDL 2.400 kN/m

Analysis results

Design moment	M = **37.281** kNm	Design shear	F = **22.942** kN
Total load on member	W_{tot} = **45.884** kN		
Reactions at support A	R_{A_max} = **22.942** kN	R_{A_min} = **22.942** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **8.327** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **7.800** kN		
Reactions at support B	R_{B_max} = **22.942** kN	R_{B_min} = **22.942** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **8.327** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.800** kN		



Glulam section details

Breadth of section	b = **185** mm	Depth of section	h = **405** mm
Number of sections	N = **1**	Breadth of member	b_b = **185** mm
Glulam strength class	**GL28h**		

Member details

Service class of timber	**1**	Load duration	**Short-term**
Length of span	L_{s1} = **6500** mm		
Length of bearing	L_b = **100** mm		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c,90,d}$ = **1.240** N/mm^2	Design compressive strength	$f_{c,90,d}$ = **1.800** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **7.371** N/mm^2 Design bending strength $f_{m.d}$ = **20.968** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.686** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **13.387** mm

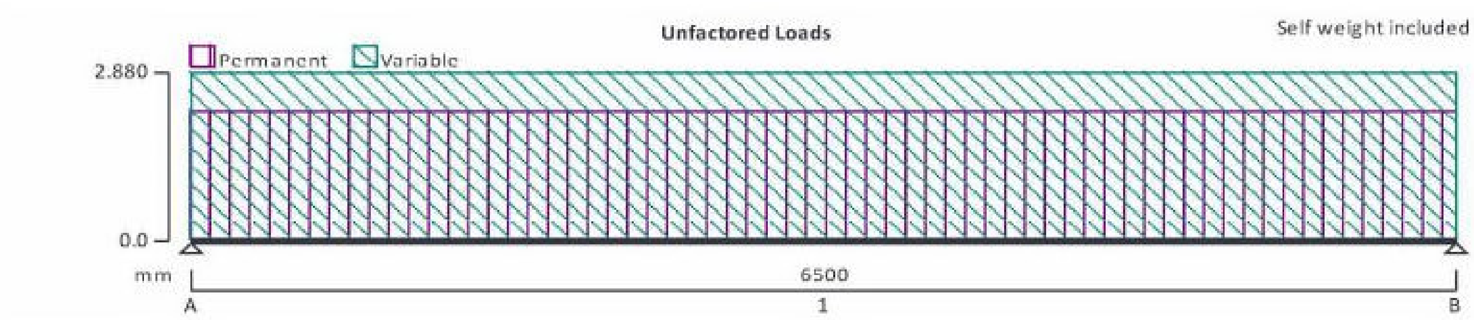
PASS - Total final deflection is less than the deflection limit

Tekla Tedds Jasper Kerr Consulting Unit 7, Riverside Studios Amethyst Road, Newcastle Business Park Newcastle upon Tyne, NE4 7YL	Project Ovingham First School				Job no. JK-6289	
	Calcs for Primary Glulam - Kitchen, Snow Drift				Start page no./Revision 19 1	
	Calcs by SRB	Calcs date 10/12/2020	Checked by	Checked date	Approved by	Approved date

GLULAM 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

Permanent self weight of beam × 1
Permanent full UDL 2.250 kN/m
Variable full UDL 2.880 kN/m

Analysis results

Design moment M = **41.083** kNm — Design shear F = **25.282** kN

Total load on member W_{tot} = **50.564** kN

Reactions at support A R_{A_max} = **25.282** kN — R_{A_min} = **25.282** kN

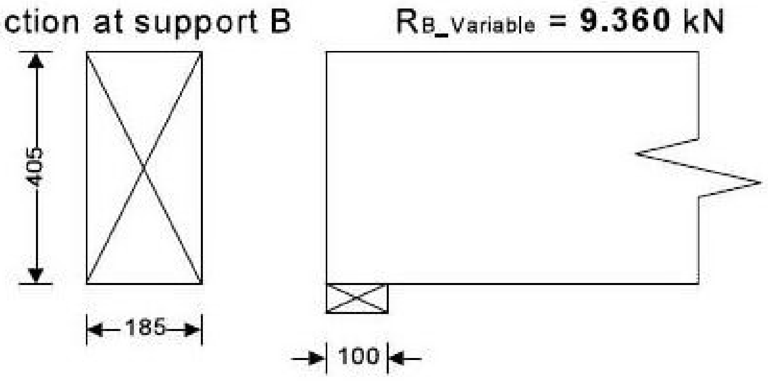
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **8.327** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **9.360** kN

Reactions at support B R_{B_max} = **25.282** kN — R_{B_min} = **25.282** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **8.327** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **9.360** kN



Glulam section details

Breadth of section b = **185** mm — Depth of section h = **405** mm

Number of sections N = **1** — Breadth of member b_b = **185** mm

Glulam strength class **GL28h**

Member details

Service class of timber **1** — Load duration **Short-term**

Length of span L_{s1} = **6500** mm

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d}$ = **1.367** N/mm^2 — Design compressive strength $f_{c,90,d}$ = **1.800** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **8.123** N/mm^2 Design bending strength $f_{m.d}$ = **20.968** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.755** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **14.481** mm

FAIL - Total final deflection exceeds the deflection limit

Project				Status	
WINGHAM FIRST SCHOOL.					
Date 10/12/20 By SRB Checked		Job No. JK-6289.	Section	Sheet No. 21	Rev 1

JASPER KERR

Rev	Date	Details	Tel
		POST-INTRUSIVE SURVEY.	Fax

Part: OUTLINE CALCULATIONS.

CEILING JOISTS - GENERAL.

IN WEST CLASSROOMS, 100 x 38mm @ 500mm c/c
→ $g_k \sim 0.15 \times 0.5$
$= 0.08 kN/m$

SPAN ~ 2050mm.

JOISTS ARE OK

SEE PAGES 22-23.

IN SOUTH EAST CLASSROOM, 65 x 50mm @ 400mm c/c.
→ $g_k \sim 0.15 \times 0.4$
$= 0.06 kN/m$

MAX. SPAN ~ 2900mm.

JOISTS ARE OK

SEE PAGES 24-25.

MAIN HALL PRIMARY GLULAM

SPAN ~ 10m.
SECTION ~ 375 x 175mm GLULAM.

SUPPORTS 1/3 of 12m MAIN HALL
→ $g_k = 0.75 \times (12/3)$
$= 3.0 kN/m$
$q_k = 0.8 \times (12/3)$
$= 3.2 kN/m$

& TBC WHETHER OTHER GLULAM EXISTS.

NO OTHER MEMBER (11/12/20)
CONCLUSIONS UNCHANGED.
SEE 28-29 REV. 2.

BEAM IS SUSCEPTIBLE TO FAILURE
UNDER STANDARD IMPOSED LOAD.

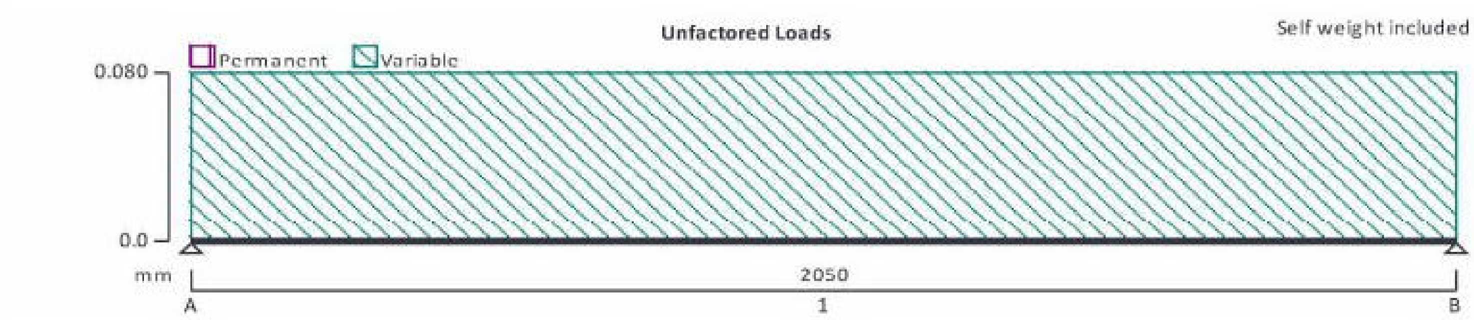
SATISFACTORY (TO US) IF IMPOSED
ACTIONS LIMITED TO SINGLE-PERSON ACCESS.

BASIC CHECK - PAGES 26-27.
MIN. LOAD CHECK - PAGES 28-29.

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

Permanent self weight of beam × 1

Variable full UDL 0.080 kN/m

Analysis results

Design moment M = **0.071** kNm — Design shear F = **0.139** kN

Total load on member W_{tot} = **0.277** kN

Reactions at support A R_{A_max} = **0.139** kN — R_{A_min} = **0.139** kN

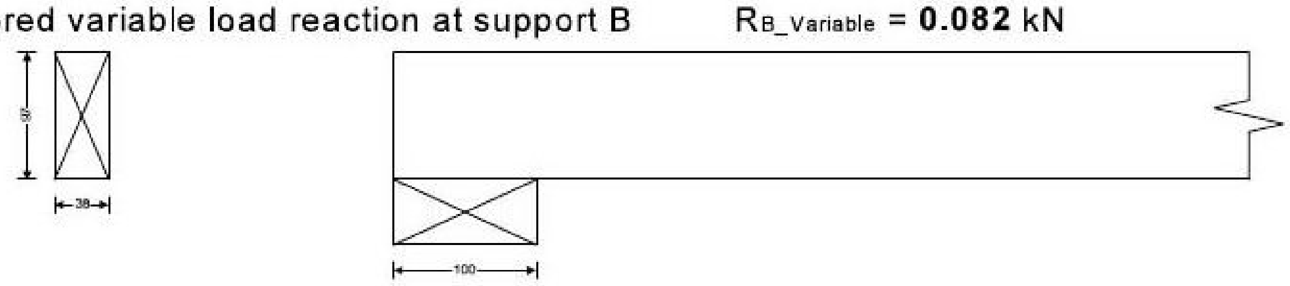
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **0.011** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.082** kN

Reactions at support B R_{B_max} = **0.139** kN — R_{B_min} = **0.139** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **0.011** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.082** kN



Timber section details

Breadth of section b = **38** mm — Depth of section h = **97** mm

Number of sections N = **1** — Breadth of member b_b = **38** mm

Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Long-term**

Length of span L_{s1} = **2050** mm

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d}$ = **0.036** N/mm^2 — Design compressive strength $f_{c,90,d}$ = **1.185** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m,d}$ = **1.191** N/mm^2 — Design bending strength $f_{m,d}$ = **9.400** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.084** N/mm^2 Permissible shear stress $f_{v.d}$ = **1.723** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

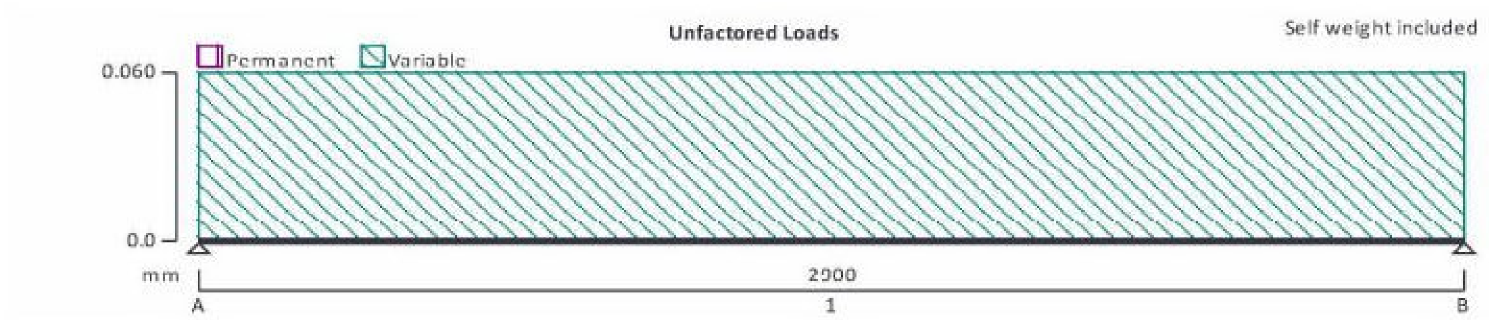
Deflection limit δ_{lim} = **8.200** mm Total final deflection δ_{fin} = **1.156** mm

PASS - Total final deflection is less than 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

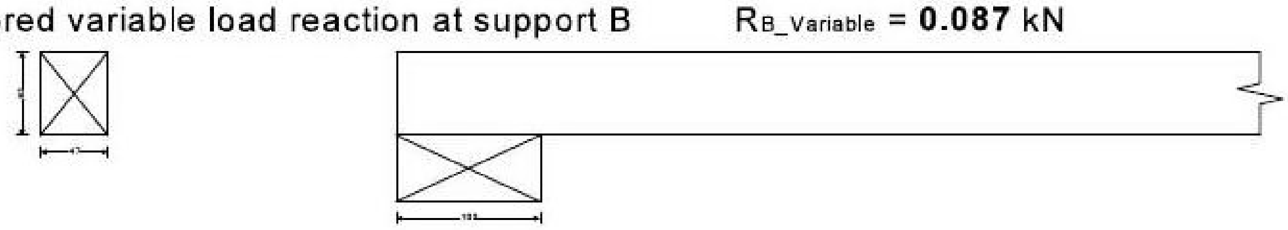
Beam loads

Permanent self weight of beam × 1

Variable full UDL 0.060 kN/m

Analysis results

Design moment	M = **0.107** kNm	Design shear	F = **0.148** kN
Total load on member	W_{tot} = **0.296** kN		
Reactions at support A	R_{A_max} = **0.148** kN	R_{A_min} = **0.148** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.013** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **0.087** kN		
Reactions at support B	R_{B_max} = **0.148** kN	R_{B_min} = **0.148** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.013** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **0.087** kN		



Timber section details

Breadth of section	b = **47** mm	Depth of section	h = **63** mm
Number of sections	N = **1**	Breadth of member	b_b = **47** mm
Timber strength class	**C16**		

Member details

Service class of timber	**1**	Load duration	**Long-term**
Length of span	L_{s1} = **2900** mm		
Length of bearing	L_b = **100** mm		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.032** N/mm^2 Design compressive strength $f_{c.90.d}$ = **1.185** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **3.454** N/mm^2 Design bending strength $f_{m.d}$ = **10.248** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.112** N/mm^2 Permissible shear stress $f_{v.d}$ = **1.723** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

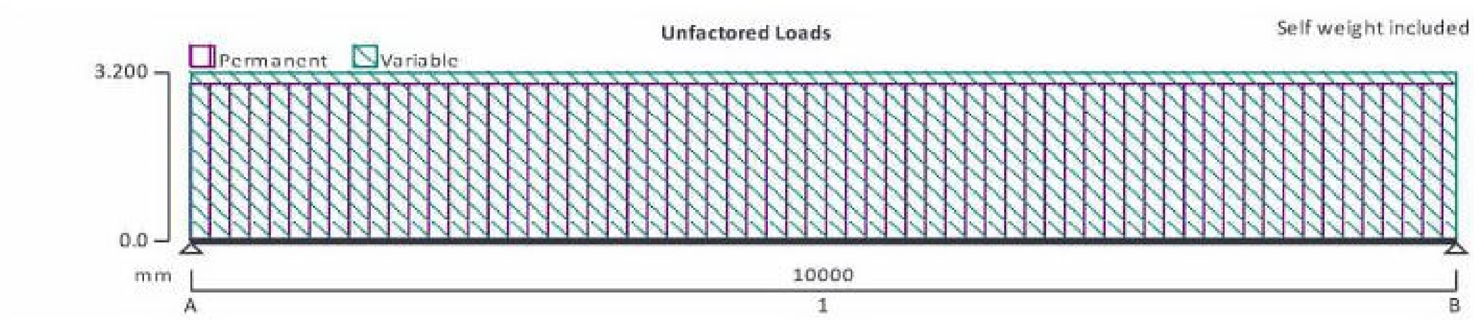
Deflection limit δ_{lim} = **11.600** mm Total final deflection δ_{fin} = **10.088** mm

PASS - Total final deflection is less than the deflection limit

GLULAM 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



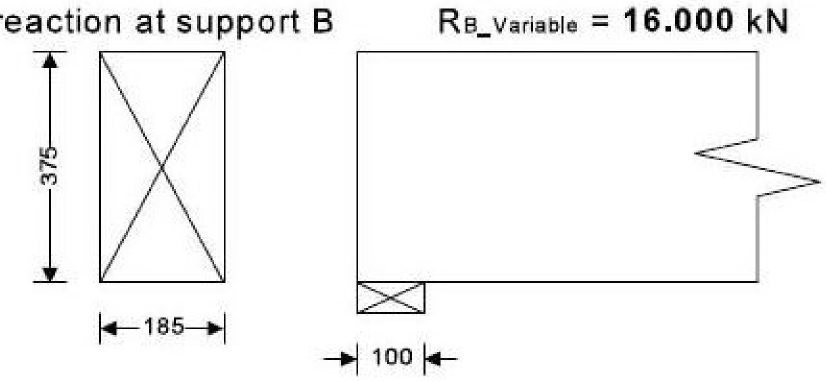
Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 3.000 kN/m
Variable full UDL 3.200 kN/m

Analysis results

Design moment	M = **115.676** kNm	Design shear	F = **46.271** kN
Total load on member	W_{tot} = **92.541** kN		
Reactions at support A	R_{A_max} = **46.271** kN	R_{A_min} = **46.271** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **16.497** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **16.000** kN		
Reactions at support B	R_{B_max} = **46.271** kN	R_{B_min} = **46.271** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **16.497** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **16.000** kN		



Glulam section details

Breadth of section	b = **185** mm	Depth of section	h = **375** mm
Number of sections	N = **1**	Breadth of member	b_b = **185** mm
Glulam strength class	**GL32h**		

Member details

Service class of timber	**1**	Load duration	**Short-term**
Length of span	L_{s1} = **10000** mm		
Length of bearing	L_b = **100** mm		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c.90.d}$ = **2.501** N/mm^2	Design compressive strength	$f_{c.90.d}$ = **1.800** N/mm^2

FAIL - Design compressive stress exceeds design compressive strength at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **26.679** N/mm^2 Design bending strength $f_{m.d}$ = **24.149** N/mm^2

FAIL - Design bending stress exceeds design bending strength

Shear - cl.6.1.7

Applied shear stress τ_d = **1.493** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

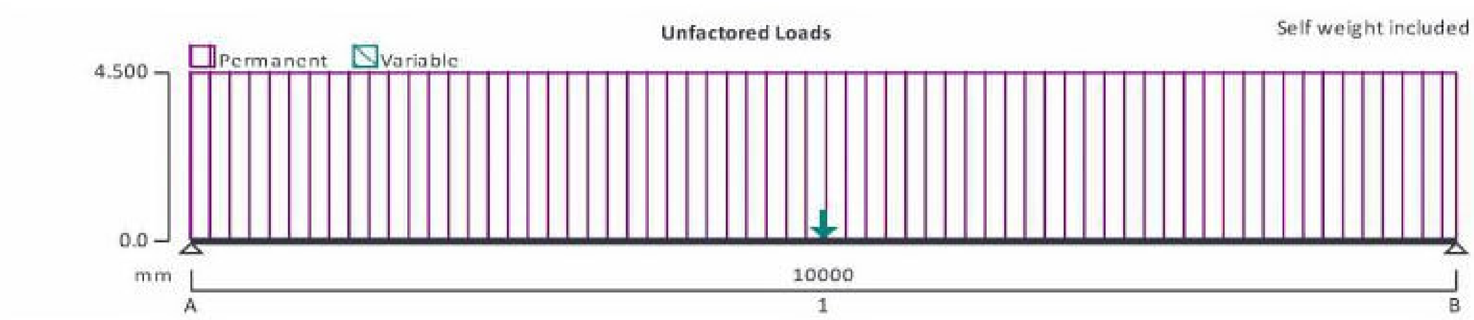
Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **105.142** mm

FAIL - Total final deflection exceeds the deflection limit

GLULAM 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

Permanent self weight of beam × 1

Permanent full UDL 4.500 kN/m

Variable point load 0.900 kN at 5000 mm

Analysis results

Design moment M = **84.364** kNm — Design shear F = **33.071** kN

Total load on member W_{tot} = **66.141** kN

Reactions at support A R_{A_max} = **33.071** kN — R_{A_min} = **33.071** kN

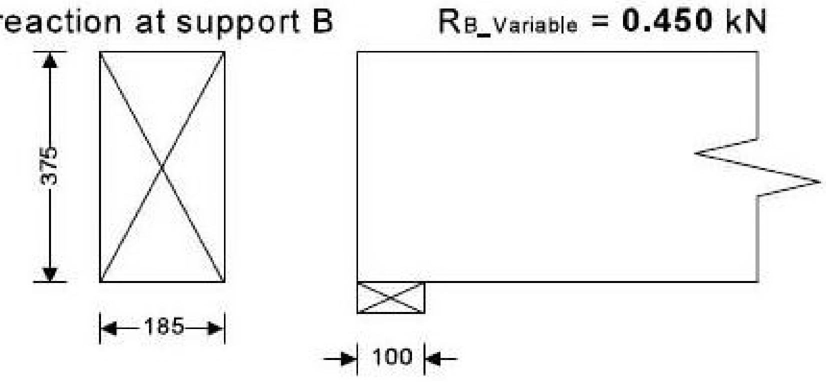
Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **23.997** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.450** kN

Reactions at support B R_{B_max} = **33.071** kN — R_{B_min} = **33.071** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **23.997** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.450** kN



Glulam section details

Breadth of section b = **185** mm — Depth of section h = **375** mm

Number of sections N = **1** — Breadth of member b_b = **185** mm

Glulam strength class **GL32h**

Member details

Service class of timber **1** — Load duration **Short-term**

Length of span L_{s1} = **10000** mm

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **1.788** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.800** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **19.457** N/mm^2 Design bending strength $f_{m.d}$ = **24.149** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **1.067** N/mm^2 Permissible shear stress $f_{v.d}$ = **2.520** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **14.000** mm Total final deflection δ_{fin} = **91.152** mm

FAIL - Total final deflection exceeds the deflection limit

Project			Status		
OVINGHAM FIRST SCHOOL					JASPER KERR CONSULTING
Date 14 / 12 / 20 By SRB Checked	Job No. JK-6289	Section	Sheet No. 30	Rev 1	

Rev	Date	Details	Tel / Fax
		POST - INTRUSIVE SURVEY	Tel Fax

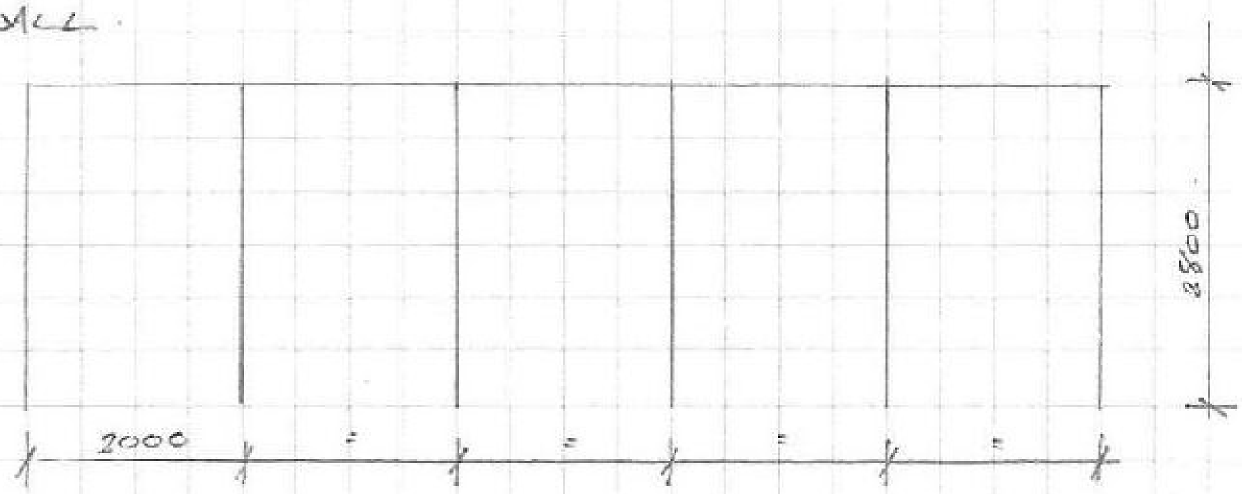
Part: OUTLINE CALCULATIONS.

LATERAL STABILITY

WIND LOAD : 0.56 kN/m² (FROM REV. Ø)

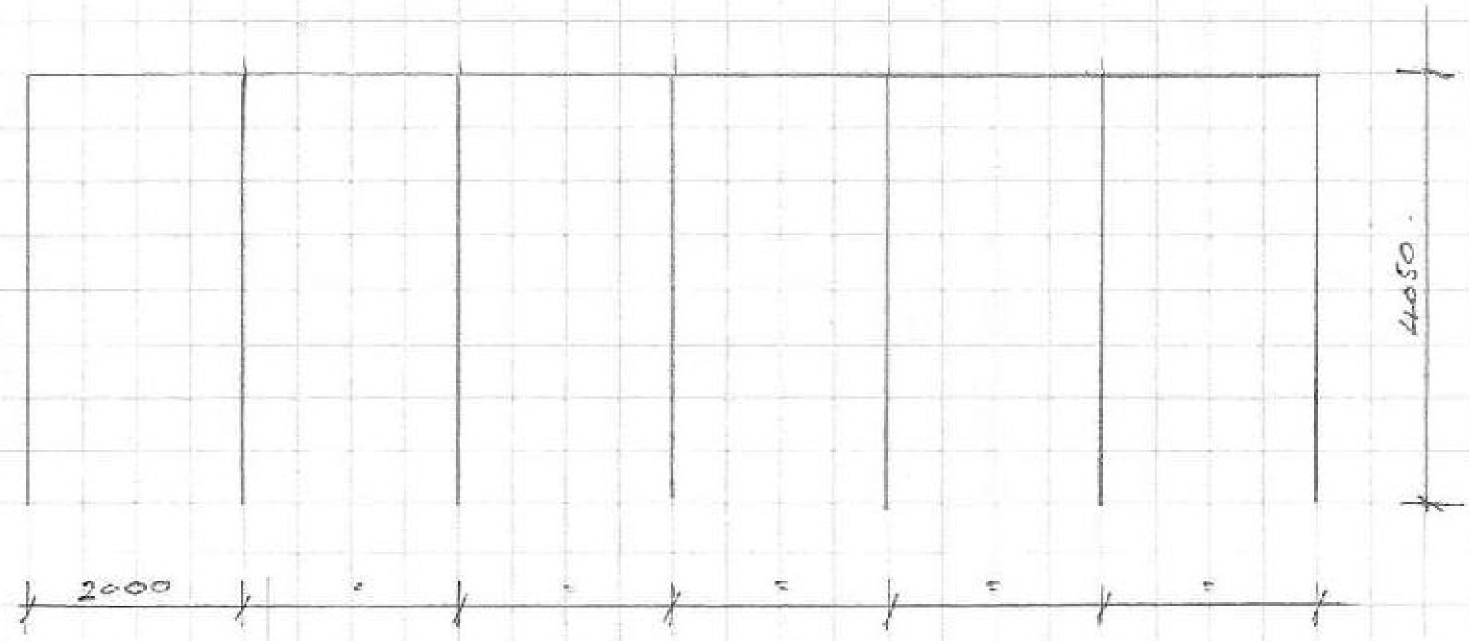
REV. Ø VERIFIES OUT-OF-PLANE STABILITY
→ CHECK 2NO. CONDITIONS;

BASIC WALL.



SPACED @ 4M. & HEIGHT / 200 = 14mm.

MAIN HALL.



SPACED @ 6M & HEIGHT / 200 = 20.25mm

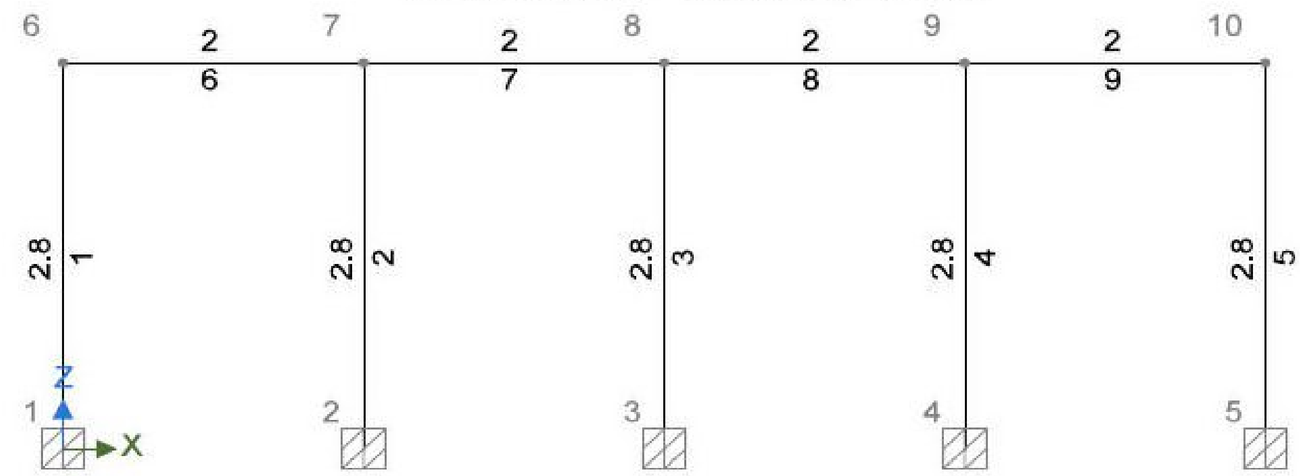
@ 4M. UDL. q_k = 0.56 × 4
= 2.24 kN/m
@ 6M. UDL. q_k = 0.56 × 6
= 3.36 kN/m

ANALYSIS

Tedds calculation version 1.0.26

Geometry

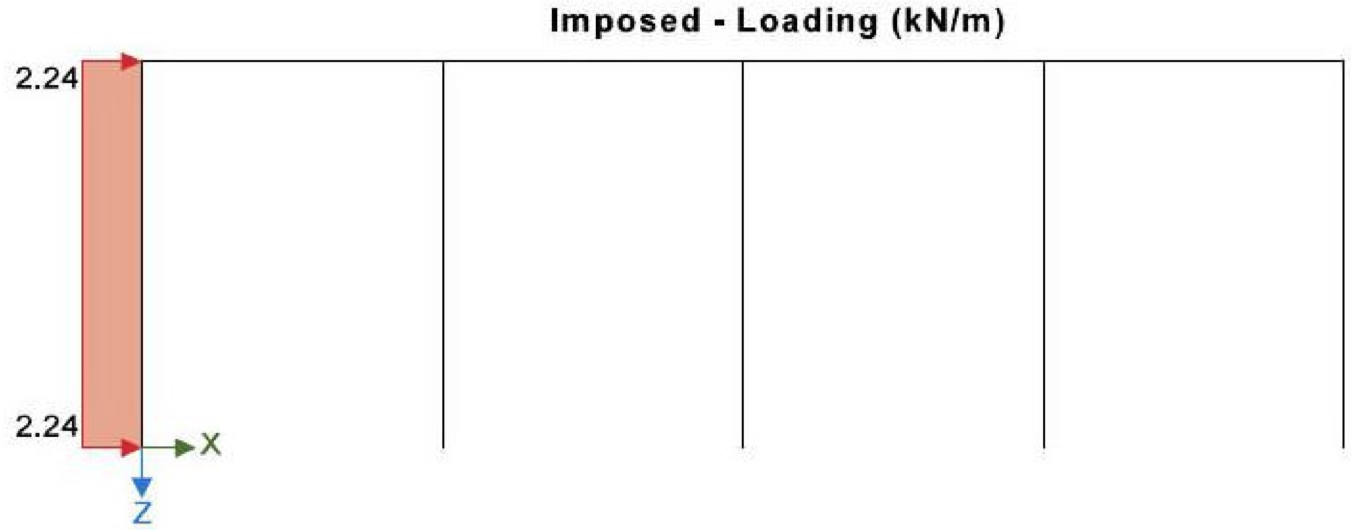
Geometry (m) - C24 (EC5) - 75x75



Loading

Self weight included

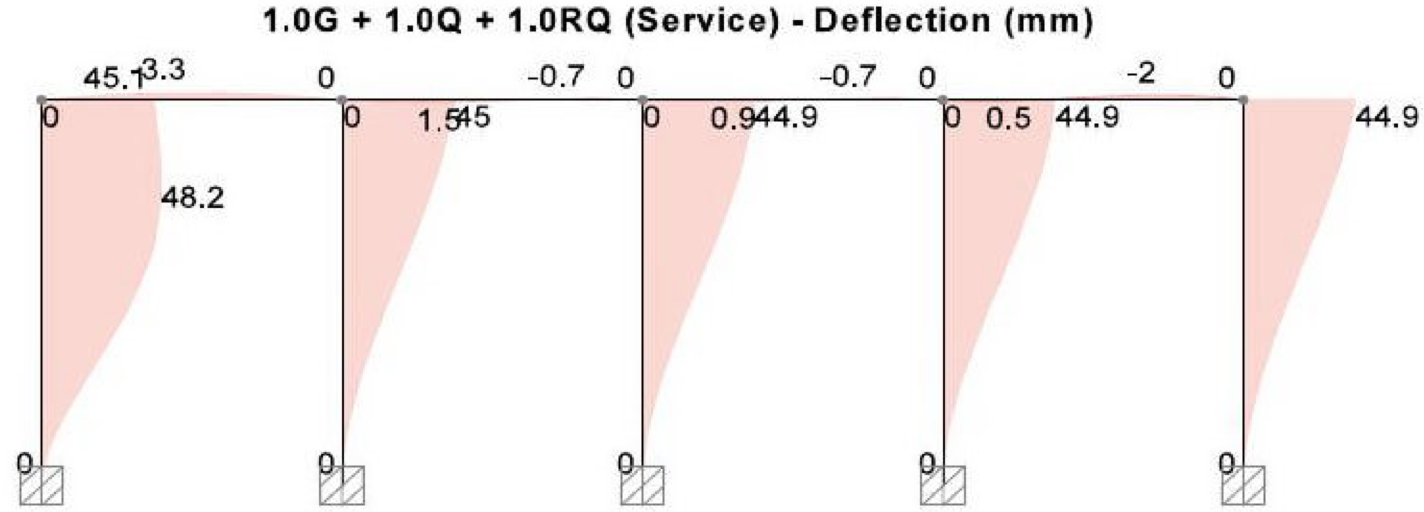
Imposed - Loading (kN/m)



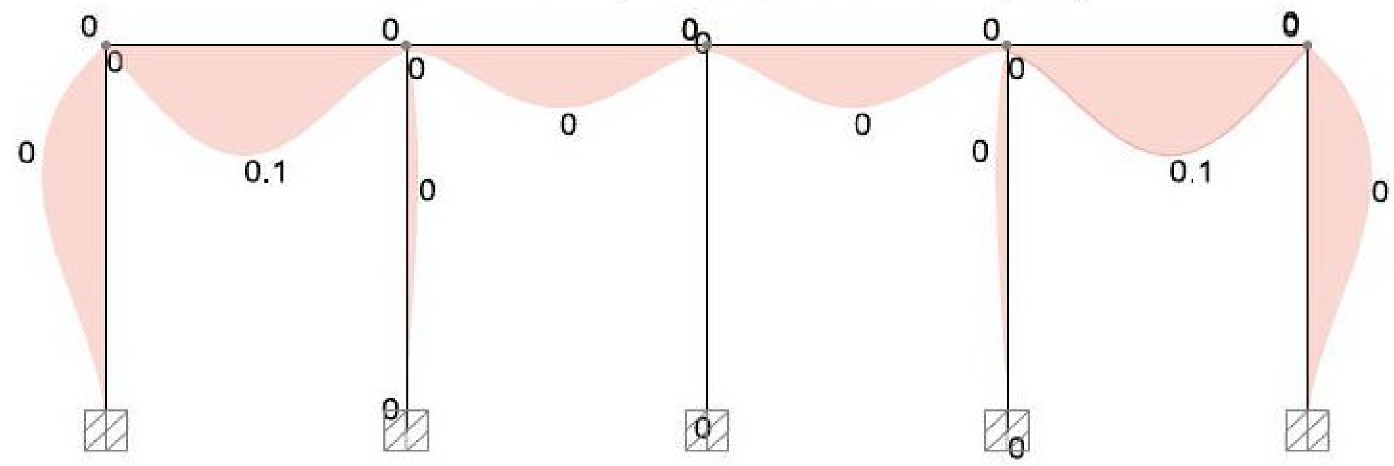
Results

Forces

1.0G + 1.0Q + 1.0RQ (Service) - Deflection (mm)



1.0G + 1.0W (Service) - Deflection (mm)

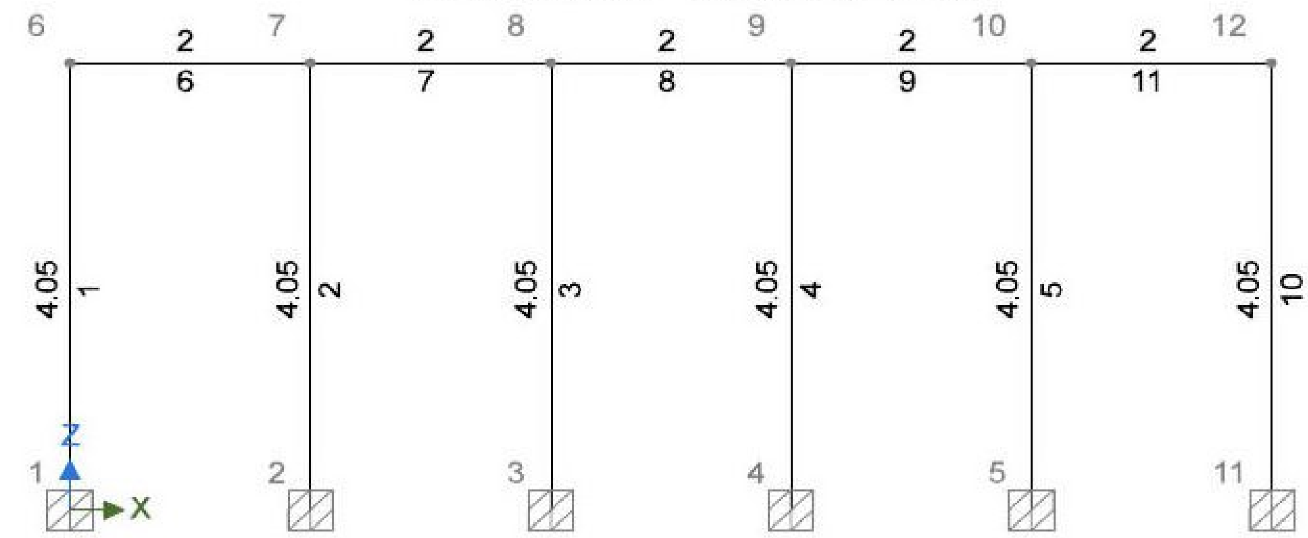


ANALYSIS

Tedds calculation version 1.0.26

Geometry

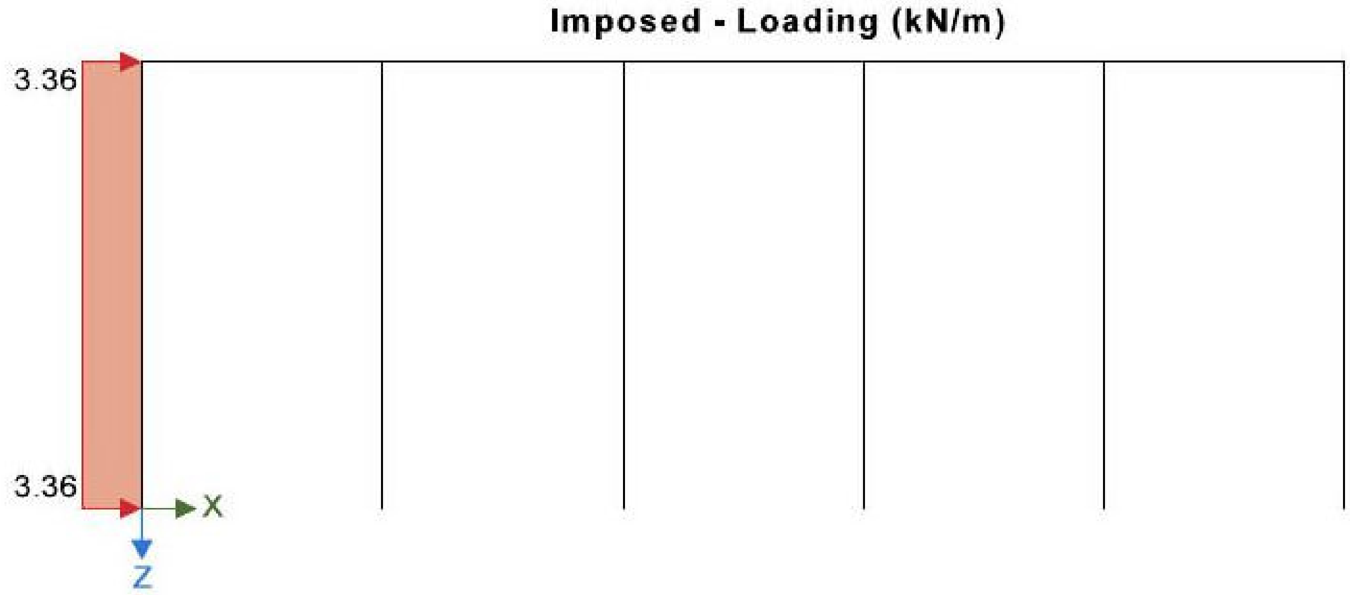
Geometry (m) - C24 (EC5) - 75x75



Loading

Self weight included

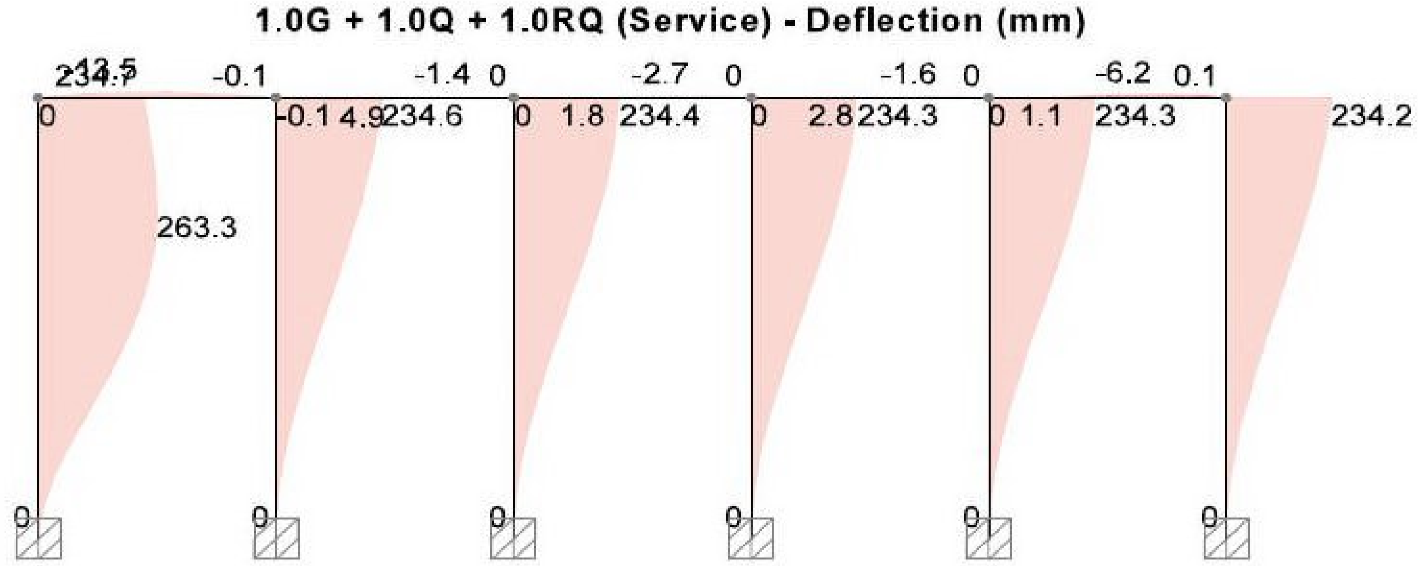
Imposed - Loading (kN/m)



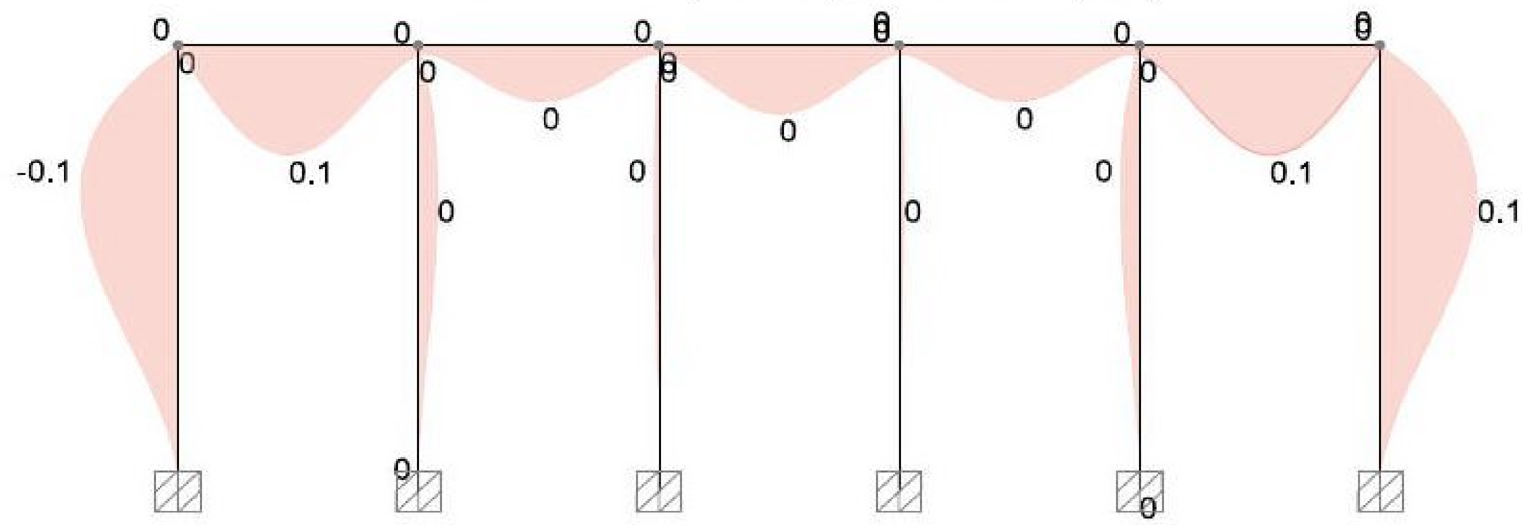
Results

Forces

1.0G + 1.0Q + 1.0RQ (Service) - Deflection (mm)



1.0G + 1.0W (Service) - Deflection (mm)



Project: EVINGHAM FIRST SCHOOL				Status	
Date 14/12/20 By SRB Checked		Job No. JK-6289	Section	Sheet No. 35	Rev 1

JASPER KERR

Rev	Date	Details
		POST-INTRUSIVE SURVEY

Tel

Fax

Part: OUTLINE CALCULATIONS

FRAMES DEFLECT SIGNIFICANTLY MORE THAN LIMITS.
→ BUILDING RESTRAINED BY INTERNAL SHEATHED WALLS BUT SHOWS THAT PERIMETER FRAME ALONE IS INADEQUATE.