

MARSH DESIGN LIMITED

STRUCTURAL DESIGN CONSULTANTS & SURVEY REPORTS

The Stable Block, Buckden Mount

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Company No: 457 2796

01484 - 341426

STRUCTURAL CALCULATIONS

FOR

INTERNAL AND EXTERNAL ALTERATIONS

FOR

CHRIS FOX

**112, HECKMONDWIKE ROAD, DEWSBURY MOOR,
DEWSBURY, WF13 3NT**

OUR REF: MDL – 9182

**REV.01: Issued for Building Control Approval Only
2nd October 2023**

Structural Notes:

These calculations are issued for building control approval only. Any structural construction works undertaken prior to approval being obtained are done so at risk by the client / contractor.

If any queries arise on site or if there are any doubts, then ask the structural engineer immediately for further guidance.

All CDM requirements are the responsibility of the client and principal contractor. We have not been engaged to undertake the role of principal designer.

It is the client's responsibility to ensure that all the requirements of the Party Wall Act are met during entire construction period of the project.

The sketches and marked up drawings contained within these structural calculations are NOT setting out or construction drawings, and they must NOT be used for setting out of steelwork or foundations.

All setting out and positioning of steel beams is the full responsibility of the building contractor responsible for the works.

All stated dimensions are clear dimensions for calculation purposes only.

The builder must confirm the lengths of all beams, columns and other structural elements from his own site measurements allowing a minimum of 100mm end bearing at both ends of the beam.

The builder is responsible for all temporary propping works, for both internal and external works.

All beams must be supported on reinforced concrete padstones unless noted otherwise in the calculations.

Structural integrity and stability of all retained masonry piers or walls is to be confirmed after all openings have been broken out. This is generally undertaken by the building control inspector, but in certain instances approval by the appointed structural engineer may be necessary.

Upon commencement of the works, when the retained walls supporting the steelwork have been exposed, new masonry piers or steel columns may be required to support the steel beams.

All steel work must be minimum grade BS EN 10025 S355.

All welded, plate and bolted cleat connections details must be confirmed by the fabricator. Any beam connection details included within these calculations are indicative only.

All bolts to be grade 8.8, unless noted otherwise.

It is recommended that any exposed steelwork in external walls is galvanised, however other coating alternatives maybe used to satisfaction of building control inspector.

The width of all proprietary lintels must be confirmed on site by the builders – to suit the width of the cavity walls.

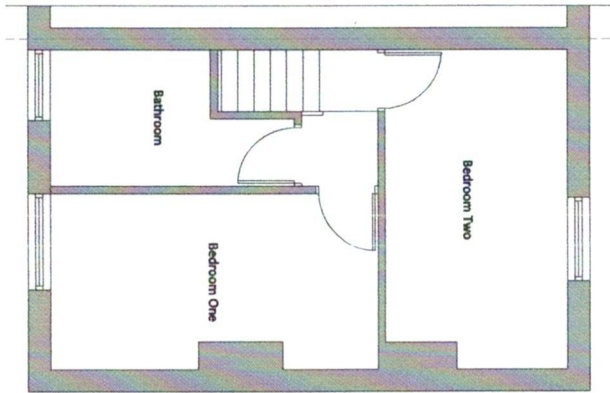
All timber to be minimum grade C24 unless noted otherwise.

Strengthening or underpinning of the existing foundations maybe required due to increased point loadings on the walls. Foundations must be exposed and checked on site prior to installations of the beams.

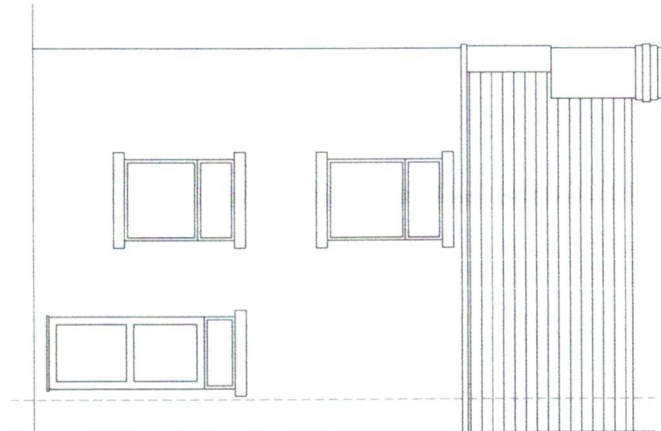
This drawing has been prepared specifically for the purpose of obtaining Planning Permission and/or Building Regulation Approval. Its suitability for other purposes without supplementary details and specification cannot be guaranteed. The Architects control, and no guarantee that such will be granted is given or to be inferred by reason of the preparation of this drawing. Only figured dimensions are to be used. All dimensions to be checked on site. This drawing together with the design is the property and copyright of the Architect and must not be reproduced without prior written permission.



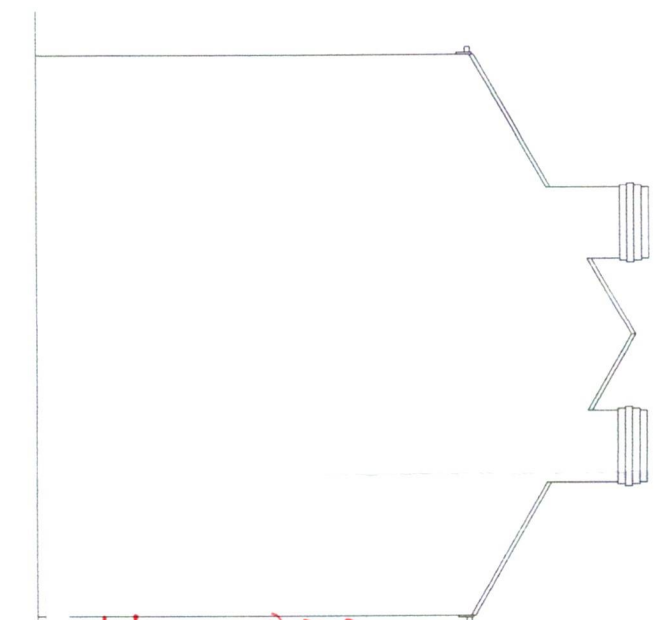
Existing Ground Floor Plan



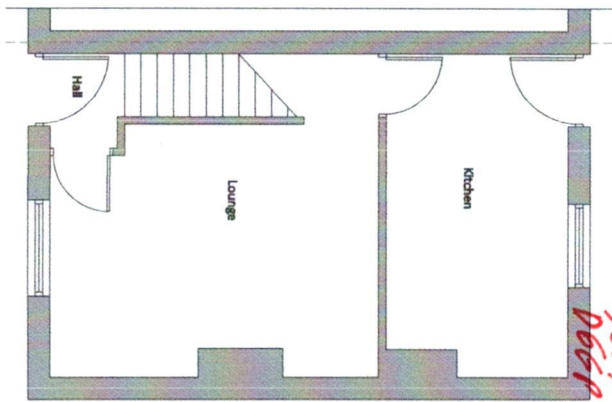
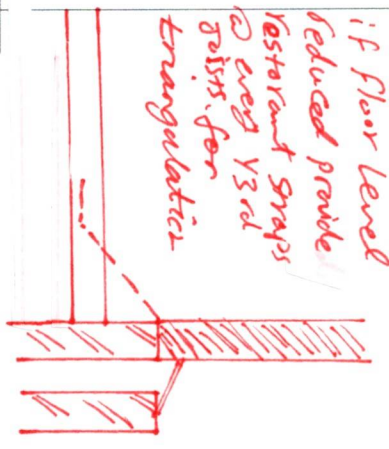
Existing First Floor Plan



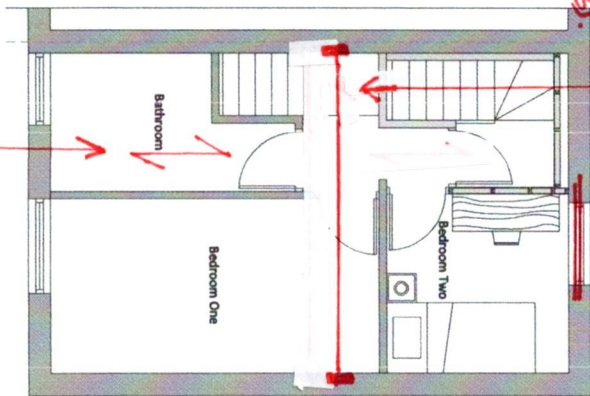
Existing Rear Elevation



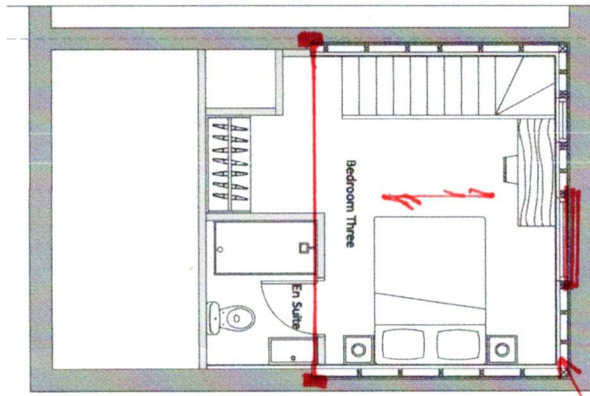
Existing Side Elevation



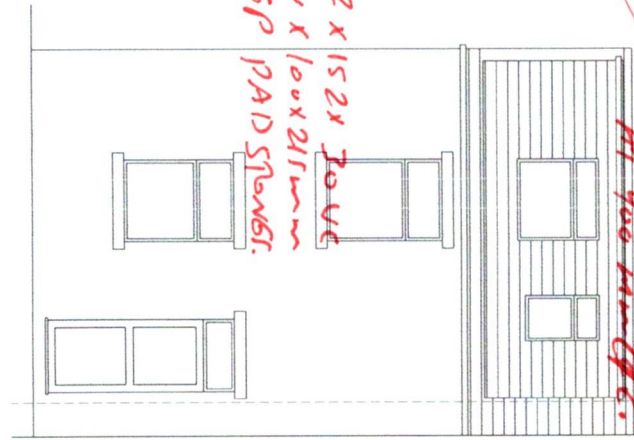
Proposed Ground Floor Plan



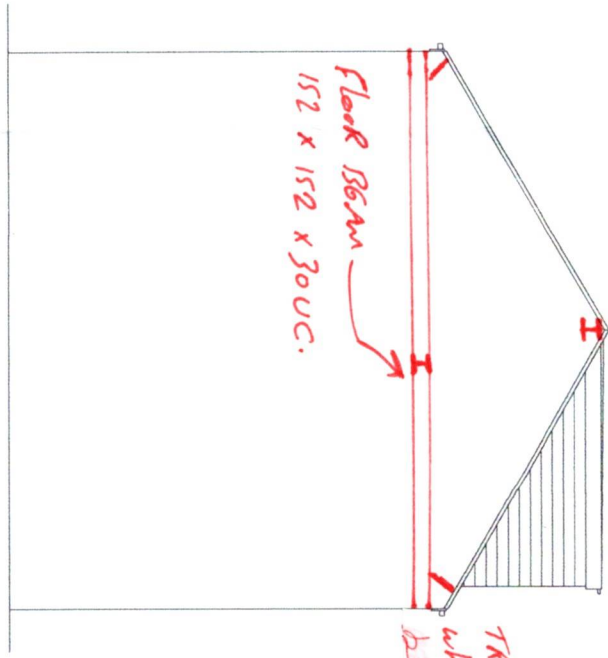
Proposed First Floor Plan



Proposed First Floor Plan



Proposed Rear Elevation



Proposed Side Elevation

Floor Beam
152 x 152 x 30 UC
300 x 100 x 215 mm
D66P PADS BOND.

2nd. 150 x 50 mm (C16)
From Reliefs listed
over 6th window.

3rd. 150 x 50 mm (C16)
to support Reliefs listed.

STUD WALL
150 x 50 mm (C16)
At 400 mm c/c.

152 x 152 x 30 UC
300 x 100 x 215 mm
D66P PADS BOND.

RIDGE 152 x 152 x 30 UC.

Floor Beam
152 x 152 x 30 UC.

TRIANGULATION
when floor level
been reduced.

Floor Joists

175 x 50 mm (C24)

At 400 mm c/c.

(MAX SPANS 3.8m).

Roof Joists

150 x 50 mm (C24)

At 400 mm c/c.

SK-01
MDL-9182

Client:-		Scheme:-	
112 Hedmondwile Road, Densbury,		Rear Dormer Loft Conversion	
WF13 3NT		PL-01	
Drawing Description:-		Date:-	
Existing & Proposed Plans and Elevations		JUL 23	
Scale:-		Architect:-	
1:50			

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Loading Summary				Start page no./Revision 1A	
Calcs by BRR	Calcs Date 02/10/2023	Checked by <i>D. Singh</i>	Checked Date 2/10/2023	Approved by <i>D. Singh</i>	Approved Date 2/10/2023

All calculations

LOADINGS

Roof

DL	=	1.5 kN/m ²	(Concrete Tiles (inc. ceilings))
IL	=	0.8 kN/m ²	(Snow (0.8))
Total	=	2.3 kN/m ²	

Dormer Roof

DL	=	1.0 kN/m ²	(Lightweight (TAPCO, GRP etc.))
IL	=	0.8 kN/m ²	(Snow (0.8))
Total	=	1.8 kN/m ²	

Cavity Walls

100 Outer Stone	=	0.1 x 24.0 kN/m ³	=	2.4 kN/m ²
Cavity Insulation	=		=	0.1 kN/m ²
100 Inner Block	=	0.1 x 20.0 kN/m ³	=	2.0 kN/m ²
Plasterboard and Skim	=		=	0.1 kN/m ²
Render	=		=	0.0 kN/m ²
		Total	=	4.6 kN/m ²

Dormer Walls

DL	=	1.0 kN/m ²
Cladding Tiles	=	0.6 kN/m ²
Total	=	1.6 kN/m ²

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Loading Summary				Start page no./Revision 1B	
Calcs by BRR	Calcs Date 02/10/2023	Checked by	Checked Date	Approved by	Approved Date

Floor

DL	=	0.5 kN/m ²	(Timber - With Partitions)
IL	=	2.0 kN/m ²	(Domestic + Partitions)
Total	=	2.5 kN/m ²	

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Roof Joists				Start page no./Revision 2	
Calcs by BRR	Calcs Date 02/10/2023	Checked by	Checked Date	Approved by	Approved Date

Dimensions

Span on Plan 3500 mm

Roof Loads

Dead	Lightweight Roof (TAPCO, GRP etc.)	1.0 kN/m ²
Imposed	Snow (0.8)	0.8 kN/m ²
Total		1.8 kN/m ²

Roof Joist Details

Centres 400 mm
Depth 150 mm
Width 50 mm
Angle 0 degrees

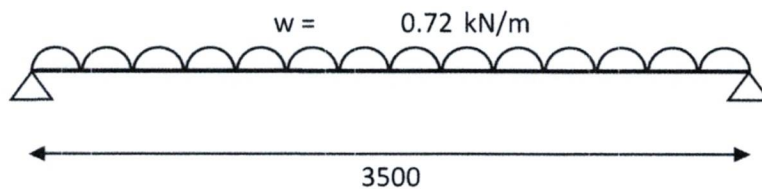
Material:
Grade C24 Softwood Timber (10800)
Allowable bending stress 7.5 N/mm²
E 10800 N/mm²

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Roof Joists				Start page no./Revision 3	
Calcs by BRR	Calcs Date 02/10/2023	Checked by	Checked Date	Approved by	Approved Date

Joists

$$\text{Max Span} = 3500 / \cos 0 = 3.5 \text{ m}$$

$$w = 0.4 \times 1.8 \text{ kN/m}^2 = 0.72 \text{ kN/m}$$



$$M = 0.125 \times 0.72 \times 3.5^2 = 1.1 \text{ kNm}$$

150 x 50 Joists (C24 Softwood Timber (10800))

$$\sigma_b = \frac{1.1 \times 10^6 \times 6}{50 \times 150^2} = 5.9 \text{ N/mm}^2 < 7.5 \text{ N/mm}^2$$

Pass

$$\delta = \frac{0.013 \times 0.72 \times 3500^4 \times 12}{10800 \times 50 \times 150^3} = 9.2 \text{ mm} < \begin{matrix} 0.003L \\ 10.5 \text{ mm} \end{matrix}$$

Pass

USE 150 x 50 JOISTS, 400mm
CENTRES - C24 Softwood Timber



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Project				Job no. MDL-9182	
Calcs for				Start page no./Revision 4	
Calcs by BRR	Calcs date 21/10/23	Checked by	Checked date	Approved by	Approved date

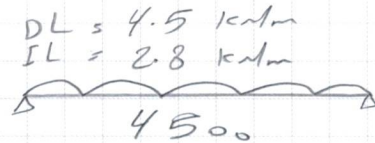
DESIGN RIDGE BEAM

$$\text{Roof DL} = 1/2 \times 3.5\text{m} \times 1.5 \text{ kNm}^2 = 2.7 \text{ kNm}$$

$$\text{D.Roof DL} = 1/2 \times 3.5\text{m} \times 1.0 \text{ kNm}^2 = 1.8 \text{ kNm}$$

$$\text{ALL Roof IL} = 1/2 \times 7.0\text{m} \times 0.8 \text{ kNm}^2 = 2.8 \text{ kNm}$$

$$\text{Clear span} = 4.5 \text{ m}$$



USE 152 X 152 X 30 UC

PADSTONES-

$$\text{Reaction, } R = 25.2 \text{ kN}$$

$$\sigma_c = (3.5 \text{ kNm}^2 \times 1.25) / 3.5 = 1.25 \text{ kNm}^2$$

$$\text{TVJ } 300 \times 100 \text{ mm}$$

$$\delta = \frac{25.2 \times 10^3}{300 \times 100} = 0.84 \text{ kNm}^2 < 1.25 \text{ kNm}^2$$

USE 300 X 100 X 215mm Deep padstones



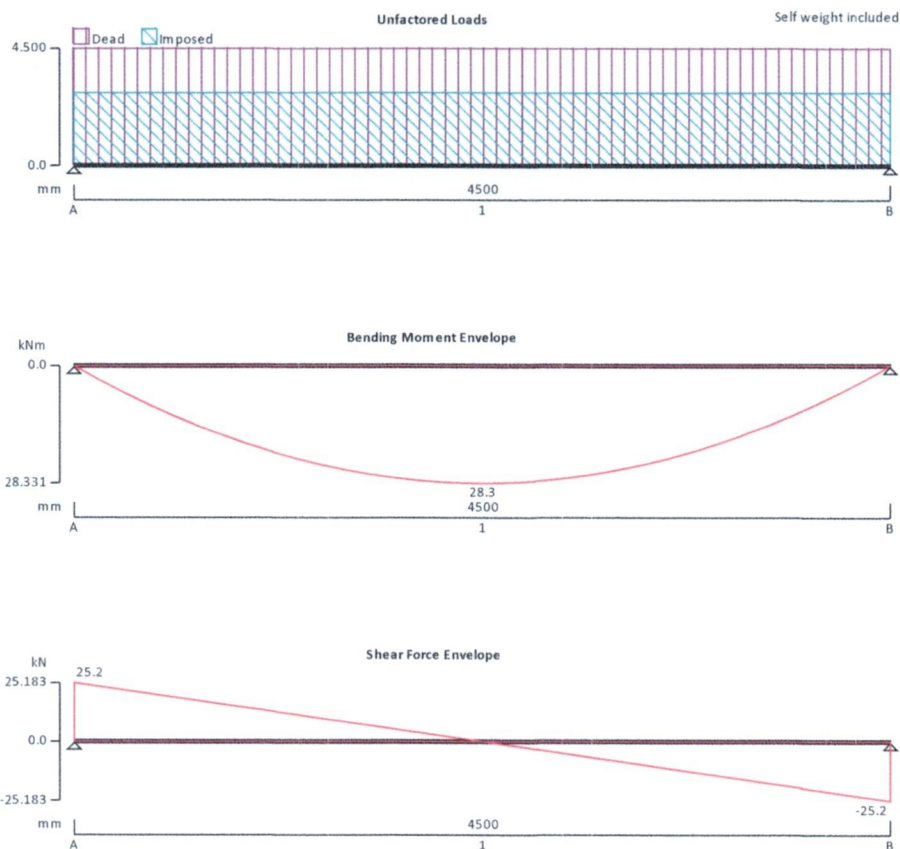
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Project				Job no.	
112 Heckmondwike Road, Dewsbury				MDL - 9182	
Calcs for				Start page no./Revision	
Ridge Steel Beam				5	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
BRR	02/10/2023				

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 self weight of beam $\times 1$

Dead full UDL 4.5 kN/m

Imposed full UDL 2.8 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$



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Project			Job no.		
112 Heckmondwike Road, Dewsbury			MDL - 9182		
Calcs for			Start page no./Revision		
Ridge Steel Beam			6		
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
BRR	02/10/2023				

Analysis results

Maximum moment	$M_{max} = 28.3 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 25.2 \text{ kN}$	$V_{min} = -25.2 \text{ kN}$
Deflection	$\delta_{max} = 11.3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 25.2 \text{ kN}$	$R_{A_min} = 25.2 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 10.8 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 6.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 25.2 \text{ kN}$	$R_{B_min} = 25.2 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 10.8 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 6.3 \text{ kN}$	

Section details

Section type	UC 152x152x30 (British Steel Section Range 2022 (BS4-1))
Steel grade	S355
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 9.4 \text{ mm}$
Design strength	$p_y = 355 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$

Lateral restraint

Span 1 has lateral restraint at supports only

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	$K_{LTA} = 1.00 + 2 \times D$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 0.88$$

Internal compression parts - Table 11

Depth of section	$d = 123.6 \text{ mm}$	
	$d / t = 21.6 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section	$b = B / 2 = 76.5 \text{ mm}$	
	$b / T = 9.2 \times \varepsilon \leq 10 \times \varepsilon$	Class 2 compact
		Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force	$F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 25.2 \text{ kN}$
	$d / t < 70 \times \varepsilon$
	Web does not need to be checked for shear buckling
Shear area	$A_v = t \times D = 1024 \text{ mm}^2$
Design shear resistance	$P_v = 0.6 \times p_y \times A_v = 218.2 \text{ kN}$
	PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment	$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 28.3 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2	$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 87.9 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling	$L_E = ((1.0 + 1.0) \times L_{s1} + 2 \times D) / 2 = 4658 \text{ mm}$
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Project 112 Heckmondwike Road, Dewsbury				Job no. MDL - 9182	
Calcs for Ridge Steel Beam				Start page no./Revision 7	
Calcs by BRR	Calcs date 02/10/2023	Checked by	Checked date	Approved by	Approved date

Slenderness ratio

$$\lambda = L_E / r_{yy} = 121.697$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.849$$

Torsional index

$$x = 15.999$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.712$$

Ratio - cl.4.3.6.9

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 73.514$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 30.198$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.303$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 374.4 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 421.5 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 210 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 21.2 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 28.3 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 21.2 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 28.3 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 28.3 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 52 \text{ kNm}$$

$$M_b / m_{LT} = 56.2 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 12.5 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 11.316 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Dormer Relieving Lintel				Start page no./Revision 8	
Calcs by BRR	Calcs Date 02/10/2023	Checked by	Checked Date	Approved by	Approved Date

Dimensions

Span on Plan	1100 mm
Width of Roof Supported	3700 mm
Height of Wall Supported	500 mm

Loads on Beam

Dormer Roof

Dead	Lightweight Roof (TAPCO, GRP etc.)	1.0 kN/m ²
Imposed	Snow (0.8)	0.8 kN/m ²
Total		<u>1.8 kN/m²</u>

Dormer Wall

Dead	Dormer Wall	1.0 kN/m ²
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Timber Beam Details

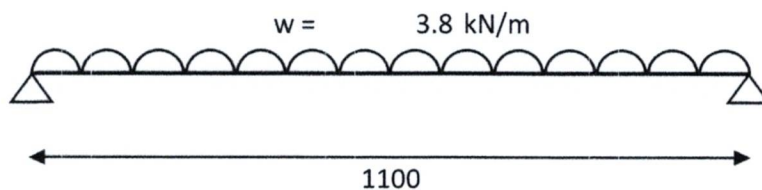
Depth	150 mm
Width	150 mm

Material:	
Grade	C16 Softwood Timber
Allowable bending stress	5.3 N/mm ²
E	8800 N/mm ²

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
Calcs for Dormer Relieving Lintel				Start page no./Revision 9	
Calcs by BRR	Calcs Date 02/10/2023	Checked by	Checked Date	Approved by	Approved Date

Timber Beam

w Roof	=	$1/2 \times 3.7\text{m} \times 1.8 \text{ kN/m}^2$	=	3.3 kN/m
w Wall	=	$0.5\text{m} \times 1.0 \text{ kN/m}^2$	=	0.5 kN/m
w Floor	=	N/A	=	0.0 kN/m
Total			=	<u>3.8 kN/m</u>



$$M = 0.125 \times 3.8 \times 1.1^2 = 0.6 \text{ kNm}$$

150 x 150 Timber Beam (C16 Softwood Timber)

$$\sigma_b = \frac{0.6 \times 10^6 \times 6}{150 \times 150^2} = 1.1 \text{ N/mm}^2 < 5.3 \text{ N/mm}^2$$

Pass

$$\delta = \frac{0.013 \times 3.8 \times 1100^4 \times 12}{8800 \times 150 \times 150^3} = 0.2 \text{ mm} < 0.003L = 3.3 \text{ mm}$$

Pass

USE 150 x 150 TIMBER BEAM - C16 Softwood Timber

Use 3 No. 150x50 joists glued and bolted together to form a single 150x150 timber lintel beam. M10 Bolts at 400mm staggered centres.



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Project

Job no.

MDL- 9182

Calcs for

Start page no./Revision

10

Calcs by

BRR

Calcs date

21/10/23

Checked by

Checked date

Approved by

Approved date

DESIGN STUD WALL

$$D. \text{Roof DL} = Y2 \times 3.5 \text{ m} \times 1.0 \text{ kNm}^2 = 1.8 \text{ kNm}$$

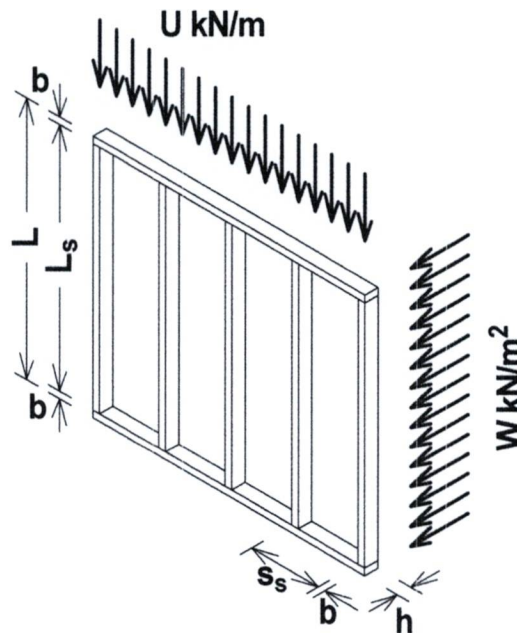
$$D. \text{Roof IL} = Y2 \times 3.5 \text{ m} \times 0.8 \text{ kNm}^2 = 1.4 \text{ kNm}$$

USG 150 x 50 mm (C16) at 400 mm C/C.

Project		112 Heckmondwike Road, Dewsbury		Job no.	
Calcs for		Timber Stud wall		Start page no./Revision	
Calcs by		Calcs date		Approved by	
BRR		02/10/2023		11	
Checked by		Checked date		Approved date	

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details

Stud breadth	$b = 50 \text{ mm}$
Stud depth	$h = 150 \text{ mm}$
Number of studs	$N_s = 1$

Strength class C16 timber (Table 8 BS5268:Pt 2:2002)

Section properties

Cross sectional area	$A = N_s \times b \times h = 7500 \text{ mm}^2$
Section modulus	$Z = N_s \times b \times h^2 / 6 = 187500 \text{ mm}^3$
Moment of inertia in the major axis	$I_x = N_s \times b \times h^3 / 12 = 14062500 \text{ mm}^4$
Moment of inertia in the minor axis	$I_y = N_s \times h \times b^3 / 12 = 1562500 \text{ mm}^4$
Radius of gyration in the major axis	$r_x = \sqrt{I_x / A} = 43.3 \text{ mm}$
Radius of gyration in the minor axis	$r_y = \sqrt{I_y / A} = 14.4 \text{ mm}$

Panel details - Studs restrained by sheathing in the plane of the panel

Panel height	$L = 2300 \text{ mm}$
Stud length	$L_s = L - (2 \times b) = 2200 \text{ mm}$
Standard stud spacing	$s_s = 400 \text{ mm}$
Panel opening	$O = 0 \text{ mm}$
Loaded panel length	$s = \max(s_s, (O + s_s) / 2) = 400 \text{ mm}$
Effective length in the major axis	$L_{ex} = 0.85 \times L_s = 1870 \text{ mm}$
Slenderness ratio	$\lambda = L_{ex} / r_x = 43.19$

Vertical loading details

Wall UDL	Dead loads	Imposed loads
Roof UDL	$U_{w,d} = 1.60 \text{ kN/m}$	$U_{r,i} = 1.40 \text{ kN/m}$
	$U_{r,d} = 1.80 \text{ kN/m}$	



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18 Howard Avenue
Lindley

Project 112 Heckmondwike Road, Dewsbury			Job no. MDL _ 9182		
Calcs for Timber Stud wall			Start page no./Revision 12		
Calcs by BRR	Calcs date 02/10/2023	Checked by	Checked date	Approved by	Approved date

Lateral loading details

Wind loading $W = 1.00 \text{ kN/m}^2$
Wind load duration **Very short term**

Modification factors

Section depth factor $K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$
Load sharing factor $K_8 = 1.10$

Consider combined axial compression and bending under very short term loads

Load duration factor $K_3 = 1.75$
Vertical loading $F = (U_{w,d} + U_{r,d} + U_{r,i}) \times s = 1.92 \text{ kN}$

Check bending stress

Bending parallel to grain $\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress $\sigma_{m,adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 11.011 \text{ N/mm}^2$
Bending moment $M_{max} = W \times s \times L^2 / 8 = 0.265 \text{ kNm}$
Applied bending stress $\sigma_{m,max} = M_{max} / Z = 1.411 \text{ N/mm}^2$

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Compression member factor $K_{12} = 0.73$
Compression parallel to grain $\sigma_c = 6.800 \text{ N/mm}^2$
Permissible compressive stress $\sigma_{c,adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 9.525 \text{ N/mm}^2$
Applied compressive stress $\sigma_{c,max} = F / (N_s \times b \times h) = 0.256 \text{ N/mm}^2$

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor $K_4 = 1.20$
Compression perpendicular to grain (no wane) $\sigma_{cp1} = 2.200 \text{ N/mm}^2$
Permissible compressive stress $\sigma_{cp1,adm} = \sigma_{cp1} \times K_3 \times K_4 = 4.620 \text{ N/mm}^2$
Applied compressive stress $\sigma_{cp1,max} = F / (N_s \times b \times h) = 0.256 \text{ N/mm}^2$

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2 = 30.693 \text{ N/mm}^2$
Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c,max} \times K_{12} / \sigma_e) = 0.991$
Combined axial compression and bending value $K = \sigma_{m,max} / (\sigma_{m,adm} \times K_{eu}) + \sigma_{c,max} / \sigma_{c,adm} = 0.156 < 1$

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2 = 30.693 \text{ N/mm}^2$
Maximum deflection $\delta_{adm} = \min(6.9 \text{ mm}, 0.003 \times (L - 2 \times b)) = 6.600 \text{ mm}$
Bending deflection $\delta_{max} = 5 \times W \times s \times L_s^4 / (384 \times E_{mean} \times I_x) = 0.986 \text{ mm}$

PASS - Deflection due to wind loading is less than permissible limit

Consider axial compression without bending under medium term loads

Load duration factor $K_3 = 1.25$
Vertical loading $F = (U_{w,d} + U_{r,d} + U_{r,i}) \times s = 1.92 \text{ kN}$

Check compressive stress on stud

Compression member factor $K_{12} = 0.76$
Compression parallel to grain $\sigma_c = 6.800 \text{ N/mm}^2$
Permissible compressive stress $\sigma_{c,adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 7.107 \text{ N/mm}^2$



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Applied compressive stress

$$\sigma_{c_max} = F / (N_s \times b \times h) = 0.256 \text{ N/mm}^2$$

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor

$$K_4 = 1.20$$

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.200 \text{ N/mm}^2$$

Permissible compressive stress

$$\sigma_{cp1_adm} = \sigma_{cp1} \times K_3 \times K_4 = 3.300 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{cp1_max} = F / (N_s \times b \times h) = 0.256 \text{ N/mm}^2$$

PASS - Applied compressive stress under medium term loads is within permissible limits

Consider axial compression without bending under long term loads

Load duration factor

$$K_3 = 1.00$$

Vertical loading

$$F = (U_{w_d} + U_{r_d}) \times s = 1.36 \text{ kN}$$

Check compressive stress on stud

Compression member factor

$$K_{12} = 0.77$$

Compression parallel to grain

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

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.795 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F / (N_s \times b \times h) = 0.181 \text{ N/mm}^2$$

PASS - Applied compressive stress under long term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor

$$K_4 = 1.20$$

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = 2.200 \text{ N/mm}^2$$

Permissible compressive stress

$$\sigma_{cp1_adm} = \sigma_{cp1} \times K_3 \times K_4 = 2.640 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{cp1_max} = F / (N_s \times b \times h) = 0.181 \text{ N/mm}^2$$

PASS - Applied compressive stress under long term loads is within permissible limits

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Dimensions

Length 3800 mm

Floor Loads

Dead	Timber Floor - No Partitions	0.5 kN/m ²
Imposed	Domestic	1.5 kN/m ²
Total		<u>2 kN/m²</u>

Floor Joist Details

Centres 400 mm
Depth 175 mm
Width 50 mm

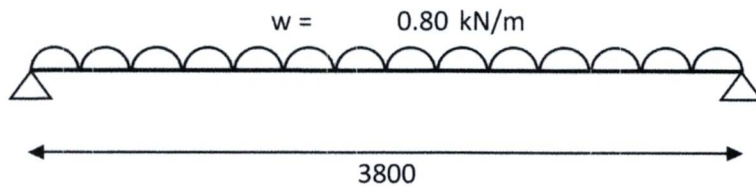
Material:
Grade C24 Softwood Timber (10800)
Allowable bending stress 7.5 N/mm²
E 10800 N/mm²

Project 112 Heckmondwike Road, Dewsbury				Job No. MDL - 9182	
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Joists

$$\text{Length} = 3.8 \text{ m}$$

$$w = 0.4 \times 2 \text{ kN/m}^2 = 0.80 \text{ kN/m}$$



$$M = 0.125 \times 0.80 \times 3.8^2 = 1.4 \text{ kNm}$$

175 x 50 Joists (C24 Softwood Timber (10800))

$$\sigma_b = \frac{1.4 \times 10^6 \times 6}{50 \times 175^2} = 5.5 \text{ N/mm}^2 < 7.5 \text{ N/mm}^2$$

Pass

$$\delta = \frac{0.013 \times 0.80 \times 3800^4 \times 12}{10800 \times 50 \times 175^3} = 9.0 \text{ mm} < 0.003L = 11.4 \text{ mm}$$

Pass

USE 175 x 50 JOISTS, 400mm
CENTRES - C24 Softwood Timber



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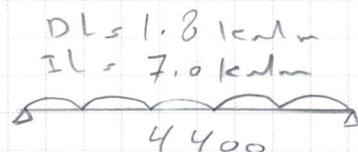
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DESIGN FLOOR BEAM

$$\text{Floor DL} = \frac{1}{2} \times 7.0 \text{ m} \times 0.5 \text{ kN/m}^2 = 1.8 \text{ kN/m}$$

$$\text{Floor IL} = \frac{1}{2} \times 7.0 \text{ m} \times 2.0 \text{ kN/m}^2 = 7.0 \text{ kN/m}$$

Clear spans 4.4 m



USE 152 X 152 X 30 UC

PADSTONES:-

$$R = 31.1 \text{ kN}$$

$$\sigma_c = 1.04 \text{ N/mm}^2 < 1.25 \text{ N/mm}^2$$

USE 300 x 100 x 215 mm Deep padstones



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Buckden Mount, Edgerton

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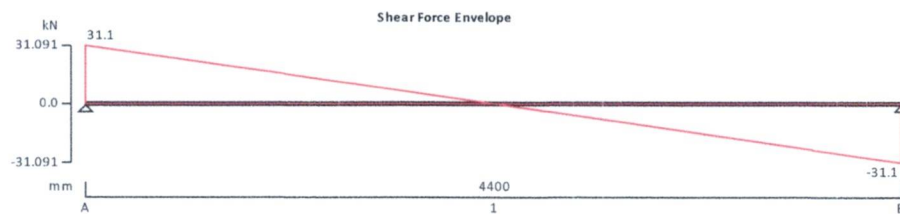
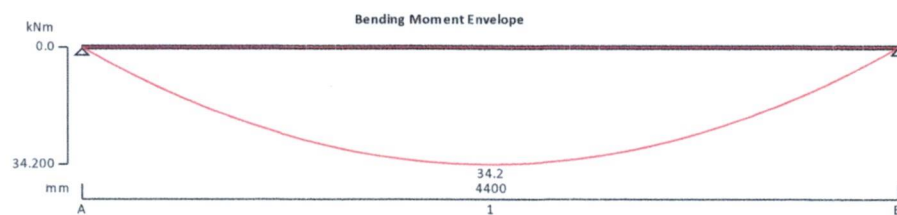
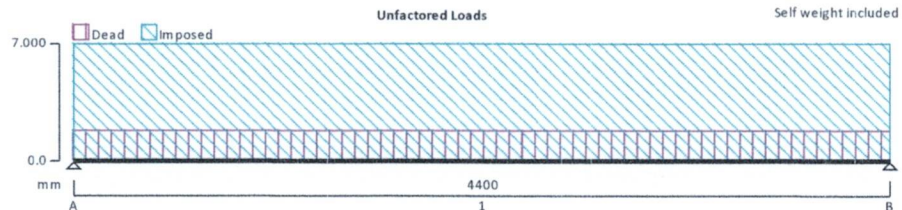
Approved by

Approved date

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 self weight of beam $\times 1$

Dead full UDL 1.8 kN/m

Imposed full UDL 7 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$



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Analysis results

Maximum moment	$M_{\max} = 34.2 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 31.1 \text{ kN}$	$V_{\min} = -31.1 \text{ kN}$
Deflection	$\delta_{\max} = 9.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 31.1 \text{ kN}$	$R_{A_{\min}} = 31.1 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 4.6 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 15.4 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 31.1 \text{ kN}$	$R_{B_{\min}} = 31.1 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 4.6 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 15.4 \text{ kN}$	

Section details

Section type	UC 152x152x30 (British Steel Section Range 2022 (BS4-1))
Steel grade	S355
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 9.4 \text{ mm}$
Design strength	$p_y = 355 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$

Lateral restraint

Span 1 has lateral restraint at supports only

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	$K_{LTA} = 1.00 + 2 \times D$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

$$\epsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 0.88$$

Internal compression parts - Table 11

Depth of section	$d = 123.6 \text{ mm}$	
	$d / t = 21.6 \times \epsilon \leq 80 \times \epsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section	$b = B / 2 = 76.5 \text{ mm}$	
	$b / T = 9.2 \times \epsilon \leq 10 \times \epsilon$	Class 2 compact
		Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force	$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 31.1 \text{ kN}$
	$d / t < 70 \times \epsilon$
	Web does not need to be checked for shear buckling
Shear area	$A_v = t \times D = 1024 \text{ mm}^2$
Design shear resistance	$P_v = 0.6 \times p_y \times A_v = 218.2 \text{ kN}$
	PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment	$M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 34.2 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2	$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 87.9 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling	$L_E = ((1.0 + 1.0) \times L_{s1} + 2 \times D) / 2 = 4558 \text{ mm}$
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Slenderness ratio

$$\lambda = L_E / r_{yy} = 119.084$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.849$$

Torsional index

$$x = 15.999$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.718$$

Ratio - cl.4.3.6.9

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 72.514$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 30.198$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.296$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 384.8 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 426.9 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 213.3 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 25.7 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 34.2 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 25.7 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 34.2 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 34.2 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 52.8 \text{ kNm}$$

$$M_b / m_{LT} = 57.1 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 12.222 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 9.533 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit