Structural Appraisal

Parker Place Farm
Pendleton Road
Wiswell
BB7 9BZ

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Date: 5th March 2021

Reference: 4509-21

CONTENTS

1.0 TERMS OF REFERENCE

This report has been prepared at the request of Mr. Bruce Mitchell, the owner of the above property.

The inspection was carried out on Friday 5th March 2021.

2.0 PURPOSE OF REPORT

The purpose of the report is to inspect and comment on the overall structural condition of the barn and its suitability for part conversion to a dwelling.

The report is limited to the structural elements of the barn only and is not intended as a detailed condition report on the barn as a whole. We have not inspected the drainage system, electrical or gas installations and are therefore unable to confirm that these are in satisfactory condition.

We advise you that this report is an appraisal only and not a full structural survey. We have not inspected the woodwork or other parts of the structure which are covered, unexposed or inaccessible and we are therefore unable to report that any such part of the property is free from defect.

3.0 INTRODUCTION

The barn is located within the grounds of Parker Place Farm, Wiswell. Parker Place Farm is located to the East of Pendleton Road.

This report focuses on a small section of the barn to the East of the main barn building, highlighted in red on the site layout plan in Appendix A. We have not carried out a full inspection of the remaining sections of the barn building, however a cursory review of the buildings overall condition and stability was undertaken.

This report is required in respect of a planning application for change of use of the existing barn into residential accommodation.

The barn has a North West to South East orientation with the front elevation facing approximately due South West.

4.0 DESCRIPTION

At the time of our inspection the conversion of the Eastern section of the barn was partially complete. The building was water tight with a new floor and roof coverings installed – see general pictures in Appendix C.

This section of the barn is rectangular in plan and appears to have been built as an extension to the original double height central barn at some time in the past. We are informed by the client that this area was previously used as a hay store.

The external walls are formed in natural stone with stone quoins to the external corners.

The windows and doors have natural stone heads and cills. Some of which have been replaced with new during the conversion work. New UPVC windows and doors have been installed to the openings.

There are clay vent pipes at verge level of the East gable elevation.

The windows and door openings have stone lintels to the outer leaf with timber lintels to the inner leaf. Some of the internal timber lintels have been replaced with concrete lintels which could be seen at the time of our inspection.

There is a single door opening to the front elevation, with various window openings in the side and rear elevations.

The barn has a duo pitch roof clad in stone roof tiles. There are 3 purlins per slope and a ridge board to the apex of the roof. The purlins span the full width of the building. We were informed by the client that he has installed a new ventilated warm roof construction.

The first floor is of timber construction. The existing 175 x 75 joists span from the front to the back of the building supported at mid span by a newly installed 203x133 steel beam.

See floor plans in Appendix B, for further details of the internal layout.

The ground floor consists of a new slab on solid construction. A visqueen barrier has been installed onto hardcore and lapped approximately 1m up the face of the existing barn walls. Above this is a 100mm concrete slab with insulation and a floating floor finish.

5.0 STRUCTURAL APPRAISAL

5.1 External Walls

The external walls to all elevations have satisfactory vertical alignment and there are no visual signs to indicate any recent movement of the foundations.

We note that the walls have been recently re-pointed as part of the conversion works.

We note that some of the stone heads and jambs have been replaced with new.

See photographs in Appendix C for general views of the external walls.

5.2 Internal Walls

The internal walls have been lined with a new non-load bearing timber stud. Although close inspection was limited due to the internal stud lining the walls appeared to have satisfactory vertical alignment.

Some of the internal timber lintels have been replaced with new concrete lintels. The newly installed concrete lintels appear to be adequately sized for the load requirements.

5.3 First Floor Construction

The existing 75 x 175 floor joists have been retained and are supported at mid span with a new 203x133 UB steel beam.

Design checks carried out on the joists and steel beams confirm that they are adequate for the proposed use. Refer to calculations in Appendix D.

5.4 Roof Construction

The existing oak purlins have been retained and treated. Design checks carried out on the purlins confirm that they are adequate for their current use. Refer to calculations in Appendix D.

We were informed by the owner of the building that the existing roof was taken off and new battens, felt and insulation has been installed as part of the conversion work. The roof when viewed externally from ground level had satisfactory horizontal alignment and appeared to be in satisfactory structural condition.

5.5 Overall Stability

The stability of the Eastern section of the barn is obtained by the diaphragm action of the floors and roof in conjunction with the perimeter load bearing masonry walls. Should the remaining unconverted area of barn be left to deteriorate we are satisfied that the East converted section will remain stable and structurally sound.

Similarly, as the Eastern section of the barn appears to be an extension to the original building, we are satisfied that the proposed works to this section of the barn do not have any structural implications on the barn as a whole.

6.0 DISCUSSION AND RECOMMENDATIONS

The existing structural elements of the barn: walls, roof and floor, appear to be in sound structural condition with no visual signs to indicate any recent movement of the foundations or other structural defects.

We have carried out design check calculations on the roof purlins, floor joists and floor support beam. The calculations confirm that these elements of the building are adequate for the proposed use.

We see no reason why the existing barn should not continue to be converted and used for residential accommodation.

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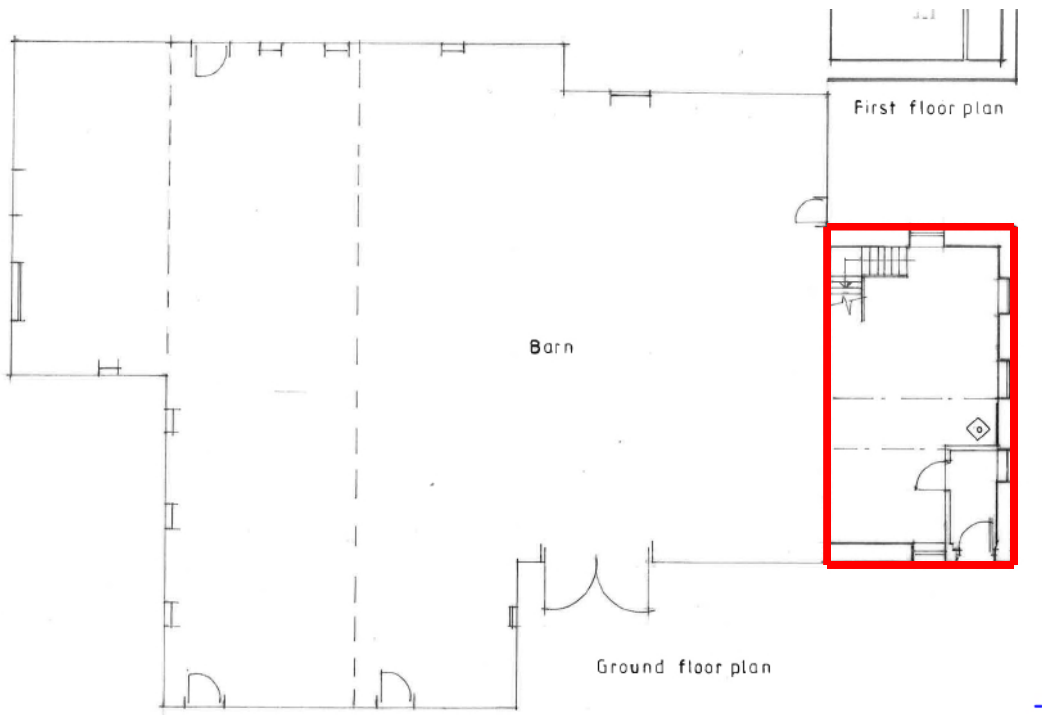
Appendix A

Sk01 –Site Layout Plan

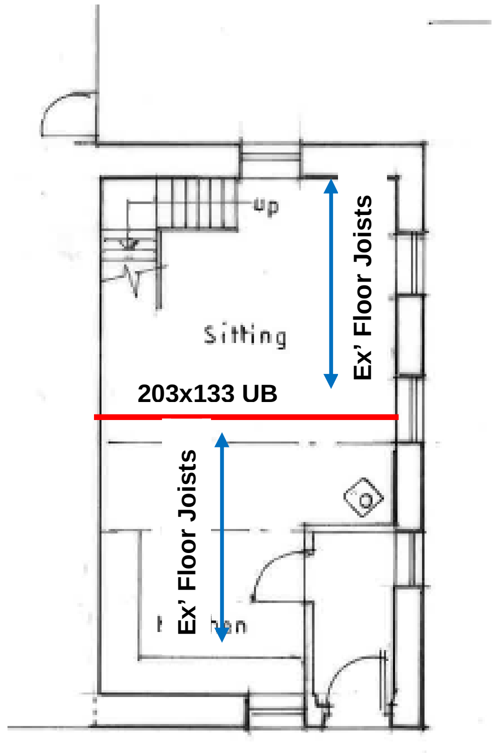


Site Layout Plan
(Not to scale)

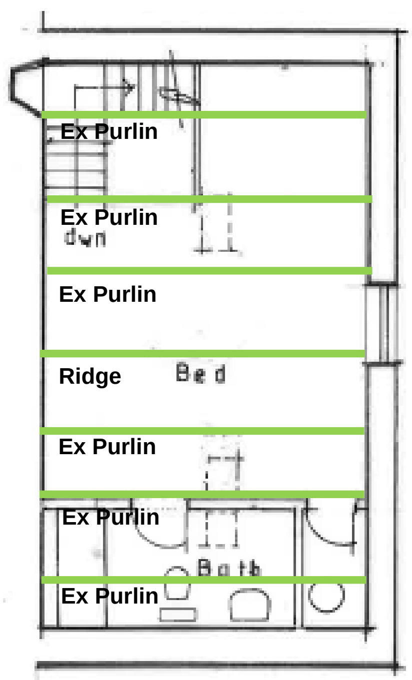
Area surveyed outlined in red

Appendix B

Sk02 – Floor Plans



Ground Floor Plan
(Not to Scale)



First Floor Plan
(Not to Scale)

Appendix C

Photographs

Front Elevation

Side Elevation (East)

Rear Elevation

Rear Elevation

Party Wall
Viewed from within main barn building

New 203x133 beam supporting existing floor joists

Existing Purlins & Ridge Member

Appendix D

Calculations

EXISTING FLOOR JOIST – DESIGN CHECK

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth; $b = \mathbf{75}$ mm

Joist depth; $h = \mathbf{175}$ mm

Joist spacing; $s = \mathbf{600}$ mm

Timber strength class; **C24**

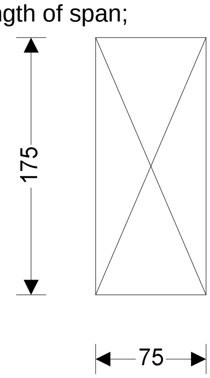
Service class of timber; **1**

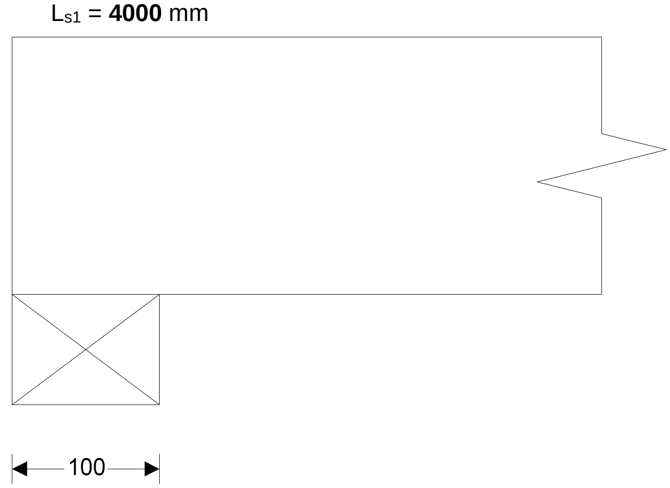
Span details

Number of spans; $N_{span} = \mathbf{1}$

Length of bearing; $L_b = \mathbf{100}$ mm

Effective length of span; $L_{s1} = \mathbf{4000}$ mm





Section properties

Second moment of area; $I = b \times h^3 / 12 = \mathbf{33496094}$ mm^4

Section modulus; $Z = b \times h^2 / 6 = \mathbf{382813}$ mm^3

Loading details

Joist self weight; $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.05}$ kN/m

Dead load; $F_{d\_udl} = \mathbf{0.45}$ kN/m^2

Imposed UDL(Long term); F_{i_udl} = **1.50** kN/m^2

Imposed point load (Medium term); F_{i_pt} = **1.40** kN

Modification factors

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

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor; K_7 = **1.06**

Load sharing factor; K_8 = **1.10**

<u>Consider long term loads</u>

Load duration factor; K_3 = **1.00**

Maximum bending moment; M = **2.430** kNm

Maximum shear force; V = **2.430** kN

Maximum support reaction; R = **2.430** kN

Maximum deflection; δ = **11.525** mm

Check bending stress

Bending stress; σ_m = **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.754** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **6.348** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.278** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.324** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm})$ = **12.000** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **11.196** mm

Shear deflection; δ_{shear} = **0.329** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **11.525** mm

PASS - Actual deflection within permissible limits

<u>Consider medium term loads</u>

Load duration factor; K_3 = **1.25**

Maximum bending moment; M = **2.030** kNm

Maximum shear force; V = **2.030** kN

Maximum support reaction; R = **2.030** kN

Maximum deflection; δ = **8.338** mm

Check bending stress

Bending stress; σ_m = **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.942** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.303** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.232** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.300** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.271** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **12.000** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **8.063** mm

Shear deflection; δ_{shear} = **0.275** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **8.338** mm

PASS - Actual deflection within permissible limits

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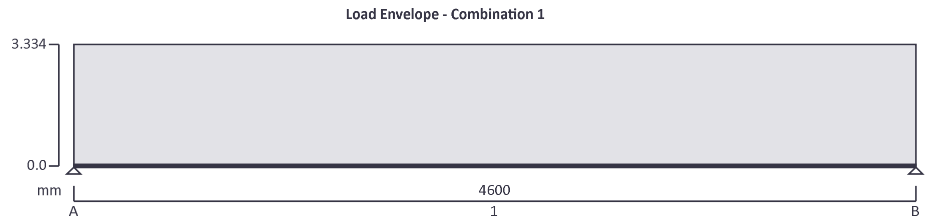
Partington Associates	Project: Parker Place Farm, Wiswell	Job Ref.: 4509-21
	Section: Existing Purlin – Design Check	Sheet no./rev.: 1

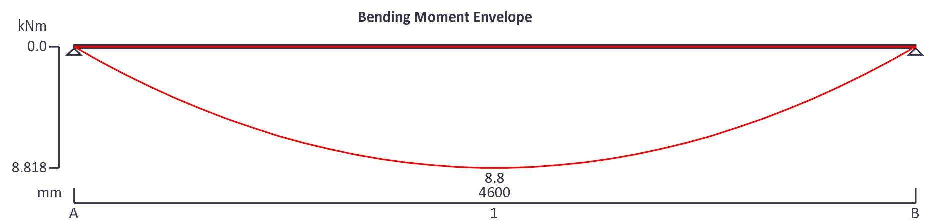
Calc. by	Date	Chk'd by	Date	App'd by	Date
SB	16/03/2021	DT	16/03/2021	DO	16/03/2021

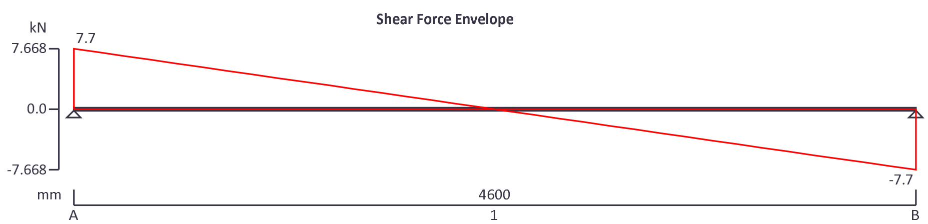
EXISTING PURLIN – DESIGN CHECK

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead full UDL 2.150 kN/m

Imposed full UDL 0.980 kN/m

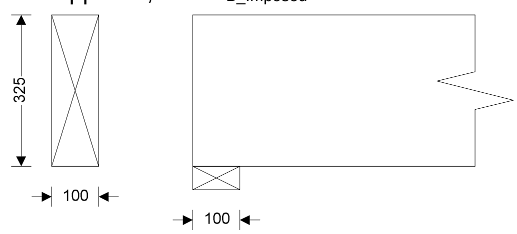
Dead self weight of beam × 1

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **8.818** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **8.818** kNm	
Maximum shear;	F_{max} = **7.668** kN;	F_{min} = **-7.668** kN
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **7.668** kN	
Total load on beam;	W_{tot} = **15.336** kN	
Reactions at support A;	R_{A_max} = **7.668** kN;	R_{A_min} = **7.668** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **5.414** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **2.254** kN	
Reactions at support B;	R_{B_max} = **7.668** kN;	R_{B_min} = **7.668** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **5.414** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **2.254** kN	



Timber section details

Breadth of sections;	b = **100** mm
Depth of sections;	h = **325** mm
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **100** mm
Timber strength class;	**D30**

Member details

Service class of timber;	**1**
Load duration;	**Long term**
Length of span;	L_{s1} = **4600** mm
Length of bearing;	L_b = **100** mm

Section properties

Cross sectional area of member;	A = N × b × h = **32500** mm^2
Section modulus;	$Z_x = N \times b \times h^2 / 6$ = **1760417** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **541667** mm^3
Second moment of area;	$I_x = N \times b \times h^3 / 12$ = **286067708** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **27083333** mm^4
Radius of gyration;	$i_x = \sqrt{(I_x / A)}$ = **93.8** mm
	$i_y = \sqrt{(I_y / A)}$ = **28.9** mm

Modification factors

Duration of loading - Table 17;	K_3 = **1.00**
Bearing stress - Table 18;	K_4 = **1.00**
Total depth of member - cl.2.10.6;	$K_7 = 0.81 \times (h^2 + 92300\ mm^2) / (h^2 + 56800\ mm^2)$ = **0.99**

Load sharing - cl.2.9; K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19; **4.00**

Actual depth-to-breadth ratio; $h / (N \times b)$ = **3.25**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane); $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.800** N/mm^2

Applied bearing stress; $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.767** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.274**

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

Bending parallel to grain

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **8.883** N/mm^2

Applied bending stress; $\sigma_{m\_a} = M / Z_x$ = **5.009** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.564**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **1.400** N/mm^2

Applied shear stress; $\tau_a = 3 \times F / (2 \times A)$ = **0.354** N/mm^2

τ_a / τ_{adm} = **0.253**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection; $E = E_{min}$ = **6000** N/mm^2

Permissible deflection; δ_{adm} = min(0.551 in, 0.003 × L_{s1}) = **13.800** mm

Bending deflection; δ_{b_s1} = **11.324** mm

Shear deflection; δ_{v_s1} = **0.868** mm

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **12.193** mm

δ_a / δ_{adm} = **0.884**

PASS - Total deflection is less than permissible deflection

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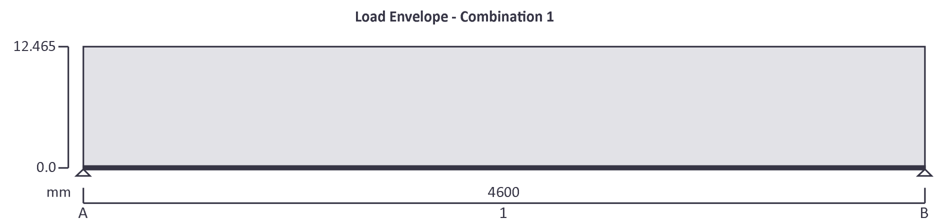
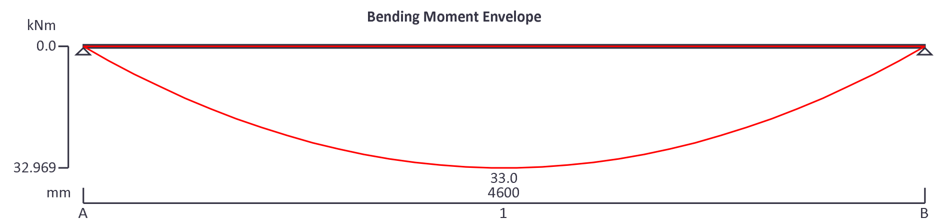
203 X 133 UB BEAM – DESIGN CHECK

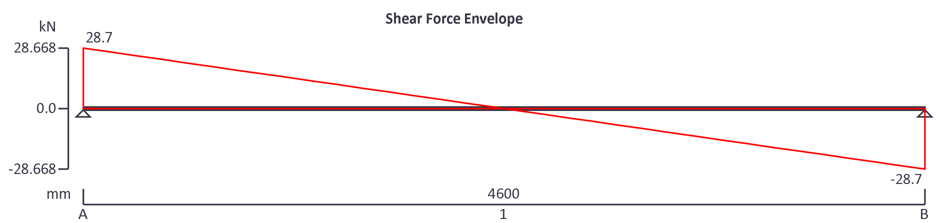
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead full UDL 1.8 kN/m
	Imposed full UDL 6 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40

Imposed × 1.60
Dead × 1.40
Imposed × 1.60
Support B — Dead × 1.40
Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **33** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **28.7** kN;	V_{min} = **-28.7** kN
Deflection;	δ_{max} = **9.8** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **28.7** kN;	R_{A_min} = **28.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **4.7** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **13.8** kN	
Maximum reaction at support B;	R_{B_max} = **28.7** kN;	R_{B_min} = **28.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **4.7** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **13.8** kN	

Section details

Section type; **UB 203x133x25 (BS4-1)**

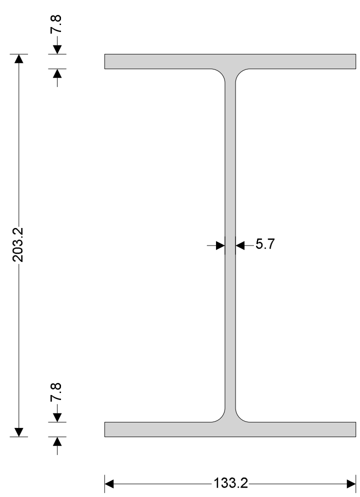
Steel grade; **S275**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **7.8** mm

Design strength; p_y = **275** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.20**; + 2 × D

$K_{LT.B}$ = **1.20**; + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275\ N/mm^2 / p_y]} =$ **1.00**

Internal compression parts - Table 11

Depth of section; $d =$ **172.4** mm

$d / t = 30.2 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 =$ **66.6** mm

$b / T = 8.5 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = max(abs(V_{max}), abs(V_{min})) =$ **28.7** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D =$ **1158** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v =$ **191.1** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **33** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **70.9** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 =$ **12.778** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **9.778** mm

PASS - Maximum deflection does not exceed deflection limit

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