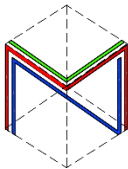
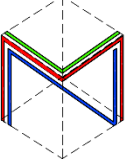
 MVN STRUCTURAL DESIGN	Job Title 11 Banfield Gardens, Henfield		Date Dec 2025
	Item DESIGN DATA	Job No. M1256	Sheet No. 01
	Prepared by MVN	Checked by	

Client		Mr W Ellinger				
Architect		Stukenberg Architectural Design				
Q.S.		N/A				
Others		N/A				
Design Codes, British Standards & References (Current editions & Parts will be used as appropriate at the time of this design)		BS6399: Dead, Imposed and Wind Loading for Buildings. BS449: The Use of Structural Steel in Buildings BS648: Schedule of Weights of Building Materials BS5268: Structural Use of Timber BS5628: Structural Use of Masonry BS5950: Structural Steelwork			BS8004: Code of Practice for Foundations BS8110: Structural Use of Concrete The Building Regulations CDM Regulations 2015 Others:	
Building Structure Description		The existing structure is an end of terrace residential property of prefabricated timber roof (trusses), timber floor and load bearing masonry walls. It is now proposed to convert the loft space by removing the diagonal struts forming the existing roof trusses.				
Fire Resistance		N/A				
Subsoil / Foundation Data		N/A				
CDM		Refer to separate Designers Risk Assessment and Project Design Review.				
Material Data			Works Cube Strength	Aggregate Size	Cement Content	Max. Free Water/ Cement Ratio
1. Concrete	Blinding Concrete					
	R.C. Foundations					
	M.C. Foundations					
	2. Reinforcement		High Tensile Fy = 500 N/mm²			
3. Timber		Grade C24				
4. Steelwork		Grade S355				

 MVN STRUCTURAL DESIGN		Job Title		Date	
		11 Banfield Gardens, Henfield		Dec 2025	
		Item		Job No.	
STANDARD LOADING		M1256		Sheet No.	
02		Prepared by		Checked by	
MVN					

STANDARD LOADINGS (kN/m²)			OUTPUT (kN/m²)		
			DEAD	LIVE	TOTAL
PITCH ROOFS	TILES	0.60	0.85	0.60	1.45
	BATTENS + FELT	0.10			
	RAFTERS	0.15			
ROOF SPACE	JOISTS + INSULATION SOFFIT	0.85 ON PLAN	0.30	0.25	0.55
		0.15			
		0.15			
ROOF + SPACE		0.30	1.15	0.85	2.00
FLAT ROOFS	CHIPPINGS + FELT	0.35	0.80	0.75	1.55
	BOARDS + JOISTS + FIRRINGS	0.30			
	SOFFIT + INSULATION	0.15			
		0.80			
TIMBER FLOORS	BOARDS + JOISTS	0.35	0.50	1.50	2.00
	SOFFIT	0.15			
	PARTITION	0.50			
BEAM & BLOCK FLOORS	SCREED SELF WEIGHT 150 BEAM & BLOCK	1.00	4.30	1.50	5.80
		1.80			
		2.50			
	STUD PARTITION	4.30	5.30	1.50	6.80
		1.00			
WALLS	TIMBER STUD INTERNAL		0.40		0.40
	PLASTERBOARD	0.15			
	STUDS	0.10			
	PLASTERBOARD	0.15	0.60		0.60
	LATH + PLASTER	0.25			
	STUDS	0.10			
	LATH + PLASTER	0.25	0.90		0.90
	TIMBER STUD EXTERNAL	0.60			
	TILES	0.55			
	BATTENS + FELT + INSULATION	0.10	3.80		3.80
	STUDS	0.10			
	PLASTERBOARD	0.15			
	BRICK AND BLOCKWORK	0.90	2.50		2.50
	CAVITY CONSTRUCTION				
	BRICK 102mm	2.00			
BLOCK 100mm	1.50	4.70		4.70	
PLASTER (ONE FACE)	0.30				
SOLID CONSTRUCTION	3.80				
BRICK 102mm	2.00	2.50		2.50	
PLASTER (TWO FACES)	0.50				
SOLID CONSTRUCTION	2.50				
BRICK 215mm	4.20	4.70		4.70	
PLASTER (TWO FACES)	0.50				
	4.70				

 MVN STRUCTURAL DESIGN	Job Title 11 Banfield Gardens, Henfield		Date Dec 2025
	Item GENERAL NOTES	Job No. M1256	Sheet No. 03
	Prepared by MVN	Checked by	

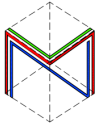
GENERAL NOTES

- 1) All dimensions to be checked on site. All details & dimensions relating to Sub-Contractors or Suppliers work must be checked & agreed between the Sub-Contractor or Supplier & the General Contractor.
- 2) **DO NOT SCALE FROM THE DRAWINGS.**
- 3) This drawing is to be read in conjunction with all relevant Architects and Engineers drawings & specifications.
- 4) The **Contractor** is responsible for ensuring the stability of the structure & the stability of adjacent structures during all stages of the works. Temporary works are to the Contractor's design
- 5) Timber:
All timber wall plates to be minimum 100x65dp. All loose timbers to be fixed with framing anchors or truss clips.

Double-up joists to support partitions & to trim stair voids unless noted otherwise. Double-up rafters & ceiling as necessary to trim roof-lights & dormers.
- 6) Floor Straps:
30 x 5thk. x 1200 long MS galvanised straps once bent at 100mm bent, spaced at 1500mm centres. Straps to be fixed to minimum 3No. joists where their span is parallel to external walls.
- 7) Roof Straps:
30 x 2.5thk. x 1200 long MS galvanised straps once bent at 100mm, spaced at 1800 centres to be provided to strap down wall plates.
30 x 5thk. x 1100 long MS galvanised straps spaced at 1800 centres to be provided at trussed rafter/rafter & at ceiling joist levels to restrain gable walls. Straps to be fixed to at least 3No. trusses/rafters/ceiling joists.
- 8) Steelwork:
All steelwork unless noted otherwise to be Grade S355. All bolts to be Grade 8.8 & all welds to be 6mm fillet welds unless noted otherwise. All steelwork connections to be designed for unfactored Dead + Imposed loads as noted on the drawings. (Minimum reactions of 75kN).

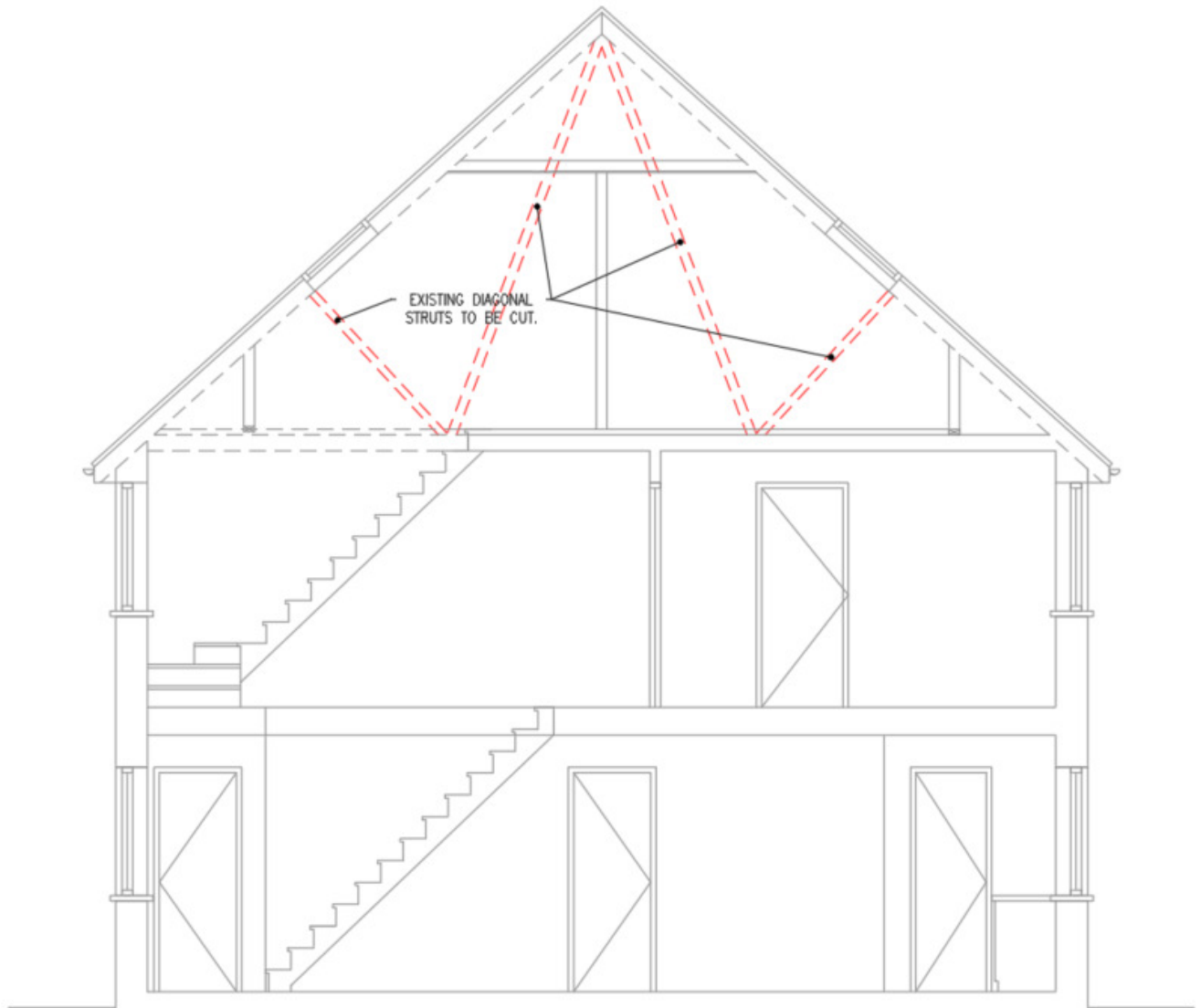
All internal steelwork unless noted otherwise to be blast cleaned to Grade SA 2.5 & receive 2No. coats of zinc phosphate primer to achieve a dry film thickness of 75 microns. Top coat and fire protection information to be provided by the Architect or Sub-Contractor.

All steelwork faces within cavity walls to be painted with 2No. coats of Bitumastic (RIW) paint.
- 9) Lintels:
All lintels detailed on drawing. Provide minimum 150mm bearings each end unless noted otherwise.

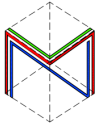


MVN Structural Design Limited

Project				Job no.	
11 Banfield Crescent, Henfield				M1256	
Calcs for				Start page no./Revision	
Loft conversion (of existing truss roof)				4	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MVN	12/12/2025				



BUILDING SECTION



MVN Structural Design Limited

Project

11 Banfield Crescent, Henfield

Job no.

M1256

Calcs for

Loft conversion (of existing truss roof)

Start page no./Revision

5

Calcs by

MVN

Calcs date

12/12/2025

Checked by

Checked date

Approved by

Approved date

CHECKING 33X95 TR26 AT 600MM CENTRES TOP CHORD AS NEW RAFTERS - 3100MM**SPAN - FAILS****TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)**

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

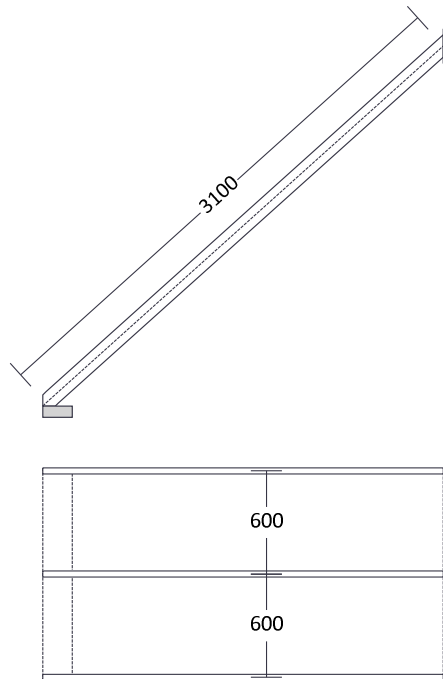
Tedds calculation version 1.0.09

Rafter details

Rafter spacing;

 $S_{\text{Rafter}} = 600 \text{ mm};$

Rafter inclination;

 $\theta_{\text{Rafter}} = 42 \text{ deg}$ **Forces input on Rafter**

Permanent load on slope;

 $F_{G_Rafter} = 0.85 \text{ kN/m}^2$

Imposed load on plan;

 $F_{Q_Rafter} = 0.60 \text{ kN/m}^2$ **Rafter loading details****Distributed loads**

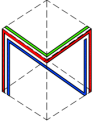
Permanent load on slope;

 $p_G = 0.51 \text{ kN/m}$

Imposed load on slope;

 $p_Q = 0.27 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	13.6	0.7	0.050	PASS
Bearing stress	N/mm ²	1.5	0.5	0.345	PASS
Bending stress	N/mm ²	18.3	18.0	0.984	PASS
Shear stress	N/mm ²	2.4	0.8	0.355	PASS
Bending and axial force				0.986	PASS
Column stability check				1.159	FAIL
Beam stability check				1.015	FAIL
Deflection	mm	12.4	44.0	3.552	FAIL

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 6	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - TR26 (EC5) - 35x97

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.51 kN/m at 0 m to 3.1 m
Member	Imposed	UDL	GlobalZ	0.27 kN/m at 0 m to 3.1 m

;

Tedds calculation version 2.2.23

Member - Span 1

Partial factor for material properties and resistances

Partial factor; $\gamma_M = 1.300$

Member details

Load duration; Long-term; Service class; 2

Timber section details

Number of timber sections; N = 1
 Breadth of sections; b = 35 mm; Depth of sections; h = 97 mm
 Timber strength class; TR26

Section properties

Cross sectional area; A = 3395 mm²
 Section modulus; $W_y = 54886$ mm³; $W_z = 19804$ mm³
 Second moment of area; $I_y = 2661963$ mm⁴; $I_z = 346573$ mm⁴
 Radius of gyration; $r_y = 28.0$ mm; $r_z = 10.1$ mm

Span details

Bearing length; $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check compression parallel to the grain - cl.6.1.4

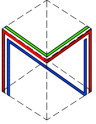
Design compressive stress; $\sigma_{c,0,d} = 0.676$ N/mm²; Design compressive strength; $f_{c,0,d} = 13.564$ N/mm²
 Utilisation = 0.050

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress; $\sigma_{c,y,90,d} = 0.531$ N/mm²; Des.perp.comp.strength; $f_{c,y,90,d} = 1.540$ N/mm²

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 7	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Utilisation = **0.345**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress; $\tau_{y,d} = 0.840 \text{ N/mm}^2$;

Design shear strength; $f_{v,y,d} = 2.369 \text{ N/mm}^2$

Utilisation = **0.355**

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis; $L_{e,y} = 2790 \text{ mm}$;

Relative slenderness ratio; $\lambda_{rel,y} = 1.764$

Effective length for z-axis; $L_{e,z} = 0 \text{ mm}$;

Relative slenderness ratio; $\lambda_{rel,z} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Utilisation = **0.176**

PASS - Column stability is acceptable

Check design 1550 mm along span

Check bending moment - Section 6.1.6

Design bending stress; $\sigma_{m,y,d} = 17.996 \text{ N/mm}^2$;

Design bending strength; $f_{m,y,d} = 18.289 \text{ N/mm}^2$

Utilisation = **0.984**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = **0.986**

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis; $L_{e,y} = 2790 \text{ mm}$;

Relative slenderness ratio; $\lambda_{rel,y} = 1.764$

Effective length for z-axis; $L_{e,z} = 0 \text{ mm}$;

Relative slenderness ratio; $\lambda_{rel,z} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Utilisation = **1.159**

FAIL - Column stability is not acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = **1.015**

FAIL - Beam stability is not acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1550 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep; $\delta_{y,Final} = 44 \text{ mm}$;

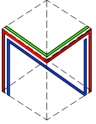
Allowable deflection;

$\delta_{y,Allowable} = 12.4 \text{ mm}$

Utilisation = **3.552**

FAIL - Final deflection exceeds allowable deflection

;

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 8	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

LOADING ON NEW RAFTERS (R1)

Span = 2.3m (plan) or 3.1m (on slope) @ 600mm centres (to match centres of existing trusses)

	Dead	Imposed
New Pitched Roof: (Slope = 42deg)		
Dead =	0.85kN/m ²	
Imposed =		0.60kN/m ²

NEW 47X150C24 RAFTERS AT 600MM CENTRES - 3100MM SPAN ON SLOPE

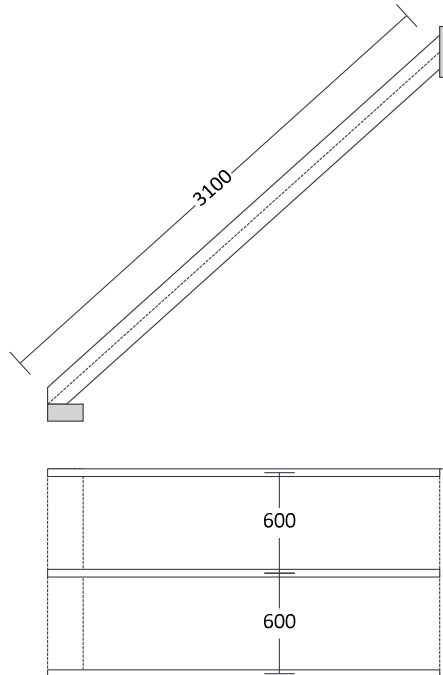
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Rafter details

Rafter spacing; $S_{\text{Rafter}} = 600 \text{ mm};$ Rafter inclination; $\theta_{\text{Rafter}} = 42 \text{ deg}$



Forces input on Rafter

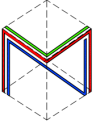
Permanent load on slope; $F_{G_Rafter} = 0.85 \text{ kN/m}^2$
Imposed load on plan; $F_{Q_Rafter} = 0.60 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope; $p_G = 0.51 \text{ kN/m}$
Imposed load on slope; $p_Q = 0.27 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	12.4	0.3	0.027	PASS
Bearing stress	N/mm ²	1.5	0.4	0.271	PASS
Bending stress	N/mm ²	14.2	5.7	0.400	PASS

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 9	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Shear stress	N/mm ²	2.4	0.4	0.173	PASS
Bending and axial force				0.401	PASS
Column stability check				0.443	PASS
Beam stability check				0.185	PASS
Deflection	mm	12.4	9.2	0.741	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - C24 (EC5) - 47x150

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.51 kN/m at 0 m to 3.1 m
Member	Imposed	UDL	GlobalZ	0.27 kN/m at 0 m to 3.1 m

;

Tedds calculation version 2.2.23

Member - Span 1

Partial factor for material properties and resistances

Partial factor; $\gamma_M = 1.300$

Member details

Load duration; Long-term; Service class; 2

Timber section details

Number of timber sections; N = 1
 Breadth of sections; b = 47 mm; Depth of sections; h = 150 mm
 Timber strength class; C24

Section properties

Cross sectional area; A = 7050 mm²
 Section modulus; $W_y = 176250$ mm³; $W_z = 55225$ mm³
 Second moment of area; $I_y = 13218750$ mm⁴; $I_z = 1297787$ mm⁴
 Radius of gyration; $r_y = 43.3$ mm; $r_z = 13.6$ mm

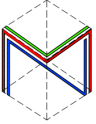
Span details

Bearing length; $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check compression parallel to the grain - cl.6.1.4

Design compressive stress; $\sigma_{c,0,d} = 0.330$ N/mm²; Design compressive strength; $f_{c,0,d} = 12.438$ N/mm²
 Utilisation = 0.027

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 10	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress; $\sigma_{c,y,90,d} = 0.401 \text{ N/mm}^2$; Des.perp.comp.strength; $f_{c,y,90,d} = 1.481 \text{ N/mm}^2$
Utilisation = 0.271

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress; $\tau_{y,d} = 0.411 \text{ N/mm}^2$; Design shear strength; $f_{v,y,d} = 2.369 \text{ N/mm}^2$
Utilisation = 0.173

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis; $L_{e,y} = 2790 \text{ mm}$; Relative slenderness ratio; $\lambda_{rel,y} = 1.093$
Effective length for z-axis; $L_{e,z} = 0 \text{ mm}$; Relative slenderness ratio; $\lambda_{rel,z} = 0$
 $\lambda_{rel,y} > 0.3$ column stability check is required
Utilisation = 0.043

PASS - Column stability is acceptable

Check design 1550 mm along span

Check bending moment - Section 6.1.6

Design bending stress; $\sigma_{m,y,d} = 5.685 \text{ N/mm}^2$; Design bending strength; $f_{m,y,d} = 14.215 \text{ N/mm}^2$
Utilisation = 0.4

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = 0.401

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis; $L_{e,y} = 2790 \text{ mm}$; Relative slenderness ratio; $\lambda_{rel,y} = 1.093$
Effective length for z-axis; $L_{e,z} = 0 \text{ mm}$; Relative slenderness ratio; $\lambda_{rel,z} = 0$
 $\lambda_{rel,y} > 0.3$ column stability check is required
Utilisation = 0.443

PASS - Column stability is acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = 0.185

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

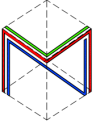
Check design 1550 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep; $\delta_{y,Final} = 9.2 \text{ mm}$; Allowable deflection; $\delta_{y,Allowable} = 12.4 \text{ mm}$
Utilisation = 0.741

PASS - Allowable deflection exceeds final deflection

;

 MVN Structural Design Limited	Project		Job no.	
	11 Banfield Crescent, Henfield		M1256	
	Calcs for		Start page no./Revision	
	Loft conversion (of existing truss roof)		11	
	Calcs by	Calcs date	Checked by	Checked date
	MVN	12/12/2025		
			Approved by	Approved date

LOADING ON NEW DOUBLE RAFTERS (DR)

Span = 3.1m (on slope)

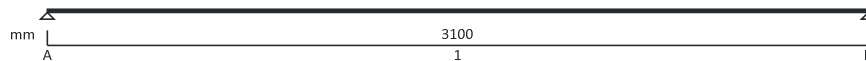
	Dead	Imposed
New Pitched Roof: (Slope = 42deg)		
Dead = $(0.85/\cos 42 + 0.3) \times (0.74 + 0.6)/2 = 1.0\text{kN/m}$		
Imposed = $(0.6 + 0.25) \times (0.74 + 0.6)/2 =$		0.56kN/m

NEW 2NO. 47X150C24 DOUBLE RAFTERS (DR)

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 1.7.05



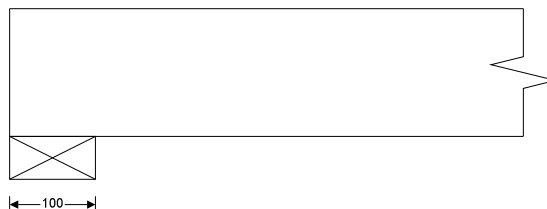
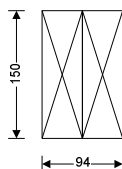
Applied loading

Beam loads

	Permanent self weight of beam $\times 1$
new pitched roof	Permanent full UDL 1.000 kN/m
new pitched roof	Variable full UDL 0.600 kN/m

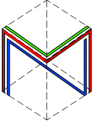
Analysis results

Design moment;	$M = 2.781 \text{ kNm};$	Design shear;	$F = 3.589 \text{ kN}$
Total load on member;	$W_{\text{tot}} = 7.178 \text{ kN}$		
Reactions at support A;	$R_{A_{\text{max}}} = 3.589 \text{ kN};$	$R_{A_{\text{min}}} = 3.589 \text{ kN}$	
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 1.625 \text{ kN}$		
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 0.930 \text{ kN}$		
Reactions at support B;	$R_{B_{\text{max}}} = 3.589 \text{ kN};$	$R_{B_{\text{min}}} = 3.589 \text{ kN}$	
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 1.625 \text{ kN}$		
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 0.930 \text{ kN}$		



Timber section details

Breadth of section;	$b = 47 \text{ mm};$	Depth of section;	$h = 150 \text{ mm}$
Number of sections;	$N = 2;$	Breadth of member;	$b_b = 94 \text{ mm}$
Timber strength class;	C24		

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Member details

Service class of timber; 1; Load duration; Long-term

Length of span; L_{s1} = 3100 mm

Length of bearing; L_b = 100 mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress; σ_{c,90,d} = 0.382 N/mm²; Design compressive strength; f_{c,90,d} = 1.346 N/mm²

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress; σ_{m,d} = 7.890 N/mm²; Design bending strength; f_{m,d} = 12.923 N/mm²

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress; τ_d = 0.570 N/mm²; Permissible shear stress; f_{v,d} = 2.154 N/mm²

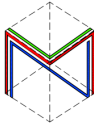
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; δ_{lim} = 10.230 mm; Total final deflection; δ_{fin} = 10.217 mm

PASS - Total final deflection is less than the deflection limit

;



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LOADING ON NEW ROOF BEAM (RB1)

Span = 4.6m

Dead Imposed

New Pitched Roof: (Slope = 42deg)

Dead = $(0.85/\cos 42 + 0.3) \times 4.5/2 = 3.3\text{kN/m}$

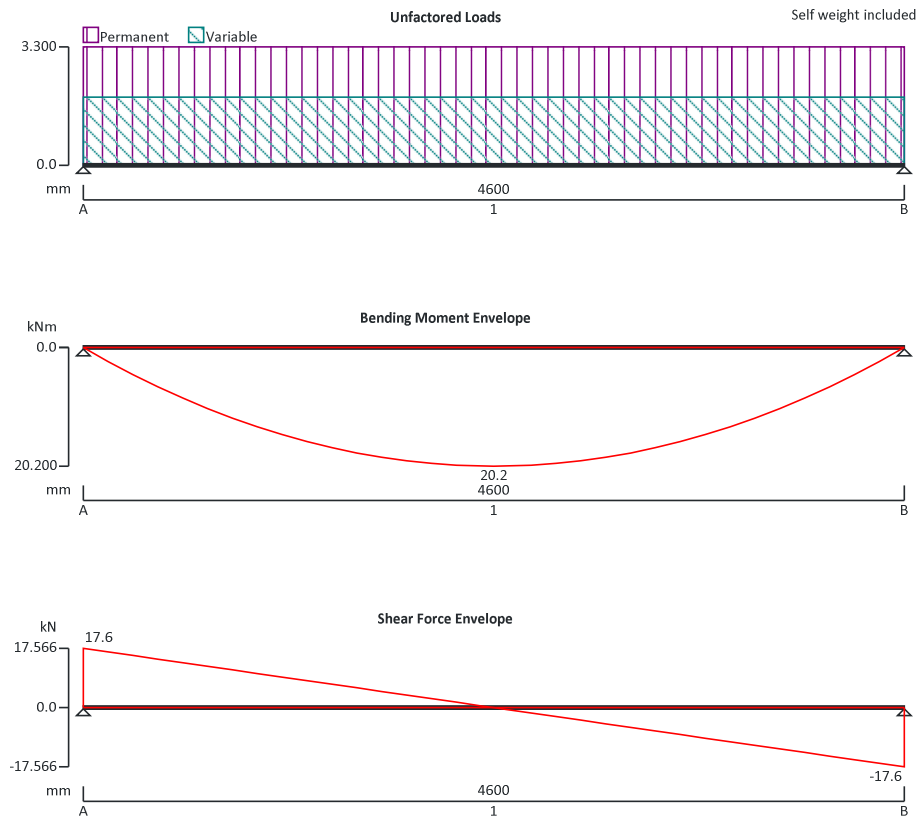
Imposed = $(0.6 + 0.25) \times 4.5/2 = 1.9\text{kN/m}$

NEW 203X133UB25 ROOF BEAM (RB1)

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

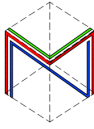
Applied loading

Beam loads

Permanent self weight of beam $\times 1$

new pitched roof - Permanent full UDL 3.3 kN/m

new pitched roof - Variable full UDL 1.9 kN/m



MVN Structural Design Limited

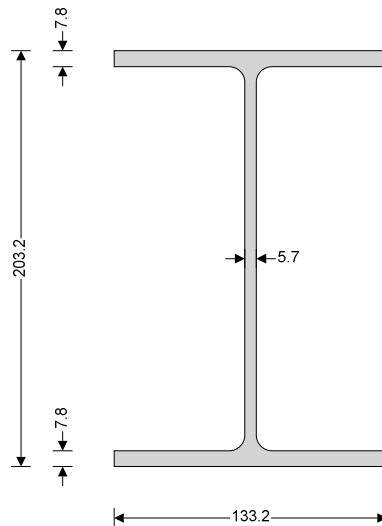
Project 11 Banfield Crescent, Henfield				Job no. M1256	
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Analysis results

Maximum moment;	$M_{max} = 20.2 \text{ kNm};$	$M_{min} = 0 \text{ kNm}$
Maximum shear;	$V_{max} = 17.6 \text{ kN};$	$V_{min} = -17.6 \text{ kN}$
Deflection;	$\delta_{max} = 6.5 \text{ mm};$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_max} = 17.6 \text{ kN};$	$R_{A_min} = 17.6 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_Permanent} = 8.2 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_Variable} = 4.4 \text{ kN}$	
Maximum reaction at support B;	$R_{B_max} = 17.6 \text{ kN};$	$R_{B_min} = 17.6 \text{ kN}$
Unfactored permanent load reaction at support B;	$R_{B_Permanent} = 8.2 \text{ kN}$	
Unfactored variable load reaction at support B;	$R_{B_Variable} = 4.4 \text{ kN}$	

Section details

Section type; **UB 203x133x25 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355**



Section classification; **Class 1**

Check shear - Section 6.2.6

Design shear force; $V_{Ed} = 18 \text{ kN};$ Design shear resistance; $V_{c,Rd} = 262.7 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{Ed} = 20.2 \text{ kNm};$ Des.bending resist.moment; $M_{c,Rd} = 91.5 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT} = 1.251;$ Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.400$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

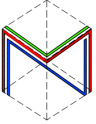
Des.buckling resist.moment; $M_{b,Rd} = 51.3 \text{ kNm}$
PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = 18.4 \text{ mm};$ Maximum deflection; $\delta = 6.461 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit

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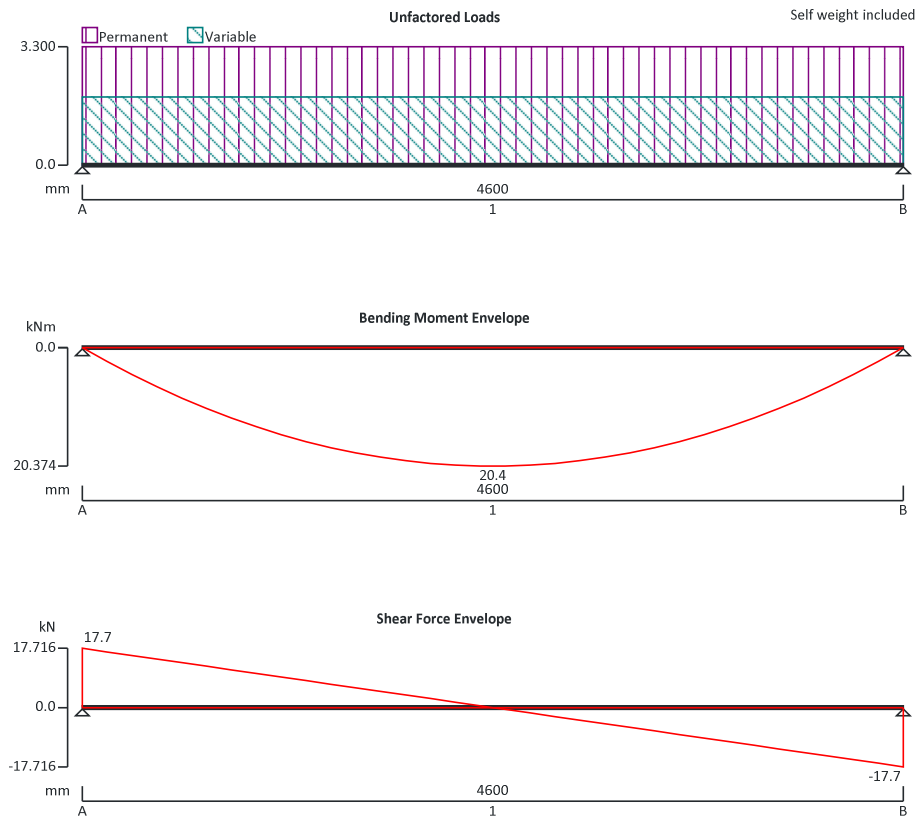
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ALTERNATIVE NEW 152X152UC30 ROOF BEAM (RB1) – ADOPT

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

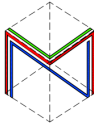
Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1 new pitched roof - Permanent full UDL 3.3 kN/m new pitched roof - Variable full UDL 1.9 kN/m
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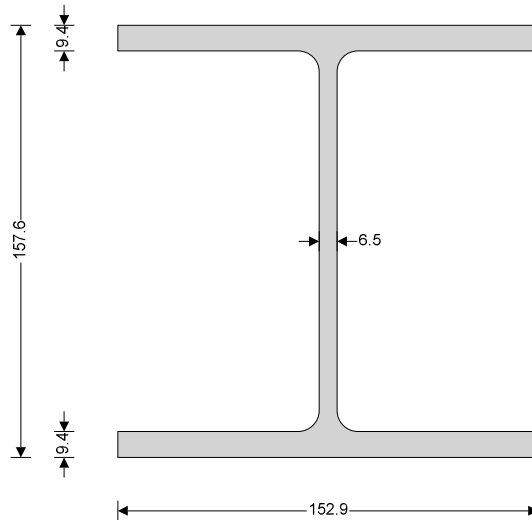
Analysis results

Maximum moment;	$M_{max} = 20.4 \text{ kNm};$	$M_{min} = 0 \text{ kNm}$
Maximum shear;	$V_{max} = 17.7 \text{ kN};$	$V_{min} = -17.7 \text{ kN}$
Deflection;	$\delta_{max} = 8.7 \text{ mm};$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{max}} = 17.7 \text{ kN};$	$R_{A_{min}} = 17.7 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_{Permanent}} = 8.3 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_{Variable}} = 4.4 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{max}} = 17.7 \text{ kN};$	$R_{B_{min}} = 17.7 \text{ kN}$



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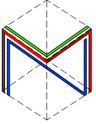
Project 11 Banfield Crescent, Henfield				Job no. M1256	
Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 16	
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Unfactored permanent load reaction at support B; $R_{B_Permanent} = 8.3$ kNUnfactored variable load reaction at support B; $R_{B_Variable} = 4.4$ kN**Section details**Section type; **UC 152x152x30 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355**Section classification; **Class 1****Check shear - Section 6.2.6**Design shear force; $V_{Ed} = 18$ kN;Design shear resistance; $V_{c,Rd} = 236.9$ kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment; $M_{Ed} = 20.4$ kNm;Des.bending resist.moment; $M_{c,Rd} = 87.9$ kNm**Slenderness ratio for lateral torsional buckling**LTB slenderness ratio; $\bar{\lambda}_{LT} = 0.898$;Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.400$ **$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored****Design resistance for buckling - Section 6.3.2.1**Des.buckling resist.moment; $M_{b,Rd} = 69$ kNm**PASS - Design buckling resistance moment exceeds design bending moment****Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = 18.4$ mm;Maximum deflection; $\delta = 8.726$ mm**PASS - Maximum deflection does not exceed deflection limit**

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EBP (ENG BRICK PADSTONE) TO SUPPORT RB1 REACTION

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	17.8 kN	17.5 kN	0.80 N/mm ²	1.75 N/mm ²	0.461	Pass

Masonry panel details

Panel length; L = **3200** mm; Panel height; h = **2400** mm
Load bearing leaf thickness; t = **100** mm; Effective height; h_{ef} = **1536** mm
Effective thickness; t_{ef} = **126** mm

Masonry material details

Unit type; **Aggregate concrete - Group 1**
Mean masonry strength; f_b = **5.0** N/mm²; Specific weight of units; γ = **18** kN/m³
Mortar type; **M4 - General Purpose**; Mortar strength; f_m = **4.0** N/mm²
Characteristic strength; f_k = **3.49** N/mm²

Design compressive strength of masonry for inner (loaded) leaf

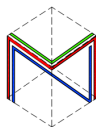
Category of manufacturing; **Category II**; Class of execution control; **Class 2**
Partial factor for material; γ_M = **3.00**; Design strength of masonry; f_d = **1.16** N/mm²

Partial safety factors for design loads

Permanent partial factor; γ_{FG} = **1.35**; Variable partial factor; γ_{FQ} = **1.50**

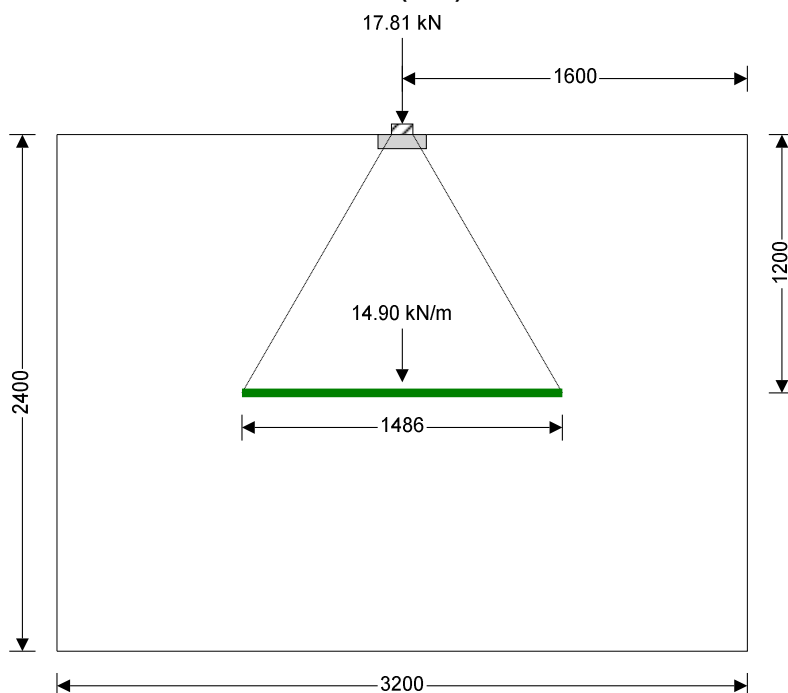
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; λ_{lim} = **27**; Slenderness ratio; λ = **12.2**
PASS - Slenderness ratio is less than slenderness limit



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Concentrated Load 1 details - Reaction from roof beam (RB1)

Permanent load;	$G_{kc1} = 8.30$ kN;	Variable load;	$Q_{kc1} = 4.40$ kN
Eccentricity of load;	$e_{c1} = 0$ mm;	Length of load;	$L_{c1} = 100$ mm
Width of load;	$w_{c1} = 100$ mm;	Height of load;	$h_{c1} = 2400$ mm
Distance to right edge;	$r_{11} = 1550$ mm;	Distance to nearest edge;	$a_{11} = 1550$ mm

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load;	$N_{Edc1} = 17.81$ kN;	Design resistance;	$N_{Rdc1} = 17.46$ kN
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Applied concentrated load exceeds design resistance, spreader required!**Design of spreader beam**

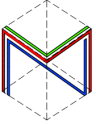
Type of spreader;	Concrete padstone;	Type of load;	Point load
Length of spreader;	$L_{sp1} = 225$ mm;	Width of spreader;	$w_{sp1} = 100$ mm
Height of spreader;	$h_{sp1} = 65$ mm;	Eccentricity of load;	$e_{sp1} = 0$ mm
Modulus of elasticity;	$E_{sp1} = 29962$ N/mm ² ;	Maximum moment;	$M_{Edsp1} = 0.50$ kNm
Maximum shear;	$V_{Edsp1} = 8.89$ kN;	Allowable stress;	$\sigma_{Rdsp1} = 1.75$ N/mm ²
Design stress;	$\sigma_{Edsp1} = 0.80$ N/mm ²		

PASS - Design stress under spreader is less than the allowable bearing stress**Walls subjected to mainly vertical loading - Section 6.1.2**

Vertical load at mid-height;	$N_{Ed1} = 14.90$ kN/m;	Design resistance;	$N_{Rd1} = 93.33$ kN/m
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PASS - Design value of vertical resistance exceeds applied vertical load

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SPLICE CONNECTION DESIGN TO NEW ROOF BEAM (RB1)

Provide splice at third point of span = $4.6/3 = 1.5\text{m}$

Ultimate Design Forces at splice:

Moment = 17.4kN,

Shear = 6kN

152X152UC30 ROOF BEAM SPLICE CONNECTION (COVER PLATE) AT 1500MM FROM END

STEEL BEAM SPLICE DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

Tedds calculation version 1.0.02

Design summary

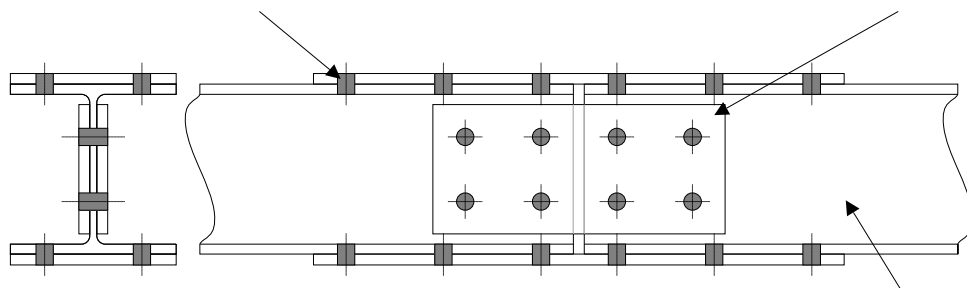
Overall design status; PASS

Overall design utilisation; 0.455

Description	Unit	Design	Resistance	Utilisation	Result
Flange bolt group	kN	27.4	60.3	0.455	PASS
Flange bolt slip resist. SLS	kN	17.2	40.0	0.429	PASS
Flange plate compression	kN	164.7	417.2	0.395	PASS
Flange plate tension flange	kN	107.4	392.1	0.274	PASS
Flange plate block tearing	kN	107.4	691.3	0.155	PASS
Flange compression	kN	164.7	510.2	0.323	PASS
Flange tension	kN	107.4	422.6	0.254	PASS
Web bolt group	kN	16.3	54.3	0.301	PASS
Web bolt slip resist. SLS	kN	10.9	79.9	0.136	PASS
Web plate shear	kN	6.0	300.0	0.020	PASS
Web plate block tearing	kN	6.0	356.6	0.017	PASS
Web plate combined				0.197	PASS
Beam web net shear	kN	6.0	227.4	0.026	PASS

490 x 152.9 x 10
12No. M16 bolts

120 x 270 x 10
8No. M16 bolts



Cover plate Splice

Beam sections:
UC 152x152x30

Design forces

Design moment (ULS);

$M_{Ed} = 17.4 \text{ kNm};$

Design moment (SLS);

$M_{Ed,ser} = 12.0 \text{ kNm}$

Design shear force (ULS);

$V_{Ed} = 6.0 \text{ kN};$

Design shear force (SLS);

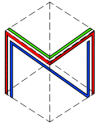
$V_{Ed,ser} = 4.0 \text{ kN}$

Design axial force (ULS);

$N_{Ed} = 75.0 \text{ kN};$

Design axial force (SLS);

$N_{Ed,ser} = 75.0 \text{ kN}$



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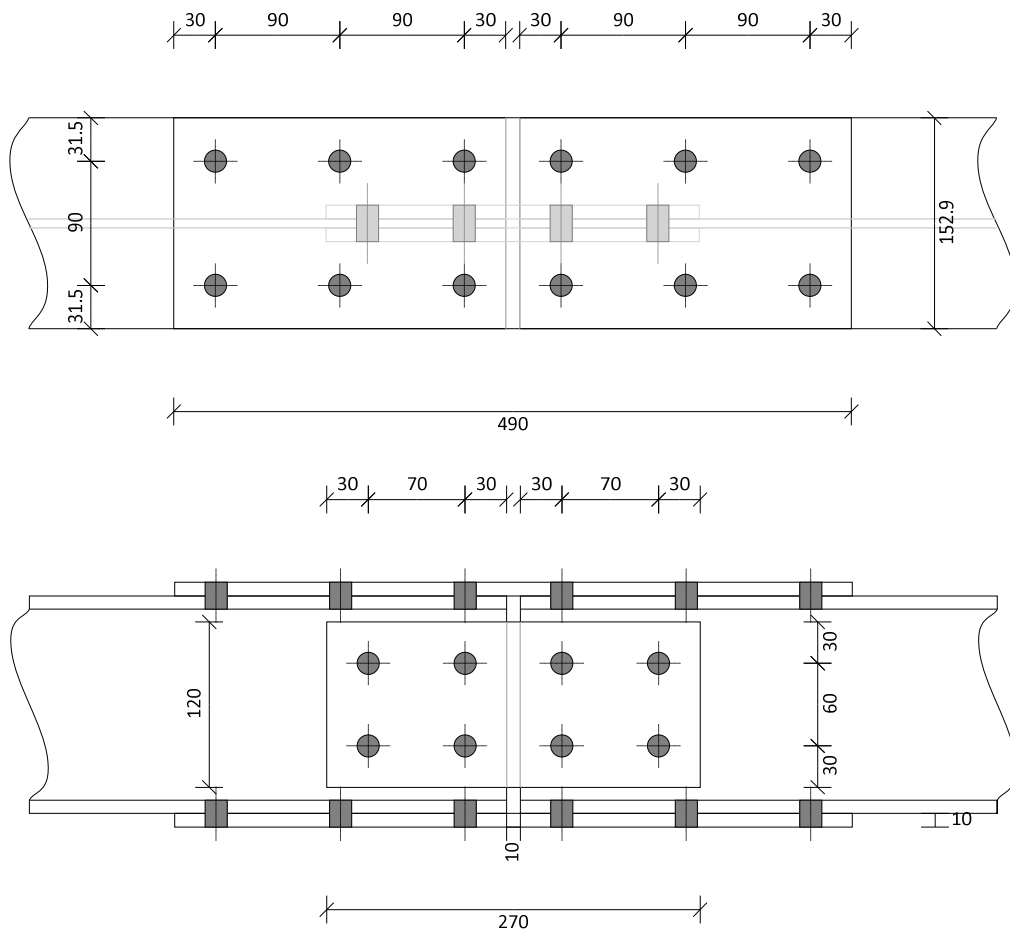
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Partial factorsResistance of cross section; $\gamma_{M0} = 1.00$;Sections in tension to fracture; $\gamma_{M2} = 1.10$;Slip resistance; $\gamma_{M3} = 1.25$;Structural integrity; $\gamma_{M,u} = 1.10$ Resist. members to instability; $\gamma_{M1} = 1.00$ Resistance of bolts; $\gamma_{M2,b} = 1.25$ Slip resistance, servability; $\gamma_{M3,ser} = 1.10$ **Beam details**

Section name; UC 152x152x30;

Yield strength; $f_{y,bm} = 355 \text{ N/mm}^2$;Section height; $h = 157.6 \text{ mm}$;Flange thickness; $t_f = 9.4 \text{ mm}$;Gap between beams; $g_v = 10.0 \text{ mm}$

Steel grade; S355

Ultimate strength; $f_{u,bm} = 470 \text{ N/mm}^2$ Section width; $b = 152.9 \text{ mm}$ Web thickness; $t_w = 6.5 \text{ mm}$ **Flange bolt details**

Number of bolts

 $n_{1,fp} = 3$

Bolt size

M16;

Yield strength

 $f_{yb,fp} = 640 \text{ N/mm}^2$;

End distance in plate

 $e_{1,fp} = 30 \text{ mm}$;

Edge distance in plate

 $e_{2,fp} = 31.5 \text{ mm}$

Bolt pitch

 $p_{1,fp} = 90 \text{ mm}$;

Bolt grade

8.8

Ultimate strength

 $f_{ub,fp} = 800 \text{ N/mm}^2$

End distance in beam

 $e_{1,cf} = 30 \text{ mm}$

Bolt gauge

 $p_{2,fp} = 90 \text{ mm}$ **Flange plate details**

External plate width

 $b_{fp,ext} = 152.9 \text{ mm}$

Plate length

 $l_{fp} = 490 \text{ mm}$;

Plate thickness

 $t_{fp} = 10 \text{ mm}$

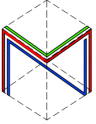
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Plate grade **S275**; Yield strength $f_{y,fp} = 275 \text{ N/mm}^2$
Ultimate strength $f_{u,fp} = 410 \text{ N/mm}^2$

Web bolt details

Number of bolt columns	$n_{1,wp} = 2$;	Number of bolt rows	$n_{2,wp} = 2$
Bolt size	M16 ;	Bolt grade	8.8
Yield strength	$f_{yb,wp} = 640 \text{ N/mm}^2$;	Ultimate strength	$f_{ub,wp} = 800 \text{ N/mm}^2$
End distance in plate	$e_{1,wp} = 30 \text{ mm}$;	End distance in beam	$e_{1,cw} = 30 \text{ mm}$
Edge distance in plate	$e_{2,wp} = 30 \text{ mm}$		
Bolt gauge	$p_{1,wp} = 70 \text{ mm}$;	Bolt pitch	$p_{2,wp} = 60 \text{ mm}$

Web plate details

Plate width	$b_{wp} = 270 \text{ mm}$;	Plate height	$h_{wp} = 120 \text{ mm}$
Plate thickness	$t_{wp} = 10 \text{ mm}$;	Number of web plates;	$N_{wp} = 2$
Plate grade	S275		
Yield strength	$f_{y,wp} = 275 \text{ N/mm}^2$;	Ultimate strength	$f_{u,wp} = 410 \text{ N/mm}^2$

Continuity - EN1993-1-8 cl.6.2.7.1(13)

Design bending resist. mnt; $M_{c,y,Rd} = 87.9 \text{ kNm}$
 $0.25 \times M_{c,y,Rd} / M_{Ed} = 1.263$

For continuity M_{Ed} should be greater than or equal to $0.25 \times M_{c,y,Rd}$ so in the following calculations $0.25 \times M_{c,y,Rd}$ will be taken for the design moment

Internal forces - ULS

Total force in tension flange; $F_{tf} = 107.4 \text{ kN}$; Total force in comp. flange; $F_{cf} = 164.7 \text{ kN}$

Flange bolt resistance

Max design force on bolts; $F_{t,v,Ed} = 27.4 \text{ kN}$; Design resistance of bolt; $F_{t,Rd} = 60.3 \text{ kN}$
 $F_{t,v,Ed} / F_{t,Rd} = 0.455$

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Total force in tension flange; $F_{tf,ser} = 45.6 \text{ kN}$; Total force in comp. flange; $F_{cf,ser} = 102.9 \text{ kN}$

Flange bolt resistance SLS

Max design force flange bolts; $F_{t,v,Ed,ser} = 17.2 \text{ kN}$; Slip resistance of single bolt; $F_{s,fp,Rd,ser} = 40.0 \text{ kN}$
 $F_{t,v,Ed,ser} / F_{s,fp,Rd,ser} = 0.429$

PASS - Bolt resistance exceeds applied force on bolt

Flange cover plates - Resistance of plates in compression

Design comp. resistance; $N_{b,fp,Rd} = 417.15 \text{ kN}$
Utilisation; $F_{cf} / N_{b,fp,Rd} = 0.395$

PASS - Flange cover plate compressive resistance greater than design compressive force

Flange cover plates - Resistance of plates in tension

Design tension resistance; $N_{t,fp,Rd} = 392.15 \text{ kN}$
Utilisation; $F_{tf} / N_{t,fp,Rd} = 0.274$

PASS - Flange cover plate tensile resistance greater than design tensile force

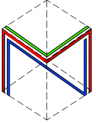
Flange cover plates - block tearing

Block tearing resistance; $N_{bt,Rd} = 691.3 \text{ kN}$
Utilisation; $F_{tf} / N_{bt,Rd} = 0.155$

PASS - Flange cover plate block tearing resistance greater than design tensile force

Beam flange - resistance in compression

Design compressive resist.; $N_{b,Rd} = 510.23 \text{ kN}$

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Utilisation;

$$F_{cf} / N_{b,Rd} = 0.323$$

PASS - Beam flange compressive resistance greater than design compressive force

Beam flange - resistance in tension

Design tension resistance; $N_{t,Rd} = 422.56$ kN

Utilisation; $F_{tf} / N_{t,Rd} = 0.254$

PASS - Beam flange tensile resistance greater than design tensile force

Internal forces - ULS

Force in web due to axial force; $F_{w,N} = 17.7$ kN;

Force in web due to vert. shear; $F_{w,V} = 6.0$ kN

Web bolt resistance ULS

Result. force on extreme bolt; $F_{w,v,Ed} = 16.3$ kN;

Design resistance of bolt; $F_{w,Rd} = 54.3$ kN

$$F_{w,v,Ed} / F_{w,Rd} = 0.301$$

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Force in web due to axial force; $F_{w,N,ser} = 17.7$ kN;

Force in web due to vert. shear; $F_{w,V,ser} = 4.0$ kN

Web bolt resistance SLS

Result. force on extreme bolt; $F_{w,v,Ed,ser} = 10.9$ kN;

Slip resistance of single bolt; $F_{s,wp,Rd,ser} = 79.9$ kN

$$F_{w,v,Ed,ser} / F_{s,wp,Rd,ser} = 0.136$$

PASS - Bolt resistance exceeds applied force on bolt

Resistance of web cover plate in shear

Plastic resist. web cover plate; $V_{pl,wp,Rd} = 300.04$ kN

$$V_{Ed} / V_{pl,wp,Rd} = 0.020$$

PASS - Shear resistance of web plate exceeds applied shear

Block tearing of web cover plate

Block tearing resistance; $V_{b,Rd} = 356.60$ kN

$$V_{Ed} / V_{b,Rd} = 0.017$$

PASS - Block tearing resistance of web plate exceeds applied shear

Resistance of beam web shear accounting for holes

Design shear net resistance; $V_{nw,Rd} = 227.4$ kN

$$V_{Ed} / V_{nw,Rd} = 0.026$$

PASS - Net shear resistance of beam web exceeds applied shear force

Resistance of web cover plate to combined bending, shear and axial

Bend. resist. web cover plate; $M_{c,wp,Rd} = 13.2$ kNm;

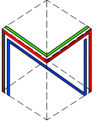
Axial resist. of web cover plate; $N_{wp,Rd} = 660.0$ kN

Applied moment; $M_{wp,Ed} = 2.2$ kNm;

Applied axial force; $N_{wp,Ed} = 17.7$ kN

Interaction formula; $N_{wp,Ed} / N_{wp,Rd} + M_{wp,Ed} / M_{c,wp,Rd} = 0.197$

PASS - Combined bending, shear and axial check satisfied

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 23	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

CHECKING EXISTING 35X122TR26 AT 600MM CENTRES BOTTOM CHORD TO SUPPORT NEW FLOOR JOISTS - FAILS

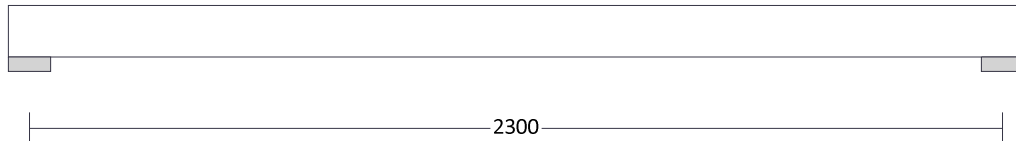
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Joist details

Joist spacing; $s_{Joist} = 600 \text{ mm}$



Forces input on Joist

Vertical permanent load on joist;

$F_{G_Joist} = 0.75 \text{ kN/m}^2$

Vertical imposed load on joist;

$F_{Q_Joist} = 2.50 \text{ kN/m}^2$

Joist loading details

Distributed loads

Vertical permanent load on joist;

$p_G = 0.45 \text{ kN/m}$

Vertical imposed load on joist;

$p_Q = 1.50 \text{ kN/m}$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.5	0.7	0.472	PASS
Bending stress	N/mm ²	17.5	21.9	1.255	FAIL
Shear stress	N/mm ²	2.4	1.7	0.733	PASS
Beam stability check				1.255	FAIL
Deflection	mm	9.2	23.1	2.509	FAIL

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - TR26 (EC5) - 35x122

Loading

Self weight included (Permanent x 1)

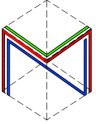
Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.45 kN/m at 0 m to 2.3 m
Member	Imposed	UDL	GlobalZ	1.5 kN/m at 0 m to 2.3 m

;

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 24	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Tedds calculation version 2.2.23

Member - Span 1

Partial factor for material properties and resistances

Partial factor; $\gamma_M = 1.300$

Member details

Load duration; Long-term; Service class; 2

Timber section details

Number of timber sections; $N = 1$
Breadth of sections; $b = 35$ mm; Depth of sections; $h = 122$ mm
Timber strength class; **TR26**

Section properties

Cross sectional area; $A = 4270$ mm²
Section modulus; $W_y = 86823$ mm³; $W_z = 24908$ mm³
Second moment of area; $I_y = 5296223$ mm⁴; $I_z = 435896$ mm⁴
Radius of gyration; $r_y = 35.2$ mm; $r_z = 10.1$ mm

Span details

Bearing length; $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress; $\sigma_{c,y,90,d} = 0.728$ N/mm²; Des.perp.comp.strength; $f_{c,y,90,d} = 1.540$ N/mm²
Utilisation = **0.472**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress; $\tau_{v,d} = 1.736$ N/mm²; Design shear strength; $f_{v,y,d} = 2.369$ N/mm²
Utilisation = **0.733**

PASS - Design shear strength exceeds design shear stress

Check design 1150 mm along span

Check bending moment - Section 6.1.6

Design bending stress; $\sigma_{m,y,d} = 21.922$ N/mm²; Design bending strength; $f_{m,y,d} = 17.469$ N/mm²
Utilisation = **1.255**

FAIL - Design bending stress exceeds design bending strength

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = **1.255**

FAIL - Beam stability is not acceptable

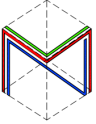
Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1150 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep; $\delta_{y,Final} = 23.1$ mm; Allowable deflection; $\delta_{y,Allowable} = 9.2$ mm
Utilisation = **2.509**

FAIL - Final deflection exceeds allowable deflection

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 25	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

;

LOADING ON NEW FLOOR JOISTS (FJ)

Span = 2.3m @ 600mm centres (to match centres of existing trusses)

	Dead	Imposed
Dead =	0.75kN/m ²	
Imposed = (1.5 + 1.0)		2.50kN/m ²

NEW 47X150C24 FLOOR JOISTS AT 600MM CENTRES

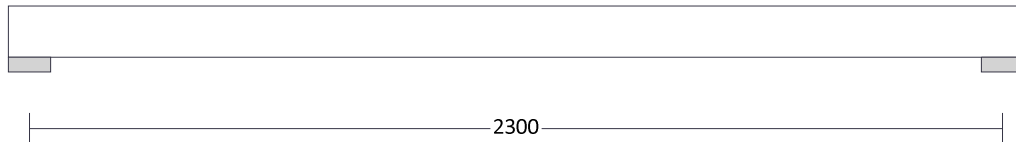
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Joist details

Joist spacing; $S_{Joist} = 600$ mm



Forces input on Joist

Vertical permanent load on joist;	$F_{G_Joist} = 0.75$ kN/m ²
Vertical imposed load on joist;	$F_{Q_Joist} = 2.50$ kN/m ²

Joist loading details

Distributed loads

Vertical permanent load on joist;	$p_G = 0.45$ kN/m
Vertical imposed load on joist;	$p_Q = 1.50$ kN/m

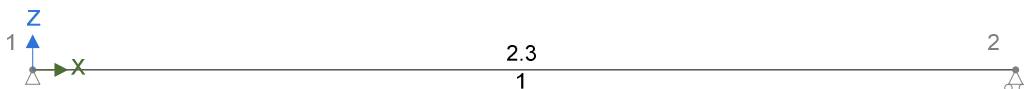
Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.5	0.7	0.472	PASS
Bending stress	N/mm ²	17.5	21.9	1.255	FAIL
Shear stress	N/mm ²	2.4	1.7	0.733	PASS
Beam stability check				1.255	FAIL
Deflection	mm	9.2	23.1	2.509	FAIL

ANALYSIS

Tedds calculation version 1.0.38

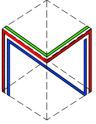
Geometry

Geometry (m) - TR26 (EC5) - 35x122



Loading

Self weight included (Permanent x 1)

 MVN Structural Design Limited	Project		Job no.	
	11 Banfield Crescent, Henfield		M1256	
	Calcs for		Start page no./Revision	
	Loft conversion (of existing truss roof)		26	
	Calcs by	Calcs date	Checked by	Checked date
	MVN	12/12/2025		
			Approved by	Approved date

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.45 kN/m at 0 m to 2.3 m
Member	Imposed	UDL	GlobalZ	1.5 kN/m at 0 m to 2.3 m

;

Tedds calculation version 2.2.23

Member - Span 1

Partial factor for material properties and resistances

Partial factor; $\gamma_M = 1.300$

Member details

Load duration; Long-term; Service class; 2

Timber section details

Number of timber sections; $N = 1$
 Breadth of sections; $b = 35$ mm; Depth of sections; $h = 122$ mm
 Timber strength class; **TR26**

Section properties

Cross sectional area; $A = 4270$ mm²
 Section modulus; $W_y = 86823$ mm³; $W_z = 24908$ mm³
 Second moment of area; $I_y = 5296223$ mm⁴; $I_z = 435896$ mm⁴
 Radius of gyration; $r_y = 35.2$ mm; $r_z = 10.1$ mm

Span details

Bearing length; $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress; $\sigma_{c,y,90,d} = 0.728$ N/mm²; Des.perp.comp.strength; $f_{c,y,90,d} = 1.540$ N/mm²
 Utilisation = **0.472**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

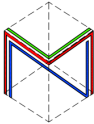
Design shear stress; $\tau_{y,d} = 1.736$ N/mm²; Design shear strength; $f_{v,y,d} = 2.369$ N/mm²
 Utilisation = **0.733**

PASS - Design shear strength exceeds design shear stress

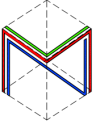
Check design 1150 mm along span

Check bending moment - Section 6.1.6

Design bending stress; $\sigma_{m,y,d} = 21.922$ N/mm²; Design bending strength; $f_{m,y,d} = 17.469$ N/mm²
 Utilisation = **1.255**

 MVN Structural Design Limited	Project				Job no.	
	11 Banfield Crescent, Henfield				M1256	
	Calcs for				Start page no./Revision	
	Loft conversion (of existing truss roof)				27	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
MVN	12/12/2025					

FAIL - Design bending stress exceeds design bending strength Check beams subjected to either bending or combined bending and compression - cl.6.3.3 Utilisation = 1.255 FAIL - Beam stability is not acceptable <u>Consider Combination 2 - 1.00G + 1.00Q (Service)</u> <u>Check design 1150 mm along span</u> Check y-y axis deflection - Section 7.2 Final deflection with creep; $\delta_{y,Final} = 23.1$ mm; Allowable deflection; $\delta_{y,Allowable} = 9.2$ mm Utilisation = 2.509 FAIL - Final deflection exceeds allowable deflection ;	
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 MVN Structural Design Limited	Project		Job no.	
	11 Banfield Crescent, Henfield		M1256	
	Calcs for		Start page no./Revision	
	Loft conversion (of existing truss roof)		28	
	Calcs by	Calcs date	Checked by	Checked date
	MVN	12/12/2025		
			Approved by	Approved date

LOADING ON FLOOR BEAM (FB1a) TO SUPPORT NEW ASHLAR STUD WALL, NEW LOFT FLOOR AND EXISTING/NEW PITCHED ROOF

Span = 4.6m

Dead Imposed

New Pitched Roof: (Slope = 42deg)

Roof Span = 4.2m (on slope dimension between new roof beam & wall plate)

Dead = $(0.85/\cos 42 + 0.3) \times 4.2/2 = 3.0\text{kN/m}$

Imposed = $0.85 \times 4.2/2 = 1.8\text{kN/m}$

New Ashlar Stud Wall:

Dead = $0.8 \times 0.9\text{m high} = 0.8\text{kN/m}$

New Loft Floor:

Dead = $0.75 \times 2.3/2 = 0.9\text{kN/m}$

Imposed = $(1.5 + 1.0) \times 2.3/2 = 2.9\text{kN/m}$

Existing Ceiling:

Dead = $0.3 \times 1.0/2 = 0.15\text{kN/m}$

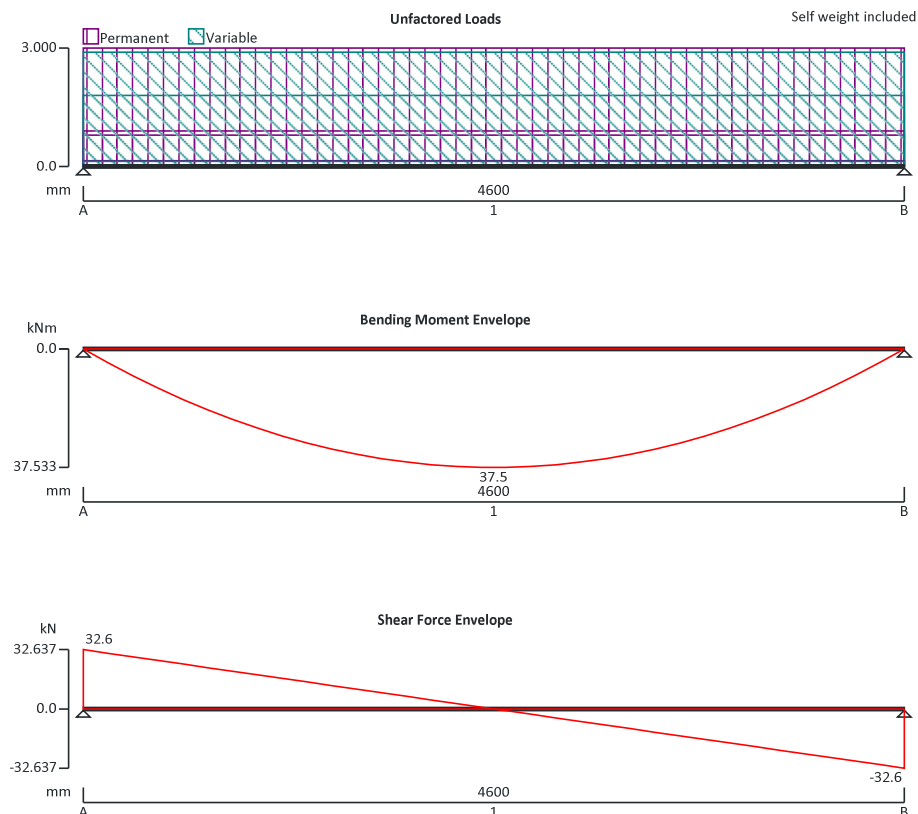
Imposed = $0.25 \times 1.0/2 = 0.13\text{kN/m}$

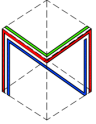
NEW 152X152UC30 FLOOR BEAM (FB1A) - 4600MM SPAN

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 29	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$
new pitched roof - Permanent full UDL 3 kN/m
new pitched roof - Variable full UDL 1.8 kN/m
new ashlar stud wall - Permanent full UDL 0.8 kN/m
new loft floor - Permanent full UDL 0.9 kN/m
new loft floor - Variable full UDL 2.9 kN/m
existing ceiling - Permanent full UDL 0.15 kN/m
existing ceiling - Variable full UDL 0.13 kN/m

Analysis results

Maximum moment;

$M_{\max} = 37.5$ kNm;

$M_{\min} = 0$ kNm

Maximum shear;

$V_{\max} = 32.6$ kN;

$V_{\min} = -32.6$ kN

Deflection;

$\delta_{\max} = 15.8$ mm;

$\delta_{\min} = 0$ mm

Maximum reaction at support A;

$R_{A_{\max}} = 32.6$ kN;

$R_{A_{\min}} = 32.6$ kN

Unfactored permanent load reaction at support A;

$R_{A_{\text{Permanent}}} = 11.8$ kN

Unfactored variable load reaction at support A;

$R_{A_{\text{Variable}}} = 11.1$ kN

Maximum reaction at support B;

$R_{B_{\max}} = 32.6$ kN;

$R_{B_{\min}} = 32.6$ kN

Unfactored permanent load reaction at support B;

$R_{B_{\text{Permanent}}} = 11.8$ kN

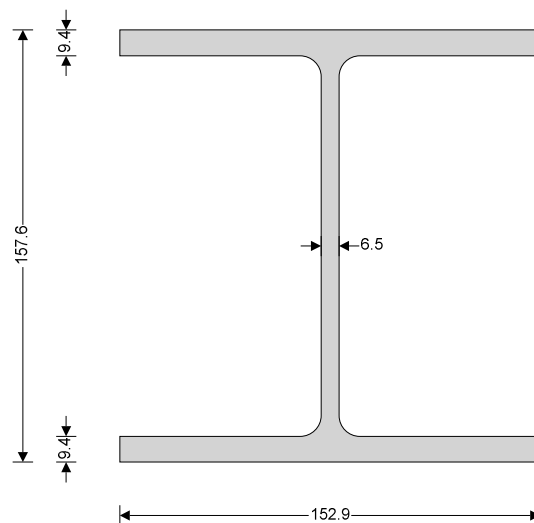
Unfactored variable load reaction at support B;

$R_{B_{\text{Variable}}} = 11.1$ kN

Section details

Section type;

UC 152x152x30 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355



Section classification;

Class 1

Check shear - Section 6.2.6

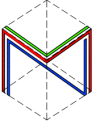
Design shear force;

$V_{Ed} = 33$ kN;

Design shear resistance;

$V_{c,Rd} = 236.9$ kN

PASS - Design shear resistance exceeds design shear force

 MVN Structural Design Limited	Project				Job no.	
	11 Banfield Crescent, Henfield				M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 30	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Check bending moment - Section 6.2.5

Design bending moment; $M_{Ed} = 37.5$ kNm;

Des.bending resist.moment; $M_{c,Rd} = 87.9$ kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT} = 0.898$;

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.400$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - *Lateral torsional buckling cannot be ignored*

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd} = 69$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

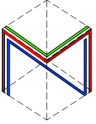
Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = 18.4$ mm;

Maximum deflection; $\delta = 15.841$ mm

PASS - Maximum deflection does not exceed deflection limit

;

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 31	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

LOADING ON FLOOR BEAM (FB1b) TO SUPPORT NEW ASHLAR STUD WALL, NEW LOFT FLOOR AND EXISTING/NEW PITCHED ROOF

Span = 3.6m

Dead Imposed

New Pitched Roof: (Slope = 42deg)

Roof Span = 4.2m (on slope dimension between new roof beam & wall plate)

Dead = $(0.85/\cos 42 + 0.3) \times 4.2/2 = 3.0 \text{ kN/m}$

Imposed = $0.85 \times 4.2/2 = 1.8 \text{ kN/m}$

New Ashlar Stud Wall:

Dead = $0.8 \times 0.9 \text{ m high} = 0.8 \text{ kN/m}$

New Loft Floor:

Dead = $0.75 \times 2.3/2 = 0.9 \text{ kN/m}$

Imposed = $(1.5 + 1.0) \times 2.3/2 = 2.9 \text{ kN/m}$

Existing Ceiling:

Dead = $0.3 \times 1.0/2 = 0.15 \text{ kN/m}$

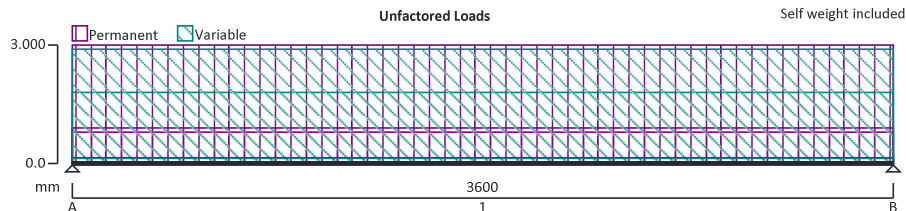
Imposed = $0.25 \times 1.0/2 = 0.13 \text{ kN/m}$

NEW 152X152UC23 FLOOR BEAM (FB1B) - 3600MM SPAN

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

new pitched roof - Permanent full UDL 3 kN/m

new pitched roof - Variable full UDL 1.8 kN/m

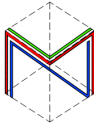
new ashlar stud wall - Permanent full UDL 0.8 kN/m

new loft floor - Permanent full UDL 0.9 kN/m

new loft floor - Variable full UDL 2.9 kN/m

existing ceiling - Permanent full UDL 0.15 kN/m

existing ceiling - Variable full UDL 0.13 kN/m



MVN Structural Design Limited

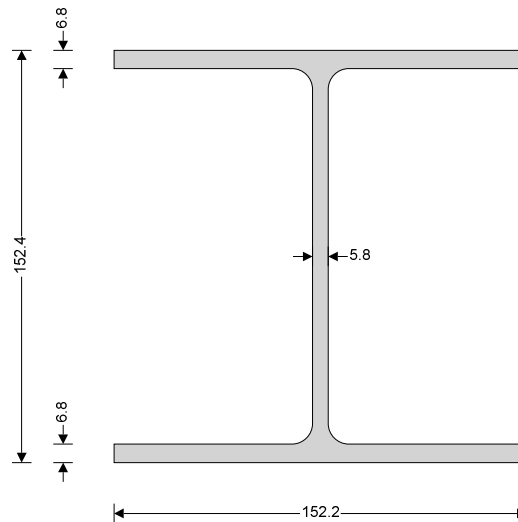
Project 11 Banfield Crescent, Henfield				Job no. M1256	
Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 32	
Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Analysis results

Maximum moment;	$M_{\max} = 22.8 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 25.4 \text{ kN};$	$V_{\min} = -25.4 \text{ kN}$
Deflection;	$\delta_{\max} = 8.3 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 25.4 \text{ kN};$	$R_{A_{\min}} = 25.4 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 9.1 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 8.7 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 25.4 \text{ kN};$	$R_{B_{\min}} = 25.4 \text{ kN}$
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 9.1 \text{ kN}$	
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 8.7 \text{ kN}$	

Section details

Section type; UC 152x152x23 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355

Section classification; **Class 3****Check shear - Section 6.2.6**

Design shear force;	$V_{Ed} = 25 \text{ kN};$	Design shear resistance;	$V_{c,Rd} = 236.9 \text{ kN}$
PASS - Design shear resistance exceeds design shear force			

Check bending moment - Section 6.2.5

Design bending moment;	$M_{Ed} = 22.8 \text{ kNm};$	Des.bending resist.moment;	$M_{c,Rd} = 58.2 \text{ kNm}$
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Slenderness ratio for lateral torsional buckling

LTB slenderness ratio;	$\bar{\lambda}_{LT} = 0.793;$	Limiting slenderness ratio;	$\bar{\lambda}_{LT,0} = 0.400$
$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored			

Design resistance for buckling - Section 6.3.2.1

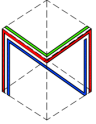
Des.buckling resist.moment;	$M_{b,Rd} = 49.3 \text{ kNm}$	PASS - Design buckling resistance moment exceeds design bending moment	
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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection	$\delta_{lim} = 14.4 \text{ mm};$	Maximum deflection;	$\delta = 8.254 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit			

;

 MVN Structural Design Limited	Project				Job no.	
	11 Banfield Crescent, Henfield				M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 33	
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LOADING ON FLOOR BEAM (FB2a) TO SUPPORT NEW LOFT FLOOR AND CEILING (BOTTOM CHORD OF EXISTING TRUSSES)

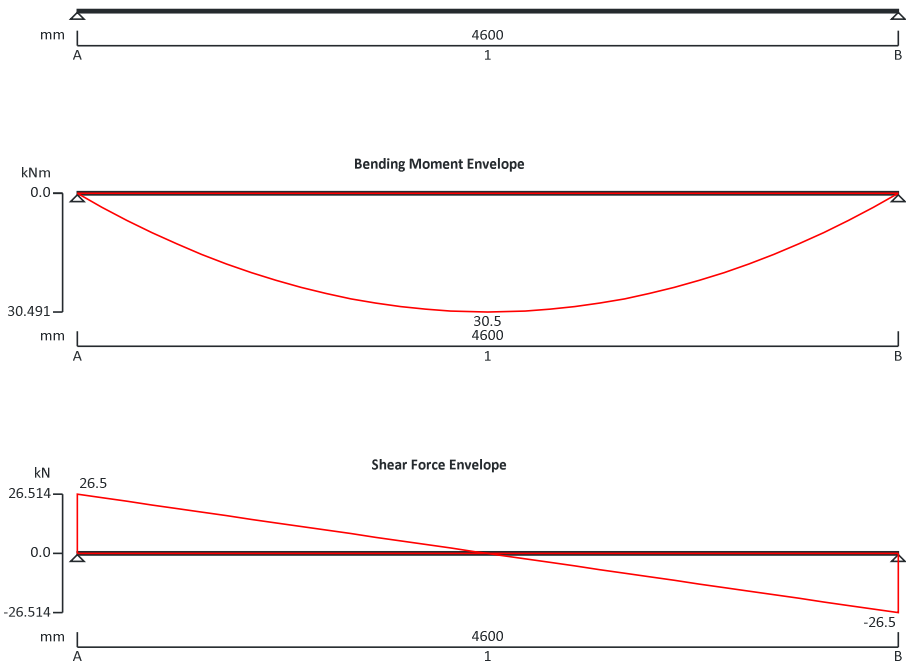
Span = 4.6m		
	Dead	Imposed
New Loft Floor:		
Dead = 0.75 x 2.3 =	1.8kN/m	
Imposed = (1.5 + 1.0) x 2.3 =		5.8kN/m

NEW 152X152UC30 FLOOR BEAM (FB2A)

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15

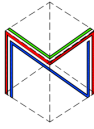


Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	new loft floor - Permanent full UDL 1.8 kN/m
	new loft floor - Variable full UDL 5.8 kN/m

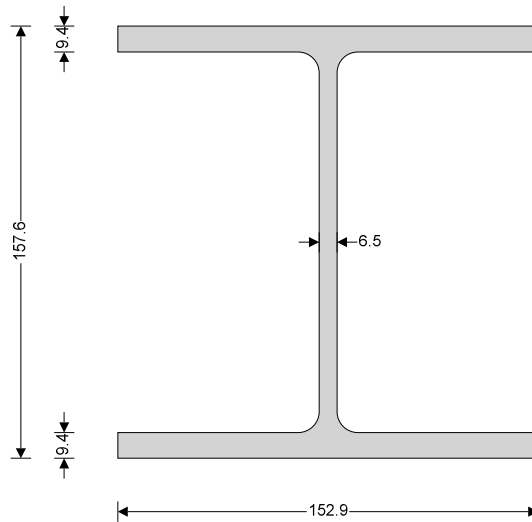


MVN Structural Design Limited

Project 11 Banfield Crescent, Henfield				Job no. M1256	
Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 34	
Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Analysis results

Maximum moment;	$M_{\max} = 30.5 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 26.5 \text{ kN};$	$V_{\min} = -26.5 \text{ kN}$
Deflection;	$\delta_{\max} = 12.5 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 26.5 \text{ kN};$	$R_{A_{\min}} = 26.5 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 4.8 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 13.3 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 26.5 \text{ kN};$	$R_{B_{\min}} = 26.5 \text{ kN}$
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 4.8 \text{ kN}$	
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 13.3 \text{ kN}$	

Section detailsSection type; **UC 152x152x30 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355**Section classification; **Class 1****Check shear - Section 6.2.6**

Design shear force;	$V_{Ed} = 27 \text{ kN};$	Design shear resistance;	$V_{c,Rd} = 236.9 \text{ kN}$
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PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment;	$M_{Ed} = 30.5 \text{ kNm};$	Des.bending resist.moment;	$M_{c,Rd} = 87.9 \text{ kNm}$
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Slenderness ratio for lateral torsional buckling

LTB slenderness ratio;	$\bar{\lambda}_{LT} = 0.898;$	Limiting slenderness ratio;	$\bar{\lambda}_{LT,0} = 0.400$
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$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment;	$M_{b,Rd} = 69 \text{ kNm}$
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PASS - Design buckling resistance moment exceeds design bending moment

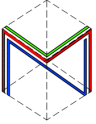
Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection	$\delta_{lim} = 18.4 \text{ mm};$	Maximum deflection;	$\delta = 12.538 \text{ mm}$
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PASS - Maximum deflection does not exceed deflection limit

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 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield		Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)		Start page no./Revision 35	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date
			Approved by	Approved date

ALTERNATIVE NEW 2NO. 47X150C24 FLOOR BEAM (FB2A) - **FAILS**

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 1.7.05

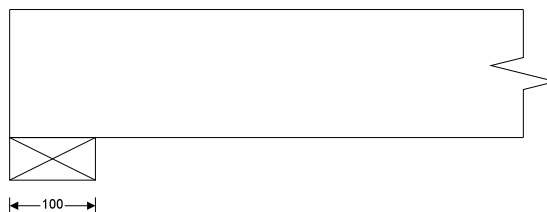
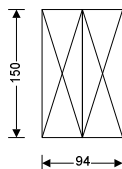
Applied loading

Beam loads

	Permanent self weight of beam $\times 1$
new pitched roof	Permanent full UDL 3.000 kN/m
new pitched roof	Variable full UDL 1.800 kN/m
new ashlar stud wall	Permanent full UDL 0.800 kN/m
new loft floor	Permanent full UDL 0.900 kN/m
new loft floor	Variable full UDL 2.900 kN/m
existing ceiling	Permanent full UDL 0.150 kN/m
existing ceiling	Variable full UDL 0.130 kN/m

Analysis results

Design moment;	$M = 22.450$ kNm;	Design shear;	$F = 24.944$ kN
Total load on member;	$W_{tot} = 49.888$ kN		
Reactions at support A;	$R_{A_max} = 24.944$ kN;	$R_{A_min} = 24.944$ kN	
Unfactored permanent load reaction at support A;	$R_{A_Permanent} = 8.817$ kN		
Unfactored variable load reaction at support A;	$R_{A_Variable} = 8.694$ kN		
Reactions at support B;	$R_{B_max} = 24.944$ kN;	$R_{B_min} = 24.944$ kN	
Unfactored permanent load reaction at support B;	$R_{B_Permanent} = 8.817$ kN		
Unfactored variable load reaction at support B;	$R_{B_Variable} = 8.694$ kN		



Timber section details

Breadth of section;	$b = 47$ mm;	Depth of section;	$h = 150$ mm
Number of sections;	$N = 2$;	Breadth of member;	$b_b = 94$ mm
Timber strength class;	C24		

Member details

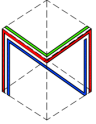
Service class of timber;	1 ;	Load duration;	Long-term
Length of span;	$L_{S1} = 3600$ mm		
Length of bearing;	$L_b = 100$ mm		

Compression perpendicular to grain - cl.6.1.4

Design compressive stress;	$\sigma_{c,90,d} = 2.654$ N/mm ² ;	Design compressive strength;	$f_{c,90,d} = 1.346$ N/mm ²
FAIL - Design compressive stress exceeds design compressive strength at bearing			

Bending - cl 6.1.6

Design bending stress;	$\sigma_{m,d} = 63.687$ N/mm ² ;	Design bending strength;	$f_{m,d} = 12.923$ N/mm ²
FAIL - Design bending stress exceeds design bending strength			

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
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Shear - cl.6.1.7

Applied shear stress;

$\tau_d = 3.961 \text{ N/mm}^2$;

Permissible shear stress;

$f_{v,d} = 2.154 \text{ N/mm}^2$

FAIL - Design shear stress exceeds design shear strength

Deflection - cl.7.2

Deflection limit;

$\delta_{lim} = 14.000 \text{ mm}$;

Total final deflection;

$\delta_{fin} = 104.506 \text{ mm}$

FAIL - Total final deflection exceeds the deflection limit

;

ALTERNATIVE NEW 2NO. 47X225C24 WITH 8MM FLITCH FLOOR BEAM (FB2A) - FAILS

FLITCH 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 1.7.05

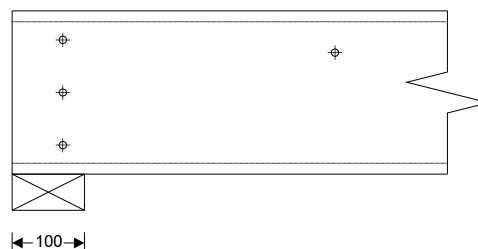
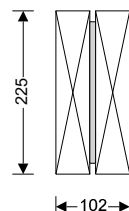
Applied loading

Beam loads

	Permanent self weight of beam $\times 1$
new pitched roof	Permanent full UDL 3.000 kN/m
new pitched roof	Variable full UDL 1.800 kN/m
new ashlar stud wall	Permanent full UDL 0.800 kN/m
new loft floor	Permanent full UDL 0.900 kN/m
new loft floor	Variable full UDL 2.900 kN/m
existing ceiling	Permanent full UDL 0.150 kN/m
existing ceiling	Variable full UDL 0.130 kN/m

Analysis results

Design moment;	$M = 22.765 \text{ kNm}$;	Design shear;	$F = 25.295 \text{ kN}$
Total load on member;	$W_{tot} = 50.589 \text{ kN}$		
Reactions at support A;	$R_{A_max} = 25.295 \text{ kN}$;	$R_{A_min} = 25.295 \text{ kN}$	
Unfactored permanent load reaction at support A;	$R_{A_Permanent} = 9.077 \text{ kN}$		
Unfactored variable load reaction at support A;	$R_{A_Variable} = 8.694 \text{ kN}$		
Reactions at support B;	$R_{B_max} = 25.295 \text{ kN}$;	$R_{B_min} = 25.295 \text{ kN}$	
Unfactored permanent load reaction at support B;	$R_{B_Permanent} = 9.077 \text{ kN}$		
Unfactored variable load reaction at support B;	$R_{B_Variable} = 8.694 \text{ kN}$		



Timber section details

Breadth of section;

$b = 47 \text{ mm}$;

Depth of section;

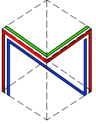
$h = 225 \text{ mm}$

Number of sections;

$N = 2$

Timber strength class;

C24

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	11 Banfield Crescent, Henfield			M1256	
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	MVN	12/12/2025			Approved date

Steel section details

Breadth of steel plate; $b_s = 8$ mm; Depth of steel plate; $h_s = 195$ mm
Number of steel plates in beam; $N_s = 1$; nominal yield stress; $f_y = 275$ N/mm²
Bolt diameter; $\phi_b = 10$ mm

Member details

Service class of timber; **1**; Load duration; **Long-term**
Length of span; $L_{s1} = 3600$ mm
Length of bearing; $L_b = 100$ mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress; $\sigma_{c,90,d} = 2.691$ N/mm²; Design compressive strength; $f_{c,90,d} = 1.346$ N/mm²
FAIL - Design compressive stress exceeds design compressive strength at bearing

Bending - cl 6.1.6

Design timber bending stress; $\sigma_{m,t,d} = 13.949$ N/mm²; Design timber bending strength; $f_{m,d} = 12.923$ N/mm²
FAIL - Design timber bending stress exceeds design timber bending strength
Design steel bending stress; $\sigma_{m,s,d} = 339.206$ N/mm²; Design steel bending strength; $f_{y,d} = 275.000$ N/mm²
FAIL - Design steel bending stress exceeds design steel bending strength

Shear - cl.6.1.7

Applied shear stress; $\tau_d = 1.301$ N/mm²; Permissible shear stress; $f_{v,d} = 2.154$ N/mm²
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = 14.000$ mm; Total final deflection; $\delta_{fin} = 16.928$ mm
FAIL - Total final deflection exceeds the deflection limit

Steel-to-timber connections - cl.8.2.3

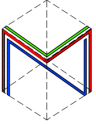
Characteristic yield moment - exp.8.30; $M_{y,Rk} = 0.3 \text{ mm}^{0.4} \times f_{u,k} \times \phi_b^{2.6} = 47773$ Nmm
Char.embed.strength par.to grain - exp.8.32; $f_{h,0,k} = 0.082e9 \text{ m/sec}^2 \times (1 \text{ mm} - 0.01 \times \phi_b) \times \rho_k = 25.830$ N/mm²
 $k_{90} = 1.35 + 0.015 \times \phi_b / 1 \text{ mm} = 1.500$
Char.embed.strength perp.to grain - exp.8.31; $f_{h,90,k} = f_{h,0,k} / k_{90} = 17.220$ N/mm²
Thickness limit for thin steel plates; $b_{s,thin} = \phi_b / 2 = 5$ mm
Thickness limit for thick steel plates; $b_{s,thk} = \phi_b = 10$ mm
Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11
 $F_{v,Rk,f} = f_{h,k} \times b \times \phi_b = 8.093$ kN
 $F_{v,Rk,g} = f_{h,k} \times b \times \phi_b \times (\sqrt{[2 + 4 \times M_{y,Rk} / (f_{h,k} \times \phi_b \times b^2)]} - 1) = 4.709$ kN
 $F_{v,Rk,h} = 2.3 \times \sqrt{[M_{y,Rk} \times f_{h,k} \times \phi_b]} = 6.597$ kN
 $F_{v,Rk} = \text{Min}(F_{v,Rk,f}, F_{v,Rk,g}, F_{v,Rk,h}) = 4.709$ kN

Flitch plate bolting requirements

Total load on member; $W_{tot} = 50.589$ kN; Total load taken by steel; $W_s = 38.217$ kN
Characteristic bolt capacity; $F_{v,Rk} = 4.709$ kN; Number of interfaces; $N_{int} = 2$
Bolts required at supports; $N_{be} = 2.029$; Bolts required to beam length; $N_{bl} = 4.058$
Limiting bolt spacing; $S_{limit} = 563$ mm; Maximum bolt spacing; $S_{max} = 375$ mm
Provide a minimum of 3 No.10 mm diameter bolts at each support
Provide 10 mm diameter bolts at maximum 375 mm centres staggered 55 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing; $S_{end} = 70$ mm; Minimum edge spacing; $S_{edge} = 40$ mm
Minimum bolt spacing; $S_{bolt} = 40$ mm
Minimum washer diameter; $\phi_w = 30$ mm; Minimum washer thickness; $t_w = 3$ mm

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			Approved by	Approved date

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LOADING ON FLOOR BEAM (FB3) TO SUPPORT NEW TIMBER WALL TO LOFT FLOOR

Span = 3.1m

New Pitched Roof:
Roof gable

	Dead	Imposed
--	------	---------

Dead = $(0.85/\cos 42 + 0.3) \times 0.6 = 0.9 \text{ kN/m}$
Imposed = $0.85 \times 0.6 = 0.5 \text{ kN/m}$

New Stud Wall:
Dead = $0.8 \times 2.4 \text{ m high} = 2.0 \text{ kN/m}$

Point load reactions from floor beam (FB1b) at 1m from end (front support):

Dead = 9.1 kN
Imposed = 8.7 kN

NEW 2NO. 47X150C24 FLOOR BEAM (FB3) - FAILS

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 1.7.05

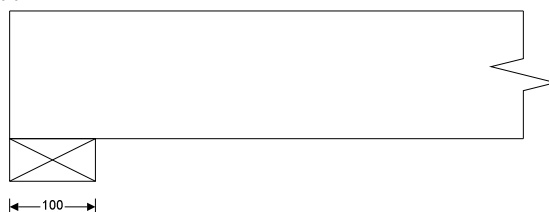
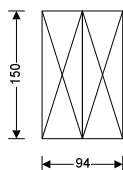
Applied loading

Beam loads

	Permanent self weight of beam $\times 1$
new pitched roof	Permanent full UDL 0.900 kN/m
new pitched roof	Variable full UDL 0.500 kN/m
new ashlar stud wall	Permanent full UDL 2.000 kN/m
reaction from new floor beam (FB1b)	Permanent point load 9.100 kN at 1000 mm
reaction from new floor beam (FB1b)	Variable point load 8.100 kN at 1000 mm

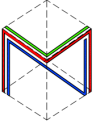
Analysis results

Design moment;	M = 23.797 kNm ;	Design shear;	F = 26.162 kN
Total load on member;	W _{tot} = 41.464 kN		
Reactions at support A;	R _{A_max} = 26.162 kN ;	R _{A_min} = 26.162 kN	
Unfactored permanent load reaction at support A;	R _{A_Permanent} = 11.879 kN		
Unfactored variable load reaction at support A;	R _{A_Variable} = 6.750 kN		
Reactions at support B;	R _{B_max} = 15.302 kN ;	R _{B_min} = 15.302 kN	
Unfactored permanent load reaction at support B;	R _{B_Permanent} = 7.835 kN		
Unfactored variable load reaction at support B;	R _{B_Variable} = 3.150 kN		



Timber section details

Breadth of section; b = **47 mm**; Depth of section; h = **150 mm**

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Number of sections; $N = 2$; Breadth of member; $b_b = 94$ mm
Timber strength class; **C24**

Member details
Service class of timber; **1**; Load duration; **Long-term**
Length of span; $L_{s1} = 3600$ mm
Length of bearing; $L_b = 100$ mm

Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c,90,d} = 2.783$ N/mm²; Design compressive strength; $f_{c,90,d} = 1.346$ N/mm²
FAIL - Design compressive stress exceeds design compressive strength at bearing

Bending - cl 6.1.6
Design bending stress; $\sigma_{m,d} = 67.509$ N/mm²; Design bending strength; $f_{m,d} = 12.923$ N/mm²
FAIL - Design bending stress exceeds design bending strength

Shear - cl.6.1.7
Applied shear stress; $\tau_d = 4.154$ N/mm²; Permissible shear stress; $f_{v,d} = 2.154$ N/mm²
FAIL - Design shear stress exceeds design shear strength

Deflection - cl.7.2
Deflection limit; $\delta_{lim} = 14.000$ mm; Total final deflection; $\delta_{fin} = 103.725$ mm
FAIL - Total final deflection exceeds the deflection limit

NEW 2NO. 47X225C24 WITH 8MM FLITCH FLOOR BEAM (FB3) - FAILS

FLITCH 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 1.7.05

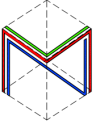
Applied loading

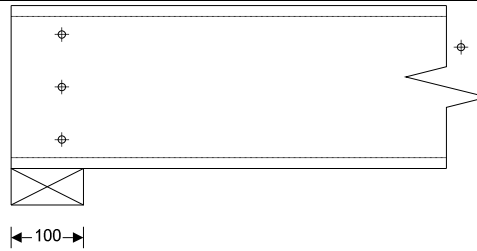
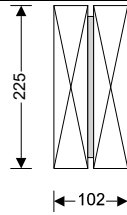
Beam loads

	Permanent self weight of beam $\times 1$
new pitched roof	Permanent full UDL 0.900 kN/m
new pitched roof	Variable full UDL 0.500 kN/m
new ashlar stud wall	Permanent full UDL 2.000 kN/m
reaction from new floro beam (FB1b)	Permanent point load 9.100 kN at 1000 mm
reaction from new floro beam (FB1b)	Variable point load 8.100 kN at 1000 mm

Analysis results

Design moment;	$M = 24.050$ kNm;	Design shear;	$F = 26.513$ kN
Total load on member;	$W_{tot} = 42.165$ kN		
Reactions at support A;	$R_{A,max} = 26.513$ kN;	$R_{A,min} = 26.513$ kN	
Unfactored permanent load reaction at support A;	$R_{A,Permanent} = 12.139$ kN		
Unfactored variable load reaction at support A;	$R_{A,Variable} = 6.750$ kN		
Reactions at support B;	$R_{B,max} = 15.653$ kN;	$R_{B,min} = 15.653$ kN	
Unfactored permanent load reaction at support B;	$R_{B,Permanent} = 8.095$ kN		
Unfactored variable load reaction at support B;	$R_{B,Variable} = 3.150$ kN		

 MVN Structural Design Limited	Project		Job no.	
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	MVN	12/12/2025		
			Approved by	Approved date



Timber section details

Breadth of section; $b = 47 \text{ mm}$; Depth of section; $h = 225 \text{ mm}$
Number of sections; $N = 2$
Timber strength class; **C24**

Steel section details

Breadth of steel plate; $b_s = 8 \text{ mm}$; Depth of steel plate; $h_s = 195 \text{ mm}$
Number of steel plates in beam; $N_s = 1$; nominal yield stress; $f_y = 275 \text{ N/mm}^2$
Bolt diameter; $\phi_b = 10 \text{ mm}$

Member details

Service class of timber; **1**; Load duration; **Long-term**
Length of span; $L_{s1} = 3600 \text{ mm}$
Length of bearing; $L_b = 100 \text{ mm}$

Compression perpendicular to grain - cl.6.1.4

Design compressive stress; $\sigma_{c,90,d} = 2.821 \text{ N/mm}^2$; Design compressive strength; $f_{c,90,d} = 1.346 \text{ N/mm}^2$
FAIL - Design compressive stress exceeds design compressive strength at bearing

Bending - cl 6.1.6

Design timber bending stress; $\sigma_{m,t,d} = 14.737 \text{ N/mm}^2$; Design timber bending strength; $f_{m,d} = 12.923 \text{ N/mm}^2$
FAIL - Design timber bending stress exceeds design timber bending strength

Design steel bending stress; $\sigma_{m,s,d} = 363.106 \text{ N/mm}^2$; Design steel bending strength; $f_{y,d} = 275.000 \text{ N/mm}^2$
FAIL - Design steel bending stress exceeds design steel bending strength

Shear - cl.6.1.7

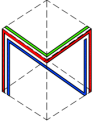
Applied shear stress; $\tau_d = 1.364 \text{ N/mm}^2$; Permissible shear stress; $f_{v,d} = 2.154 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = 14.000 \text{ mm}$; Total final deflection; $\delta_{fin} = 16.960 \text{ mm}$
FAIL - Total final deflection exceeds the deflection limit

Steel-to-timber connections - cl.8.2.3

Characteristic yield moment - exp.8.30; $M_{y,R,k} = 0.3 \text{ mm}^{0.4} \times f_{u,k} \times \phi_b^{2.6} = 47773 \text{ Nmm}$
Char.embed.strength par.to grain - exp.8.32; $f_{h,0,k} = 0.082e9 \text{ m/sec}^2 \times (1 \text{ mm} - 0.01 \times \phi_b) \times \rho_k = 25.830 \text{ N/mm}^2$
 $k_{90} = 1.35 + 0.015 \times \phi_b / 1 \text{ mm} = 1.500$
Char.embed.strength perp.to grain - exp.8.31; $f_{h,90,k} = f_{h,0,k} / k_{90} = 17.220 \text{ N/mm}^2$
Thickness limit for thin steel plates; $b_{s,thin} = \phi_b / 2 = 5 \text{ mm}$
Thickness limit for thick steel plates; $b_{s,thk} = \phi_b = 10 \text{ mm}$
Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11
 $F_{v,Rk,f} = f_{h,k} \times b \times \phi_b = 8.093 \text{ kN}$
 $F_{v,Rk,g} = f_{h,k} \times b \times \phi_b \times (\sqrt{2 + 4 \times M_{y,Rk} / (f_{h,k} \times \phi_b \times b^2)}) - 1 = 4.709 \text{ kN}$
 $F_{v,Rk,h} = 2.3 \times \sqrt{[M_{y,Rk} \times f_{h,k} \times \phi_b]} = 6.597 \text{ kN}$

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$$F_{v.Rk} = \text{Min}(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h}) = 4.709 \text{ kN}$$

Flitch plate bolting requirements

Total load on member; $W_{\text{tot}} = 42.165 \text{ kN}$; Total load taken by steel; $W_s = 32.276 \text{ kN}$
 Characteristic bolt capacity; $F_{v.Rk} = 4.709 \text{ kN}$; Number of interfaces; $N_{\text{int}} = 2$
 Bolts required at supports; $N_{\text{be}} = 2.155$; Bolts required to beam length; $N_{\text{bl}} = 3.427$
 Limiting bolt spacing; $S_{\text{limit}} = 563 \text{ mm}$; Maximum bolt spacing; $S_{\text{max}} = 550 \text{ mm}$
 Provide a minimum of 3 No.10 mm diameter bolts at each support
 Provide 10 mm diameter bolts at maximum 550 mm centres staggered 55 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing; $S_{\text{end}} = 70 \text{ mm}$; Minimum edge spacing; $S_{\text{edge}} = 40 \text{ mm}$
 Minimum bolt spacing; $S_{\text{bolt}} = 40 \text{ mm}$
 Minimum washer diameter; $\phi_w = 30 \text{ mm}$; Minimum washer thickness; $t_w = 3 \text{ mm}$

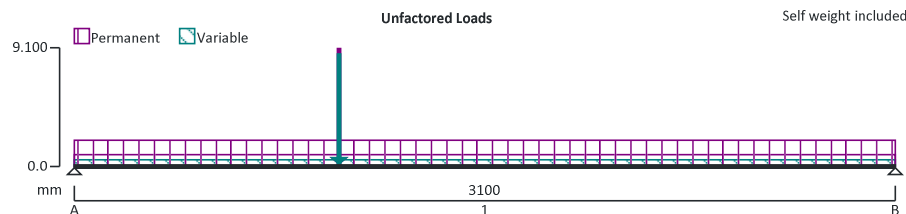
;

NEW 152X152UC23 FLOOR BEAM (FB3) - **ADOPT**

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

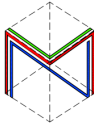
Support A: Vertically restrained, Rotationally free
 Support B: Vertically restrained, Rotationally free

Applied loading

Beam loads:
 Permanent self weight of beam $\times 1$
 new pitched roof - Permanent full UDL 0.9 kN/m
 new pitched roof - Variable full UDL 0.5 kN/m
 new stud wall - Permanent full UDL 2 kN/m
 reaction from new floor beam (FB1b) - Permanent point load 9.1 kN at 1000 mm
 reaction from new floor beam (FB1b) - Variable point load 8.7 kN at 1000 mm

Analysis results

Maximum moment; $M_{\text{max}} = 22.4 \text{ kNm}$; $M_{\text{min}} = 0 \text{ kNm}$
 Maximum shear; $V_{\text{max}} = 24.9 \text{ kN}$; $V_{\text{min}} = -15.9 \text{ kN}$



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Deflection;

 $\delta_{\max} = 2 \text{ mm};$ $\delta_{\min} = 0 \text{ mm}$

Maximum reaction at support A;

 $R_{A_{\max}} = 24.9 \text{ kN};$ $R_{A_{\min}} = 24.9 \text{ kN}$

Unfactored permanent load reaction at support A;

 $R_{A_{\text{Permanent}}} = 11 \text{ kN}$

Unfactored variable load reaction at support A;

 $R_{A_{\text{Variable}}} = 6.7 \text{ kN}$

Maximum reaction at support B;

 $R_{B_{\max}} = 15.9 \text{ kN};$ $R_{B_{\min}} = 15.9 \text{ kN}$

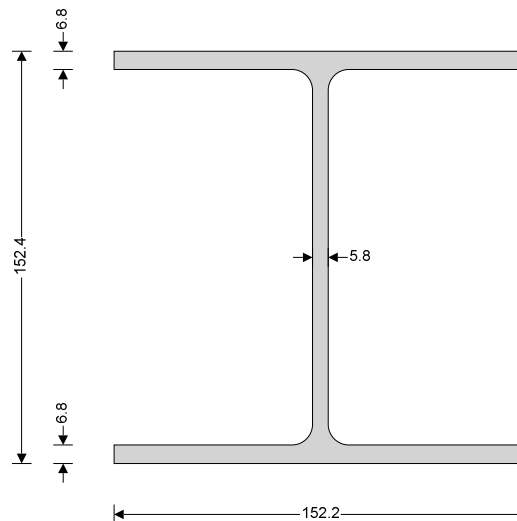
Unfactored permanent load reaction at support B;

 $R_{B_{\text{Permanent}}} = 7.8 \text{ kN}$

Unfactored variable load reaction at support B;

 $R_{B_{\text{Variable}}} = 3.6 \text{ kN}$ **Section details**

Section type;

UC 152x152x23 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355

Section classification;

Class 3**Check shear - Section 6.2.6**

Design shear force;

 $V_{Ed} = 25 \text{ kN};$

Design shear resistance;

 $V_{c,Rd} = 411.7 \text{ kN}$ **PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**

Design bending moment;

 $M_{Ed} = 22.4 \text{ kNm};$

Des.bending resist.moment;

 $M_{c,Rd} = 58.2 \text{ kNm}$ **Slenderness ratio for lateral torsional buckling**

LTB slenderness ratio;

 $\bar{\lambda}_{LT} = 0.708;$

Limiting slenderness ratio;

 $\bar{\lambda}_{LT,0} = 0.400$ **$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored****Design resistance for buckling - Section 6.3.2.1**

Des.buckling resist.moment;

 $M_{b,Rd} = 51.9 \text{ kNm}$ **PASS - Design buckling resistance moment exceeds design bending moment****Check vertical deflection - Section 7.2.1**

Consider deflection due to variable loads

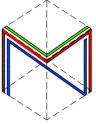
Limiting deflection

 $\delta_{lim} = 8.6 \text{ mm};$

Maximum deflection;

 $\delta = 1.96 \text{ mm}$ **PASS - Maximum deflection does not exceed deflection limit**

;

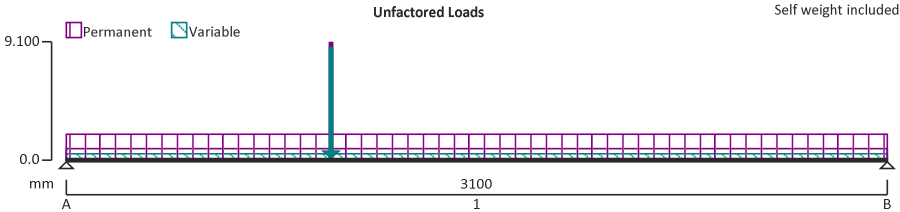
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ALTERNATIVE NEW 152X89UB16 FLOOR BEAM (FB3)

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

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

Applied loading

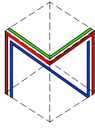
Beam loads	Permanent self weight of beam × 1
	new pitched roof - Permanent full UDL 0.9 kN/m
	new pitched roof - Variable full UDL 0.5 kN/m
	new stud wall - Permanent full UDL 2 kN/m
	reaction from new floor beam (FB1b) - Permanent point load 9.1 kN at 1000 mm
	reaction from new floor beam (FB1b) - Variable point load 8.7 kN at 1000 mm

Analysis results

Maximum moment;	M _{max} = 22.3 kNm;	M _{min} = 0 kNm
Maximum shear;	V _{max} = 24.7 kN;	V _{min} = -15.7 kN
Deflection;	δ _{max} = 2.9 mm;	δ _{min} = 0 mm
Maximum reaction at support A;	R _{A_max} = 24.7 kN;	R _{A_min} = 24.7 kN
Unfactored permanent load reaction at support A;	R _{A_Permanent} = 10.9 kN	
Unfactored variable load reaction at support A;	R _{A_Variable} = 6.7 kN	
Maximum reaction at support B;	R _{B_max} = 15.7 kN;	R _{B_min} = 15.7 kN
Unfactored permanent load reaction at support B;	R _{B_Permanent} = 7.7 kN	
Unfactored variable load reaction at support B;	R _{B_Variable} = 3.6 kN	

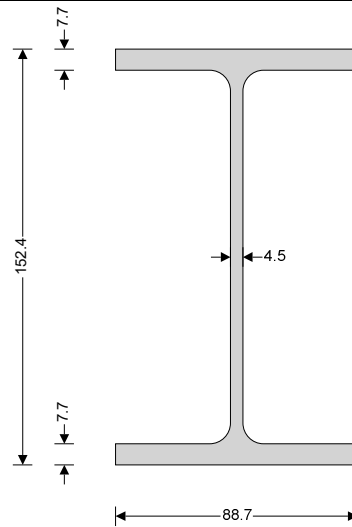
Section details

Section type;	UB 152x89x16 (British Steel Section Range 2022 (BS4-1));	Steel grade;	S355
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Section classification; **Class 1**

Check shear - Section 6.2.6

Design shear force; $V_{Ed} = 25 \text{ kN}$;

Design shear resistance; $V_{c,Rd} = 167.6 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{Ed} = 22.3 \text{ kNm}$;

Des.bending resist.moment; $M_{c,Rd} = 43.8 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT} = 1.154$;

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.400$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd} = 27.1 \text{ kNm}$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

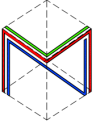
Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = 8.6 \text{ mm}$;

Maximum deflection; $\delta = 2.936 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

;

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LOADING ON LINTEL (L1) TO SUPPORT NEW LOFT FLOOR BEAM (FB3)

Span = $1.2 + 0.15 = 1.4\text{m}$

	Dead	Imposed
Existing Ceiling:		
Dead = $0.3 \times 1/2 =$	0.15kN/m	
Imposed = $0.25 \times 1/2 =$		0.13kN/m

Point load reactions from floor beam (FB3 support A) at midspan:

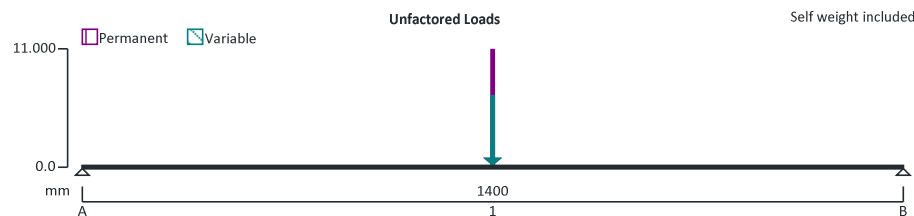
Dead = 11kN
Imposed = 6.7kN

NEW 127X76UB13 LINTEL (L1)

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

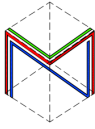
Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent self weight of beam $\times 1$ existing ceiling - Permanent full UDL 0.15 kN/m existing ceiling - Variable full UDL 0.13 kN/m reaction from floor beam FB3 support A - Permanent point load 11 kN at 700 mm reaction from floor beam FB3 support A - Variable point load 6.7 kN at 700 mm
------------	---

Analysis results

Maximum moment;	$M_{\max} = 8.9 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 12.8 \text{ kN};$	$V_{\min} = -12.8 \text{ kN}$
Deflection;	$\delta_{\max} = 0.4 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 12.8 \text{ kN};$	$R_{A_{\min}} = 12.8 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 5.7 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 3.4 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 12.8 \text{ kN};$	$R_{B_{\min}} = 12.8 \text{ kN}$
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 5.7 \text{ kN}$	
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 3.4 \text{ kN}$	

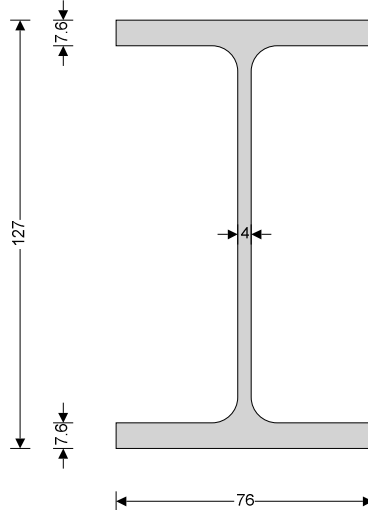


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Section details

Section type;

UB 127x76x13 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355

Section classification;

Class 1**Check shear - Section 6.2.6**

Design shear force;

 $V_{Ed} = 13 \text{ kN}$;

Design shear resistance;

 $V_{c,Rd} = 131.7 \text{ kN}$ **PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**

Design bending moment;

 $M_{Ed} = 8.9 \text{ kNm}$;

Des.bending resist.moment;

 $M_{c,Rd} = 29.9 \text{ kNm}$ **Slenderness ratio for lateral torsional buckling**

LTB slenderness ratio;

 $\bar{\lambda}_{LT} = 0.638$;

Limiting slenderness ratio;

 $\bar{\lambda}_{LT,0} = 0.400$ **$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored****Design resistance for buckling - Section 6.3.2.1**

Des.buckling resist.moment;

 $M_{b,Rd} = 28.8 \text{ kNm}$ **PASS - Design buckling resistance moment exceeds design bending moment****Check vertical deflection - Section 7.2.1**

Consider deflection due to variable loads

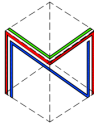
Limiting deflection

 $\delta_{lim} = 3.9 \text{ mm}$;

Maximum deflection;

 $\delta = 0.392 \text{ mm}$ **PASS - Maximum deflection does not exceed deflection limit**

;



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LOADING ON FLOOR BEAM (FB2b) TO SUPPORT NEW LOFT FLOOR AND CEILING (BOTTOM CHORD OF EXISTING TRUSSES)

Span = 4.6m

	Dead	Imposed
New Loft Floor:		
Dead = $0.75 \times 2.3 =$	1.8kN/m	
Imposed = $(1.5 + 1.0) \times 2.3 =$		5.8kN/m

Point load reactions from floor beam (FB3 support B) at 1m from end (party wall support):

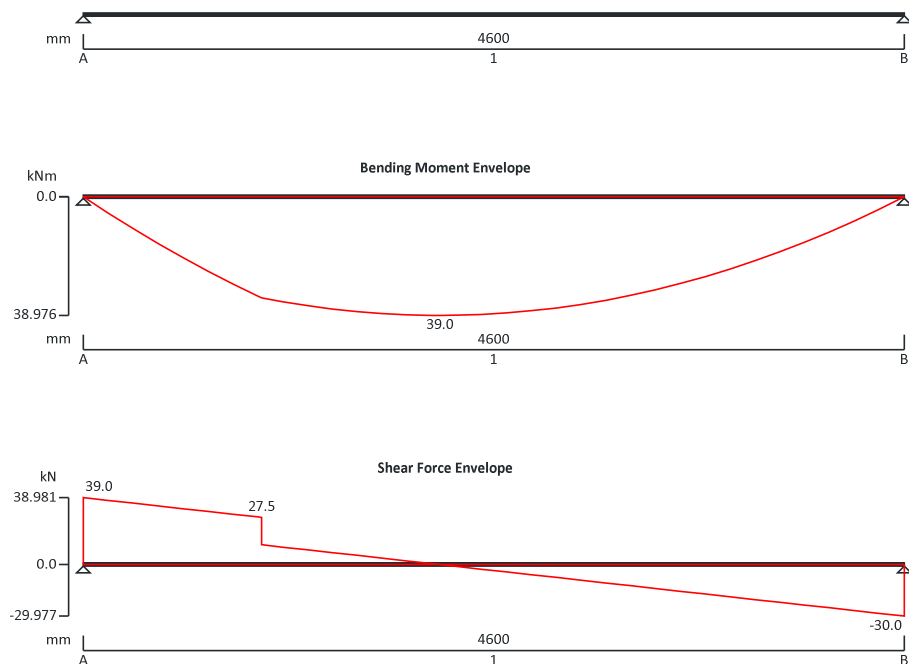
Dead = 7.8kN
Imposed = 3.6kN

NEW 152X152UC30 FLOOR BEAM (FB2B) WITH FB3 REACTIONS

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

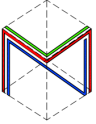
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.15



Support conditions

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

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Applied loading

Beam loads

Permanent self weight of beam $\times 1$
new loft floor - Permanent full UDL 1.8 kN/m
new loft floor - Variable full UDL 5.8 kN/m
reaction from floor beam FB3 support B - Permanent point load 7.8 kN at 1000 mm
reaction from floor beam FB3 support B - Variable point load 3.6 kN at 1000 mm

Analysis results

Maximum moment;

$M_{\max} = 39 \text{ kNm};$

$M_{\min} = 0 \text{ kNm}$

Maximum shear;

$V_{\max} = 39 \text{ kN};$

$V_{\min} = -30 \text{ kN}$

Deflection;

$\delta_{\max} = 16.4 \text{ mm};$

$\delta_{\min} = 0 \text{ mm}$

Maximum reaction at support A;

$R_{A_{\max}} = 39 \text{ kN};$

$R_{A_{\min}} = 39 \text{ kN}$

Unfactored permanent load reaction at support A;

$R_{A_{\text{Permanent}}} = 10.9 \text{ kN}$

Unfactored variable load reaction at support A;

$R_{A_{\text{Variable}}} = 16.2 \text{ kN}$

Maximum reaction at support B;

$R_{B_{\max}} = 30 \text{ kN};$

$R_{B_{\min}} = 30 \text{ kN}$

Unfactored permanent load reaction at support B;

$R_{B_{\text{Permanent}}} = 6.5 \text{ kN}$

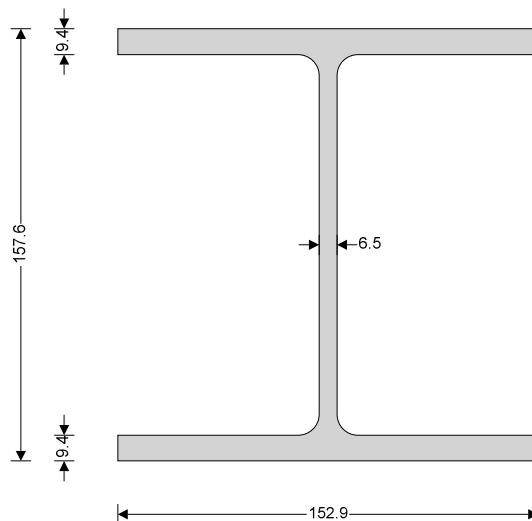
Unfactored variable load reaction at support B;

$R_{B_{\text{Variable}}} = 14.1 \text{ kN}$

Section details

Section type;

UC 152x152x30 (British Steel Section Range 2022 (BS4-1)); Steel grade; S355



Section classification;

Class 1

Check shear - Section 6.2.6

Design shear force;

$V_{Ed} = 39 \text{ kN};$

Design shear resistance;

$V_{c,Rd} = 236.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment;

$M_{Ed} = 39 \text{ kNm};$

Des.bending resist.moment;

$M_{c,Rd} = 87.9 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio;

$\bar{\lambda}_{LT} = 0.898;$

Limiting slenderness ratio;

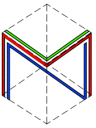
$\bar{\lambda}_{LT,0} = 0.400$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment;

$M_{b,Rd} = 69 \text{ kNm}$

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PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = 18.4$ mm; Maximum deflection; $\delta = 16.41$ mm

PASS - Maximum deflection does not exceed deflection limit

;

EBP (ENG BRICK PADSTONE) TO SUPPORT FB1A, FB1B, FB2A REACTIONS

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	32.6 kN	26.2 kN	1.54 N/mm ²	1.75 N/mm ²	0.880	Pass

Masonry panel details

Panel length; $L = 3200$ mm; Panel height; $h = 2400$ mm
Load bearing leaf thickness; $t = 100$ mm; Effective height; $h_{ef} = 1536$ mm
Effective thickness; $t_{ef} = 126$ mm

Masonry material details

Unit type; **Aggregate concrete - Group 1**
Mean masonry strength; $f_b = 5.0$ N/mm²; Specific weight of units; $\gamma = 18$ kN/m³
Mortar type; **M4 - General Purpose**; Mortar strength; $f_m = 4.0$ N/mm²
Characteristic strength; $f_k = 3.49$ N/mm²

Design compressive strength of masonry for inner (loaded) leaf

Category of manufacturing; **Category II**; Class of execution control; **Class 2**
Partial factor for material; $\gamma_M = 3.00$; Design strength of masonry; $f_d = 1.16$ N/mm²

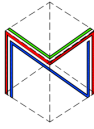
Partial safety factors for design loads

Permanent partial factor; $\gamma_{FG} = 1.35$; Variable partial factor; $\gamma_{FQ} = 1.50$

Slenderness ratio of masonry wall - Section 5.5.1.4

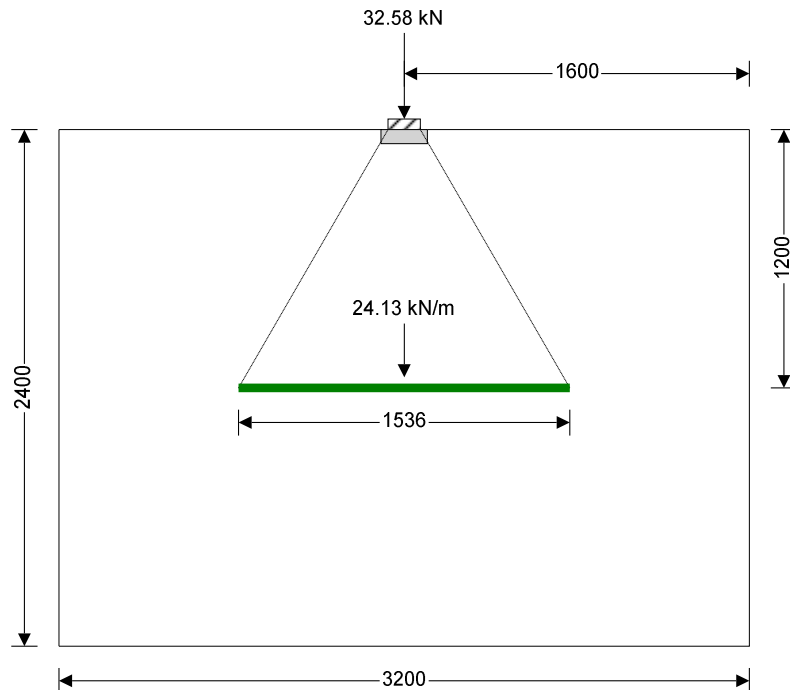
Slenderness ratio limit; $\lambda_{lim} = 27$; Slenderness ratio; $\lambda = 12.2$

PASS - Slenderness ratio is less than slenderness limit



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Concentrated Load 1 details - Max reactions from floor beams (FB1a, FB1b, FB2a) on party wall

Permanent load;	$G_{kc1} = 11.80$ kN;	Variable load;	$Q_{kc1} = 11.10$ kN
Eccentricity of load;	$e_{c1} = 0$ mm;	Length of load;	$L_{c1} = 150$ mm
Width of load;	$w_{c1} = 100$ mm;	Height of load;	$h_{c1} = 2400$ mm
Distance to right edge;	$r_{11} = 1525$ mm;	Distance to nearest edge;	$a_{11} = 1525$ mm

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load;	$N_{Edc1} = 32.58$ kN;	Design resistance;	$N_{Rdc1} = 26.19$ kN
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Applied concentrated load exceeds design resistance, spreader required!**Design of spreader beam**

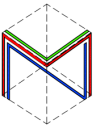
Type of spreader;	Concrete padstone;	Type of load;	Point load
Length of spreader;	$L_{sp1} = 215$ mm;	Width of spreader;	$w_{sp1} = 100$ mm
Height of spreader;	$h_{sp1} = 65$ mm;	Eccentricity of load;	$e_{sp1} = 0$ mm
Modulus of elasticity;	$E_{sp1} = 29962$ N/mm ² ;	Maximum moment;	$M_{Edsp1} = 0.87$ kNm
Maximum shear;	$V_{Edsp1} = 16.29$ kN;	Allowable stress;	$\sigma_{Rdsp1} = 1.75$ N/mm ²
Design stress;	$\sigma_{Edsp1} = 1.54$ N/mm ²		

PASS - Design stress under spreader is less than the allowable bearing stress**Walls subjected to mainly vertical loading - Section 6.1.2**

Vertical load at mid-height;	$N_{Ed1} = 24.13$ kN/m;	Design resistance;	$N_{Rd1} = 93.33$ kN/m
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PASS - Design value of vertical resistance exceeds applied vertical load

;

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300LG X 100W X 100DP PADSTONE (PS1) TO SUPPORT FB2B REACTIONS

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	39.0 kN	26.2 kN	1.32 N/mm ²	1.75 N/mm ²	0.756	Pass

Masonry panel details

Panel length; L = **3200** mm; Panel height; h = **2400** mm
Load bearing leaf thickness; t = **100** mm; Effective height; h_{ef} = **1536** mm
Effective thickness; t_{ef} = **126** mm

Masonry material details

Unit type; **Aggregate concrete - Group 1**
Mean masonry strength; f_b = **5.0** N/mm²; Specific weight of units; γ = **18** kN/m³
Mortar type; **M4 - General Purpose**; Mortar strength; f_m = **4.0** N/mm²
Characteristic strength; f_k = **3.49** N/mm²

Design compressive strength of masonry for inner (loaded) leaf

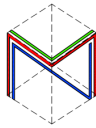
Category of manufacturing; **Category II**; Class of execution control; **Class 2**
Partial factor for material; γ_M = **3.00**; Design strength of masonry; f_d = **1.16** N/mm²

Partial safety factors for design loads

Permanent partial factor; γ_{FG} = **1.35**; Variable partial factor; γ_{FQ} = **1.50**

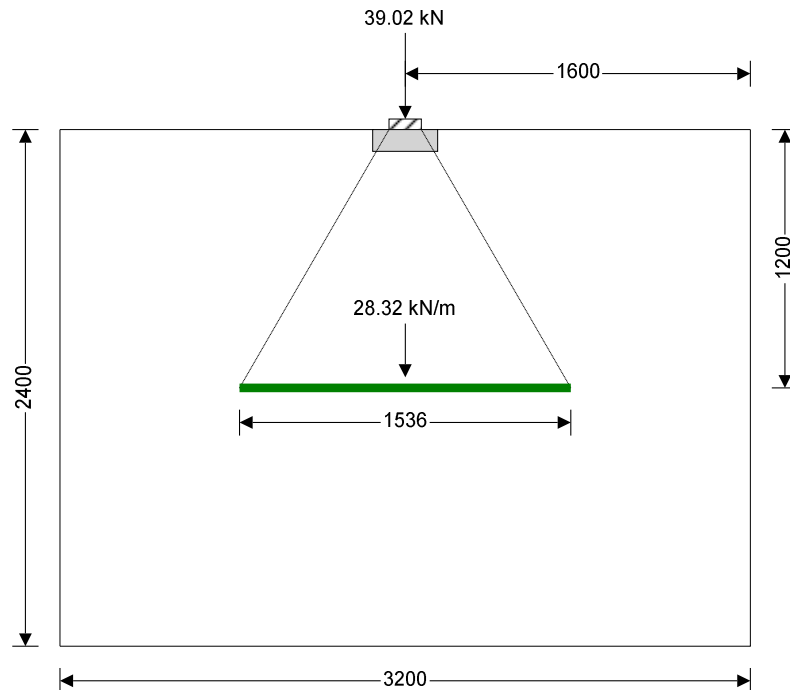
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; λ_{lim} = **27**; Slenderness ratio; λ = **12.2**
PASS - Slenderness ratio is less than slenderness limit



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Concentrated Load 1 details - Reactions from floor beams on party wall (max from FB2b)

Permanent load;	$G_{kc1} = 10.90$ kN;	Variable load;	$Q_{kc1} = 16.20$ kN
Eccentricity of load;	$e_{c1} = 0$ mm;	Length of load;	$L_{c1} = 150$ mm
Width of load;	$w_{c1} = 100$ mm;	Height of load;	$h_{c1} = 2400$ mm
Distance to right edge;	$r_{11} = 1525$ mm;	Distance to nearest edge;	$a_{11} = 1525$ mm

Walls subjected to concentrated loads - Section 6.1.3

Design concentrated load;	$N_{Edc1} = 39.02$ kN;	Design resistance;	$N_{Rdc1} = 26.19$ kN
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Applied concentrated load exceeds design resistance, spreader required!**Design of spreader beam**

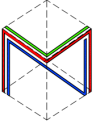
Type of spreader;	Concrete padstone;	Type of load;	Point load
Length of spreader;	$L_{sp1} = 300$ mm;	Width of spreader;	$w_{sp1} = 100$ mm
Height of spreader;	$h_{sp1} = 100$ mm;	Eccentricity of load;	$e_{sp1} = 0$ mm
Modulus of elasticity;	$E_{sp1} = 29962$ N/mm ² ;	Maximum moment;	$M_{Edsp1} = 1.45$ kNm
Maximum shear;	$V_{Edsp1} = 19.51$ kN;	Allowable stress;	$\sigma_{Rdsp1} = 1.75$ N/mm ²
Design stress;	$\sigma_{Edsp1} = 1.32$ N/mm ²		

PASS - Design stress under spreader is less than the allowable bearing stress**Walls subjected to mainly vertical loading - Section 6.1.2**

Vertical load at mid-height;	$N_{Ed1} = 28.32$ kN/m;	Design resistance;	$N_{Rd1} = 93.33$ kN/m
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PASS - Design value of vertical resistance exceeds applied vertical load

;

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SPLICE CONNECTION DESIGN TO NEW FLOOR BEAMS (FB1a)

Provide splice at third point of span = $4.6/3 = 1.5\text{mm}$

Ultimate Design Forces at splice:

Moment = 32.8kN,

Shear = 11.3kN

SPLICE CONNECTION DESIGN TO NEW FLOOR BEAMS (FB2a)

Provide splice at third point of span = $4.6/3 = 1.5\text{mm}$

Ultimate Design Forces at splice:

Moment = 26.5kN,

Shear = 9.1kN

SPLICE CONNECTION DESIGN TO NEW FLOOR BEAMS (FB2b)

Provide splice at third point of span = $4.6/3 = 1.5\text{mm}$ (**worst case ADOPT**)

Ultimate Design Forces at splice:

Moment = 37.3kN,

Shear = 5.6kN

Provide splice at third point of span = $4.6/3 = 3.1\text{mm}$ -

Ultimate Design Forces at splice:

Moment = 31.7kN,

Shear = 12.6kN

152X152UC30 FLOOR BEAM SPLICE CONNECTION (COVER PLATE) AT 1500MM FROM END

STEEL BEAM SPLICE DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

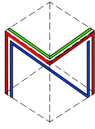
Tedds calculation version 1.0.02

Design summary

Overall design status; PASS

Overall design utilisation; 0.842

Description	Unit	Design	Resistance	Utilisation	Result
Flange bolt group	kN	46.0	60.3	0.764	PASS
Flange bolt slip resist. SLS	kN	33.7	40.0	0.842	PASS
Flange plate compression	kN	276.2	417.2	0.662	PASS
Flange plate tension flange	kN	218.9	392.1	0.558	PASS
Flange plate block tearing	kN	218.9	691.3	0.317	PASS
Flange compression	kN	276.2	510.2	0.541	PASS
Flange tension	kN	218.9	422.6	0.518	PASS
Web bolt group	kN	26.6	54.3	0.490	PASS
Web bolt slip resist. SLS	kN	19.8	79.9	0.248	PASS
Web plate shear	kN	10.0	300.0	0.033	PASS
Web plate block tearing	kN	10.0	356.6	0.028	PASS
Web plate combined				0.331	PASS
Beam web net shear	kN	10.0	227.4	0.044	PASS



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Calcs date

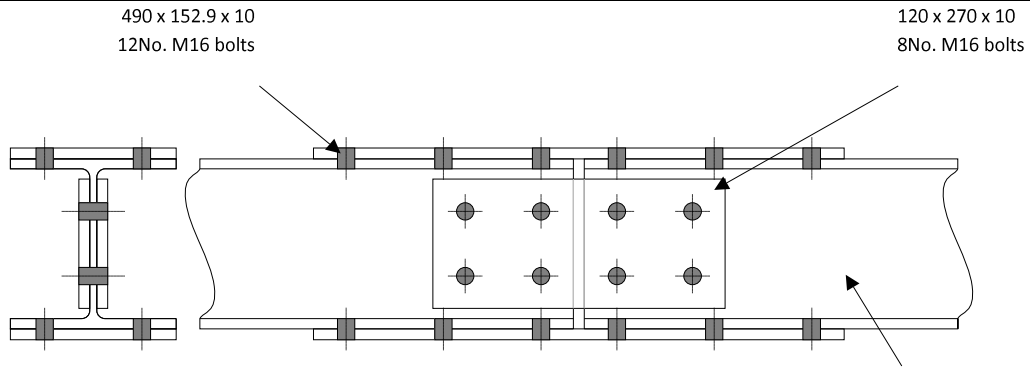
12/12/2025

Checked by

Checked date

Approved by

Approved date



Cover plate Splice

Beam sections:
UC 152x152x30

Design forces

Design moment (ULS); $M_{Ed} = 40.0$ kNm;

Design moment (SLS); $M_{Ed,ser} = 28.0$ kNm

Design shear force (ULS); $V_{Ed} = 10.0$ kN;

Design shear force (SLS); $V_{Ed,ser} = 7.5$ kN

Design axial force (ULS); $N_{Ed} = 75.0$ kN;

Design axial force (SLS); $N_{Ed,ser} = 75.0$ kN

Partial factors

Resistance of cross section; $\gamma_{M0} = 1.00$;

Resist. members to instability; $\gamma_{M1} = 1.00$

Sections in tension to fracture; $\gamma_{M2} = 1.10$;

Resistance of bolts; $\gamma_{M2,b} = 1.25$

Slip resistance; $\gamma_{M3} = 1.25$;

Slip resistance, servability; $\gamma_{M3,ser} = 1.10$

Structural integrity; $\gamma_{M,u} = 1.10$

Beam details

Section name; UC 152x152x30;

Steel grade; S355

Yield strength; $f_{y,bm} = 355$ N/mm²;

Ultimate strength; $f_{u,bm} = 470$ N/mm²

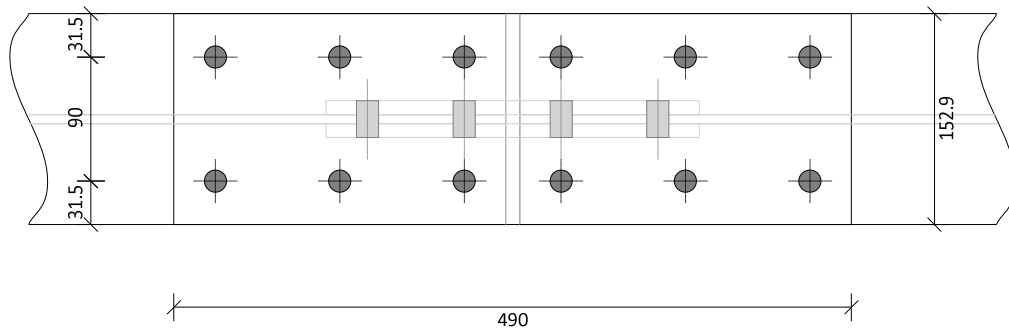
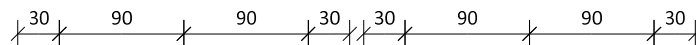
Section height; $h = 157.6$ mm;

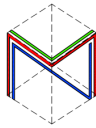
Section width; $b = 152.9$ mm

Flange thickness; $t_f = 9.4$ mm;

Web thickness; $t_w = 6.5$ mm

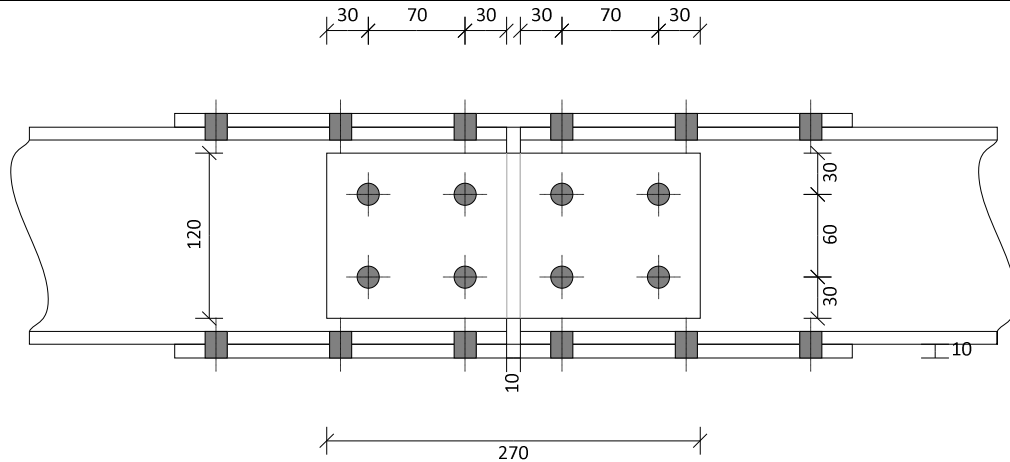
Gap between beams; $g_v = 10.0$ mm





MVN Structural Design Limited

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**Flange bolt details**

Number of bolts	$n_{1,fp} = 3$	Bolt grade	8.8
Bolt size	M16;	Ultimate strength	$f_{ub,fp} = 800 \text{ N/mm}^2$
Yield strength	$f_{yb,fp} = 640 \text{ N/mm}^2$;	End distance in beam	$e_{1,cf} = 30 \text{ mm}$
End distance in plate	$e_{1,fp} = 30 \text{ mm}$;		
Edge distance in plate	$e_{2,fp} = 31.5 \text{ mm}$		
Bolt pitch	$p_{1,fp} = 90 \text{ mm}$;	Bolt gauge	$p_{2,fp} = 90 \text{ mm}$

Flange plate details

External plate width	$b_{fp,ext} = 152.9 \text{ mm}$	Plate thickness	$t_{fp} = 10 \text{ mm}$
Plate length	$l_{fp} = 490 \text{ mm}$;	Yield strength	$f_{y,fp} = 275 \text{ N/mm}^2$
Plate grade	S275;		
Ultimate strength	$f_{u,fp} = 410 \text{ N/mm}^2$		

Web bolt details

Number of bolt columns	$n_{1,wp} = 2$;	Number of bolt rows	$n_{2,wp} = 2$
Bolt size	M16;	Bolt grade	8.8
Yield strength	$f_{yb,wp} = 640 \text{ N/mm}^2$;	Ultimate strength	$f_{ub,wp} = 800 \text{ N/mm}^2$
End distance in plate	$e_{1,wp} = 30 \text{ mm}$;	End distance in beam	$e_{1,ow} = 30 \text{ mm}$
Edge distance in plate	$e_{2,wp} = 30 \text{ mm}$		
Bolt gauge	$p_{1,wp} = 70 \text{ mm}$;	Bolt pitch	$p_{2,wp} = 60 \text{ mm}$

Web plate details

Plate width	$b_{wp} = 270 \text{ mm}$;	Plate height	$h_{wp} = 120 \text{ mm}$
Plate thickness	$t_{wp} = 10 \text{ mm}$;	Number of web plates;	$N_{wp} = 2$
Plate grade	S275		
Yield strength	$f_{y,wp} = 275 \text{ N/mm}^2$;	Ultimate strength	$f_{u,wp} = 410 \text{ N/mm}^2$

Continuity - EN1993-1-8 cl.6.2.7.1(13)

Design bending resist. mnt;	$M_{c,y,Rd} = 87.9 \text{ kNm}$
	$0.25 \times M_{c,y,Rd} / M_{Ed} = 0.550$

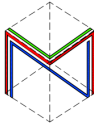
M_{Ed} is greater or equal to $0.25 \times M_{c,y,Rd}$ so continuity is satisfied

Internal forces - ULS

Total force in tension flange;	$F_{tf} = 218.9 \text{ kN}$;	Total force in comp. flange;	$F_{cf} = 276.2 \text{ kN}$
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Flange bolt resistance

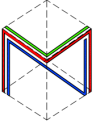
Max design force on bolts;	$F_{f,v,Ed} = 46.0 \text{ kN}$;	Design resistance of bolt;	$F_{t,Rd} = 60.3 \text{ kN}$
	$F_{f,v,Ed} / F_{t,Rd} = 0.764$		



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PASS - Bolt resistance exceeds applied force on bolt**Internal forces - SLS**Total force in tension flange; $F_{tf,ser} = 144.6$ kN;Total force in comp. flange; $F_{cf,ser} = 201.9$ kN**Flange bolt resistance SLS**Max design force flange bolts; $F_{t,v,Ed,ser} = 33.7$ kN;Slip resistance of single bolt; $F_{s,fp,Rd,ser} = 40.0$ kN $F_{t,v,Ed,ser} / F_{s,fp,Rd,ser} = 0.842$ **PASS - Bolt resistance exceeds applied force on bolt****Flange cover plates - Resistance of plates in compression**Design comp. resistance; $N_{b,fp,Rd} = 417.15$ kNUtilisation; $F_{cf} / N_{b,fp,Rd} = 0.662$ **PASS - Flange cover plate compressive resistance greater than design compressive force****Flange cover plates - Resistance of plates in tension**Design tension resistance; $N_{t,fp,Rd} = 392.15$ kNUtilisation; $F_{tf} / N_{t,fp,Rd} = 0.558$ **PASS - Flange cover plate tensile resistance greater than design tensile force****Flange cover plates - block tearing**Block tearing resistance; $N_{bt,Rd} = 691.3$ kNUtilisation; $F_{tf} / N_{bt,Rd} = 0.317$ **PASS - Flange cover plate block tearing resistance greater than design tensile force****Beam flange - resistance in compression**Design compressive resist.; $N_{b,Rd} = 510.23$ kNUtilisation; $F_{cf} / N_{b,Rd} = 0.541$ **PASS - Beam flange compressive resistance greater than design compressive force****Beam flange - resistance in tension**Design tension resistance; $N_{t,Rd} = 422.56$ kNUtilisation; $F_{tf} / N_{t,Rd} = 0.518$ **PASS - Beam flange tensile resistance greater than design tensile force****Internal forces - ULS**Force in web due to axial force; $F_{w,N} = 17.7$ kN;Force in web due to vert. shear; $F_{w,V} = 10.0$ kN**Web bolt resistance ULS**Result. force on extreme bolt; $F_{w,v,Ed} = 26.6$ kN;Design resistance of bolt; $F_{w,Rd} = 54.3$ kN $F_{w,v,Ed} / F_{w,Rd} = 0.490$ **PASS - Bolt resistance exceeds applied force on bolt****Internal forces - SLS**Force in web due to axial force; $F_{w,N,ser} = 17.7$ kN;Force in web due to vert. shear; $F_{w,V,ser} = 7.5$ kN**Web bolt resistance SLS**Result. force on extreme bolt; $F_{w,v,Ed,ser} = 19.8$ kN;Slip resistance of single bolt; $F_{s,wp,Rd,ser} = 79.9$ kN $F_{w,v,Ed,ser} / F_{s,wp,Rd,ser} = 0.248$ **PASS - Bolt resistance exceeds applied force on bolt****Resistance of web cover plate in shear**Plastic resist. web cover plate; $V_{pl,wp,Rd} = 300.04$ kN $V_{Ed} / V_{pl,wp,Rd} = 0.033$ **PASS - Shear resistance of web plate exceeds applied shear**

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Block tearing of web cover plate

Block tearing resistance; $V_{b,Rd} = 356.60 \text{ kN}$
 $V_{Ed} / V_{b,Rd} = 0.028$

PASS - Block tearing resistance of web plate exceeds applied shear

Resistance of beam web shear accounting for holes

Design shear net resistance; $V_{nw,Rd} = 227.4 \text{ kN}$
 $V_{Ed} / V_{nw,Rd} = 0.044$

PASS - Net shear resistance of beam web exceeds applied shear force

Resistance of web cover plate to combined bending, shear and axial

Bend. resist. web cover plate; $M_{c,wp,Rd} = 13.2 \text{ kNm}$; Axial resist. of web cover plate; $N_{wp,Rd} = 660.0 \text{ kN}$
Applied moment; $M_{wp,Ed} = 4.0 \text{ kNm}$; Applied axial force; $N_{wp,Ed} = 17.7 \text{ kN}$
Interaction formula; $N_{wp,Ed} / N_{wp,Rd} + M_{wp,Ed} / M_{c,wp,Rd} = 0.331$

PASS - Combined bending, shear and axial check satisfied

ALTERNATIVE 152X152UC30 FLOOR BEAM SPLICE CONNECTION (END PLATE) AT 1500MM FROM END - FAILS

STEEL BEAM SPLICE DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

Tedds calculation version 1.0.02

Design summary

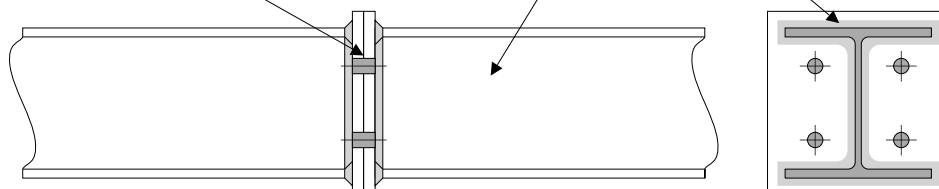
Overall design status; FAIL
Overall design utilisation; 1.859

Description	Unit	Design	Resistance	Utilisation	Result
Moment resistance joint	kNm	31.7	17.1	1.859	FAIL
Shear resist. bolt group	kN	5.6	154.3	0.036	PASS
Bearing resist. bolt group	kN	5.6	159.5	0.035	PASS
Tension flange weld	kN	151.2	695.6	0.217	PASS
Web weld tension zone	kN/mm	0.59	2.92	0.203	PASS
Web weld shear zone	kN/mm	0.20	2.38	0.083	PASS
Comp. flange weld	mm	6.0	8.0	0.750	PASS

2No. M16 bolts in tension zone
2No. M16 bolts in compression zone

Beam sections:
UC 152x152x30

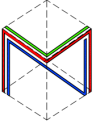
End plate
190 x 190



End plate Splice

Design forces

Design moment; $M_{Ed} = 37.3 \text{ kNm}$; Design shear force; $V_{Ed} = 5.6 \text{ kN}$
Design axial force; $N_{Ed} = 75.0 \text{ kN}$

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Partial factors

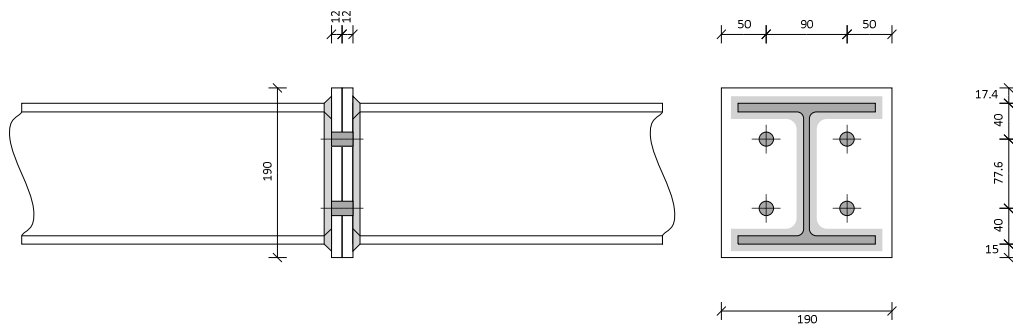
Resistance of cross section; $\gamma_{M0} = 1.00$;
Sections in tension to fracture; $\gamma_{M2} = 1.10$;
Slip resistance; $\gamma_{M3} = 1.25$;
Structural integrity; $\gamma_{M,u} = 1.10$

Resist. members to instability; $\gamma_{M1} = 1.00$
Resistance of bolts; $\gamma_{M2,b} = 1.25$
Slip resistance, servability; $\gamma_{M3,ser} = 1.10$

Beam details

Section name; UC 152x152x30;
Yield strength; $f_{y,bm} = 355 \text{ N/mm}^2$;
Section height; $h = 157.6 \text{ mm}$;
Flange thickness; $t_f = 9.4 \text{ mm}$;

Steel grade; S355
Ultimate strength; $f_{u,bm} = 470 \text{ N/mm}^2$
Section width; $b = 152.9 \text{ mm}$
Web thickness; $t_w = 6.5 \text{ mm}$



End plate bolt details

Number of bolt columns $n_{1,ep} = 2$
Number of upper bolt rows $n_{2u,ep} = 1$;
Bolt gauge $p_{1,ep} = 90 \text{ mm}$;
Bolt size **M16**;
Yield strength $f_{yb,ep} = 640 \text{ N/mm}^2$;
Edge distance in plate $e_{1,ep} = 50 \text{ mm}$
Dist. int. bolt to face of flange $X_{2if,ep} = 40 \text{ mm}$
Design tension resist. of bolt; $F_{t,Rd} = 90.4 \text{ kN}$

Number of lower bolt rows $n_{2l,ep} = 1$
Bolt pitch $p_{2,ep} = 40 \text{ mm}$
Bolt grade **8.8**
Ultimate strength $f_{ub,ep} = 800 \text{ N/mm}^2$

End plate details

Plate width $b_{ep} = 190 \text{ mm}$;
Plate thickness $t_{ep} = 12 \text{ mm}$;
Yield strength $f_{y,ep} = 275 \text{ N/mm}^2$;

Plate height $h_{ep} = 190 \text{ mm}$
Plate grade **S275**
Ultimate strength $f_{u,ep} = 410 \text{ N/mm}^2$

End plate bending resistance

Variables used for effective length (Figure 6.10)

$m_x = 30.4 \text{ mm}$;
 $w = 90 \text{ mm}$;
 $e = 50 \text{ mm}$;

$m_2 = 21.0 \text{ mm}$
 $p = 40 \text{ mm}$
 $e_x = 50 \text{ mm}$

Quarter of bolt washer dia.; $e_w = 7 \text{ mm}$

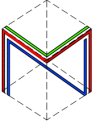
Bolt row 1 - other end tension flange of beam

Design resist. T-stub - mode 1; $F_{T,1,Rd,r1} = 170.6 \text{ kN}$;
Design resist. T-stub - mode 3; $F_{T,3,Rd,r1} = 180.9 \text{ kN}$;
Resist of beam web in tension; $F_{t,wb,Rd,r1} = 304.5 \text{ kN}$

Design resist. T-stub - mode 2; $F_{T,2,Rd,r1} = 151.2 \text{ kN}$
Resist of end plate to bending; $F_{t,ep,Rd,r1} = 151.2 \text{ kN}$

Resistance of bolt row 1

Resistance of bolt row; $F_{t,Rd,r1} = 151.2 \text{ kN}$

 MVN Structural Design Limited	Project 11 Banfield Crescent, Henfield				Job no. M1256	
	Calcs for Loft conversion (of existing truss roof)				Start page no./Revision 59	
	Calcs by MVN	Calcs date 12/12/2025	Checked by	Checked date	Approved by	Approved date

Minimum resistance of bolt rows

Effective resist. of bolt row 1; $F_{t,Rd,eff,r1} = 151.2$ kN

Design resistance of beam flange and adjacent compression zone of web

Design bending resistance mnt $M_{c,y,Rd} = 87.9$ kNm; Comp zone resist of flng & wb; $F_{c,fb,Rd} = 593.3$ kN

Lever arm of bolts from mid thickness of compression flange

Plastic distribution limit

Design tension resistance limit; $F_{t,Rd,lim} = 171.8$ kN

As $t_{ep} \leq d_{b,ep} / 1.9 \times \sqrt{(f_{ub,ep} / f_{y,bm})}$ full plastic distribution can be assumed is the application of Equation 6.26 is not required (NA.2.7)

Total tension resistance of bolt rows

Total resistance of bolt rows; $\Sigma F_{t,Rd} = 151.2$ kN

Equilibrium of forces

For horizontal equilibrium $\Sigma F_{t,Rd} + N_{Ed}$ cannot exceed $F_{c,fb,Rd}$

$\Sigma F_{t,Rd} + N_{Ed} \leq F_{c,fb,Rd}$ - no reduction in any row resistances is required

Moment resistance of joint

Mod. mnt due to axial force; $M_{mod,Ed} = 31.7$ kNm

Moment resistance of joint; $M_{j,Rd} = 17.1$ kNm

$M_{mod,Ed} / M_{j,Rd} = 1.859$

FAIL - Applied moment greater than moment resistance of joint

Shear resistance of joint

Shear resistance of joint; $F_{v,ep,Rd} = 154.3$ kN

$V_{Ed} / F_{v,ep,Rd} = 0.036$

PASS - Shear resistance of joint greater than applied shear

Bearing resistance of bolts

Bearing resistance of bolts; $F_{b,ep,Rd} = 159.5$ kN

$V_{Ed} / F_{b,ep,Rd} = 0.035$

PASS - Bearing resistance of bolts greater than applied shear

Tension flange weld

Design force - local bolt forces; $F_{tf,w,Ed} = 151.2$ kN;

Design resistance of weld; $F_{tn,w,Rd} = 695.6$ kN

$F_{tf,w,Ed} / F_{tn,w,Rd} = 0.217$

PASS - Design resistance of weld greater or equal than design force due to local bolt forces

Web weld in tension zone

Tension force per unit length; $F_{T,ww,Ed} = 0.591$ kN/mm;

Design resistance of weld; $F_{T,ww,Rd} = 2.916$ kN/mm

$F_{T,ww,Ed} / F_{T,ww,Rd} = 0.203$

PASS - Design resistance of weld greater or equal than tension force in weld

Web weld in shear zone

Tension force per unit length; $F_{t,ww,Ed} = 0.198$ kN/mm;

Design resistance of weld; $F_{t,ww,Rd} = 2.381$ kN/mm

$F_{t,ww,Ed} / F_{t,ww,Rd} = 0.083$

PASS - Design resistance of weld greater or equal than tension force in weld

Compression flange weld - beam end is sawn with a bearing fit so design for nominal welds

Leg length of weld;

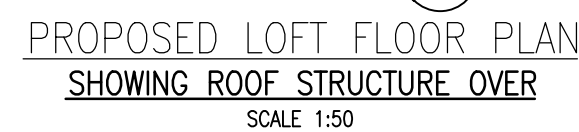
$S_{t,top} = 8.00$ mm;

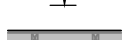
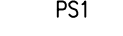
Leg length of weld required;

$S_{t,top,nom} = 6.00$ mm

PASS - Weld greater or equal to nominal weld size

;



- # LEGEND
- | | |
|---|--|
|  | SPAN OF NEW 47x100C24 CEILING JOISTS AT 600mm CENTRES (OR TO MATCH EXISTING TRUSS CENTRES) |
|  | SPAN OF NEW 47x150C24 RAFTERS AT 600mm CENTRES (OR TO MATCH EXISTING TRUSS CENTRES) |
|  | SPAN OF NEW 47x150C24 FLOOR JOISTS AT 600mm CENTRES (OR TO MATCH EXISTING TRUSS CENTRES) |
|  | DENOTES DOUBLE JOISTS |
|  | DENOTES N.G.S. CALY. RESTRAINT STRAPS, REFER TO GENERAL NOTES AND STANDARD DETAILS. |
|  | NEW STUD WALL |
|  | DENOTES EXISTING MASONRY WALL (TO REMAIN) |
|  | DENOTES EXISTING WALL (TO BE REMOVED) |
|  | RB1: 152x152UC30 ROOF BEAMS [S355] |
|  | FB1A, FB2A & FB2B: 152x152UC30 FLOOR BEAMS [S355] |
|  | FB1B & FB3: 152x152UC23 FLOOR BEAMS [S355] |
|  | L1: 127x76UB13 LINTEL [S355] |
|  | DENOTES ENGINEERING BRICK PADSTONES. |
|  | DENOTES 300kg. x 100kg. x 100dp. MASS CONCRETE PADSTONES. |

- # GENERAL NOTES
1. ALL DIMENSIONS TO BE CHECKED ON SITE. ALL DETAILS AND DIMENSIONS RELATING TO SUB-CONTRACTOR'S OR SUPPLIER'S WORK MUST BE CHECKED AND AGREED BETWEEN THE SUB-CONTRACTOR OR THE SUPPLIER AND THE GENERAL CONTRACTOR.
 2. THE MAIN CONTRACTOR IS RESPONSIBLE FOR ENSURING THE STABILITY OF THE STRUCTURE WHILST THE WORKS ARE IN PROGRESS.
 3. BUILDING BLOCKS WITH A UNIT WEIGHT OF 20kg OR MORE ARE NOT TO BE USED IN THE CONSTRUCTION OF THIS PROJECT.
 4. ALL DIMENSIONS ARE IN MILLIMETRES, UNLESS NOTED OTHERWISE.
 5. THE CONTRACTOR IS RESPONSIBLE FOR VERIFYING ALL SITE DIMENSIONS BEFORE COMMENCING WORK.
 6. DIMENSIONS ON THIS DRAWING SHALL NOT BE CALED.
 7. NO VARIATION OR AMENDMENT TO THE DETAILS SHOWN ON THIS PLAN SHALL BE CARRIED OUT WITHOUT THE PRIOR APPROVAL OF THE ENGINEER.
 8. 30x36x1200 (g) M.S GALV. STRAPS SPACED AT 1800 CRS TO RAFTER/JOISTS TO RESTRAIN CABLE WAISTS. STRAPS TO BE FIXED TO AT LEAST 3No. RAFTERS/JOISTS.
- NOTATION:-
- | | |
|-------|----------------------|
| F.F.L | FINISHED FLOOR LEVEL |
| T.O.S | TOP OF STEEL |
| B.O.S | BOTTOM OF STEEL |

THE FOLLOWING ITEMS ARE CONSIDERED BY THE DESIGNER TO BE A SIGNIFICANT RISK DURING CONSTRUCTION. THE LIST IS NOT EXHAUSTIVE BUT HAS BEEN PREPARED TO COMMUNICATE RISKS WHICH HAVE BEEN IDENTIFIED DURING THE DESIGN PROCESS.

1. WORKS ARE NOT TO COMMENCED UNTIL A PARTY WALL AGREEMENT IS IN PLACE.
2. CONTRACTOR IS RESPONSIBLE FOR STABILITY OF STRUCTURE. TEMPORARY WORKS TO CONTRACTOR'S DESIGN.

ALL PROPS TO BE LACED AND BRACED IN BOTH DIRECTIONS.
3. CONTRACTOR TO CHECK SIZE, LEVELS AND DIMENSIONS OF EXISTING CONSTRUCTION PRIOR TO FABRICATION OF NEW STEELWORK.



THE FOLLOWING RECOMMENDED CONSTRUCTION STEPS HAVE BEEN PREPARED TO COMMUNICATE RISKS OF INSTABILITY AND THEREFORE ARE GUIDELINES ONLY. CONTRACTOR IS RESPONSIBLE FOR THE OVERALL STABILITY OF STRUCTURE AS PER H&S GUIDELINES.

1. BACK PROP BOTTOM CHORD OF EXISTING ROOF TRUSSES (CEILING) TO GROUND FLOOR.
2. STRENGTHEN WINDOW OPENING WITH NEW LINTEL (L1).
3. INSTALL NEW FLOOR BEAMS (FB2s) AND TRIM BOTTOM CHORD OF EXISTING ROOF TRUSSES (CEILING) AS DETAILED. SUPPORT AND TIE EXISTING CEILING TO NEW FLOOR BEAMS.
4. INSTALL NEW FLOOR BEAM (FB3).
5. INSTALL NEW FLOOR BEAMS (FB1s) TO SUPPORT NEW ASHLAR STUD WALL.
6. INSTALL NEW FLOOR JOISTS (FJ).
7. INSTALL ROOF BEAMS (RB1) AND THE NEW CEILING JOISTS (CJ) TO BEAMS.
8. BOLT NEW RAFTERS TO TOP CHORD OF EXISTING ROOF TRUSSES AND TRUSS CLIP TO WALL PLATES AS DETAILED IN SECTIONS.
9. ONCE ALL RAFTERS HAVE BEEN STRENGTHENED, REMOVE EXISTING TRUSS DIAGONAL STRUTS.





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STANDARD DETAILS

M1256

STD-1

MVN

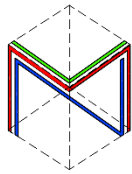
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N.T.S.



N.T.S.



**MVN
STRUCTURAL
DESIGN**

Job Title

11 Banfield Gardens, Henfield

Date

Dec 2025

Item

STANDARD DETAILS

Job No.

M1256

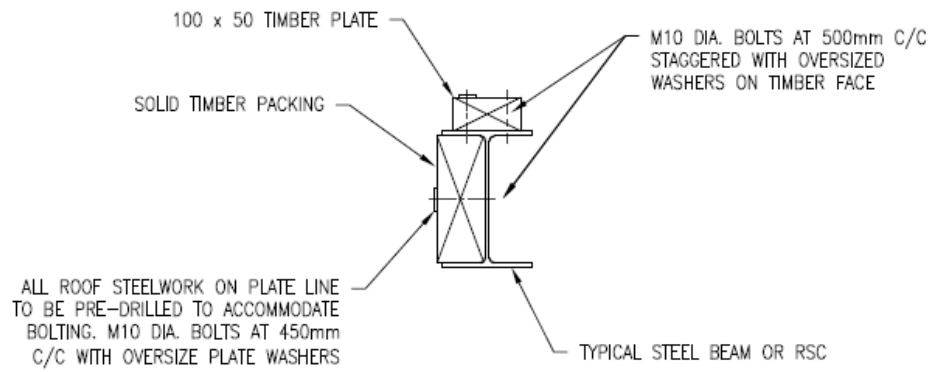
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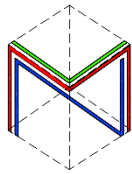
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TYPICAL PLATE TO STEEL CONNECTION
(AT FLOOR OR ROOF LEVEL)



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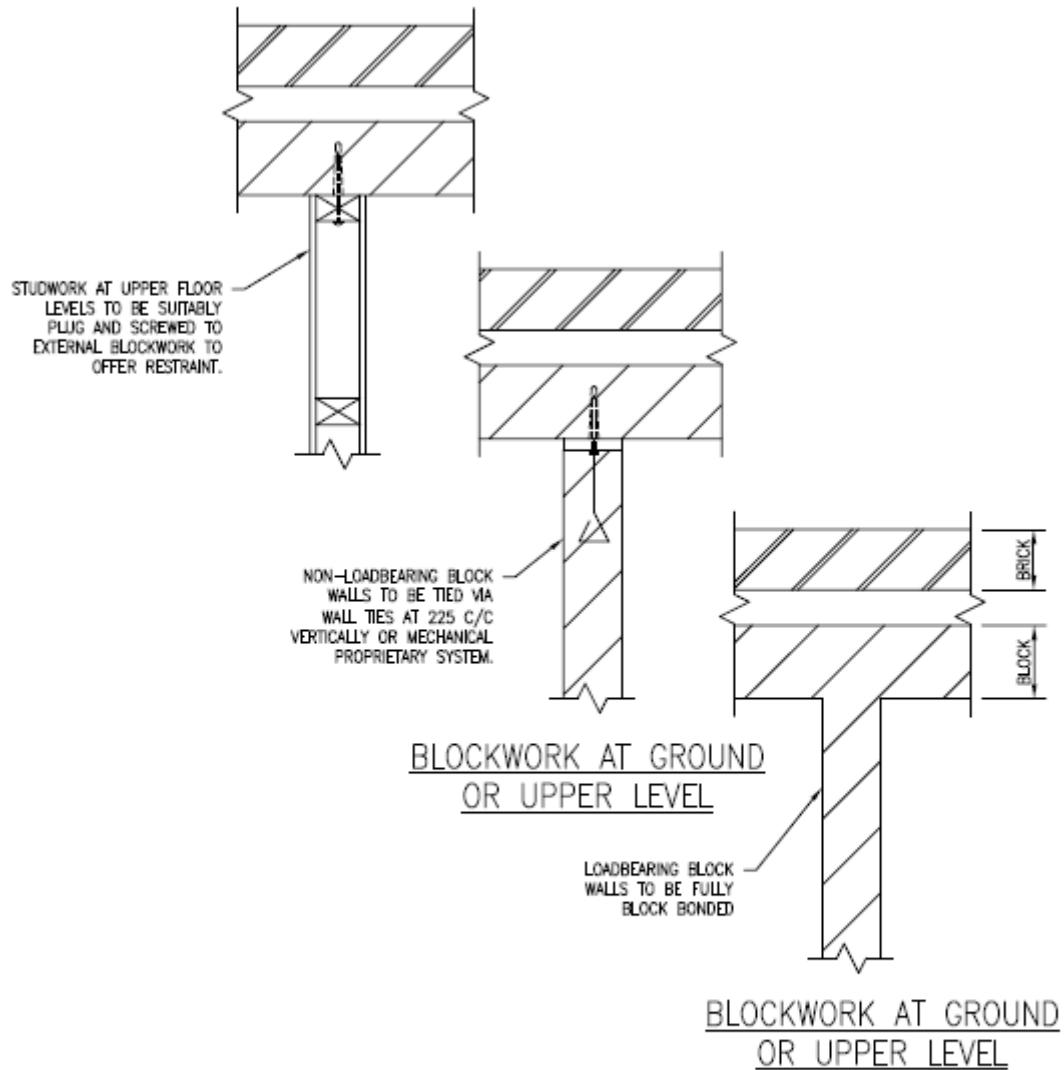
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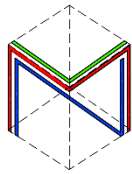
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TYPICAL INTERNAL TO EXTERNAL BLOCKWORK
JOINTING AND WALL RESTRAINT



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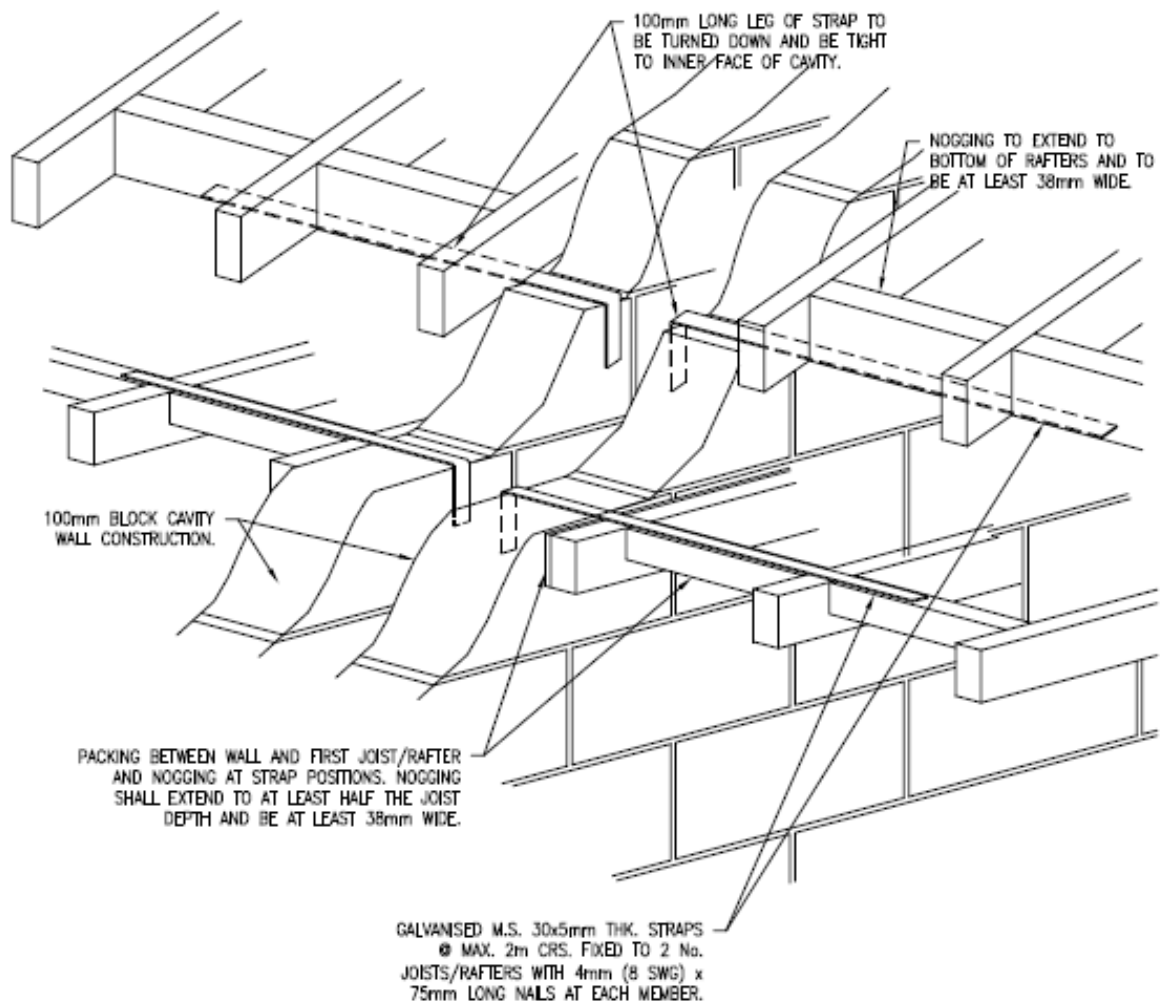
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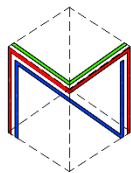
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LATERAL SUPPORT TO PARTY WALL
AT ROOF LEVEL
N.T.S.



**MVN
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DESIGN**

Job Title

11 Banfield Gardens, Henfield

Date

Dec 2025

Item

STANDARD DETAILS

Job No.

M1256

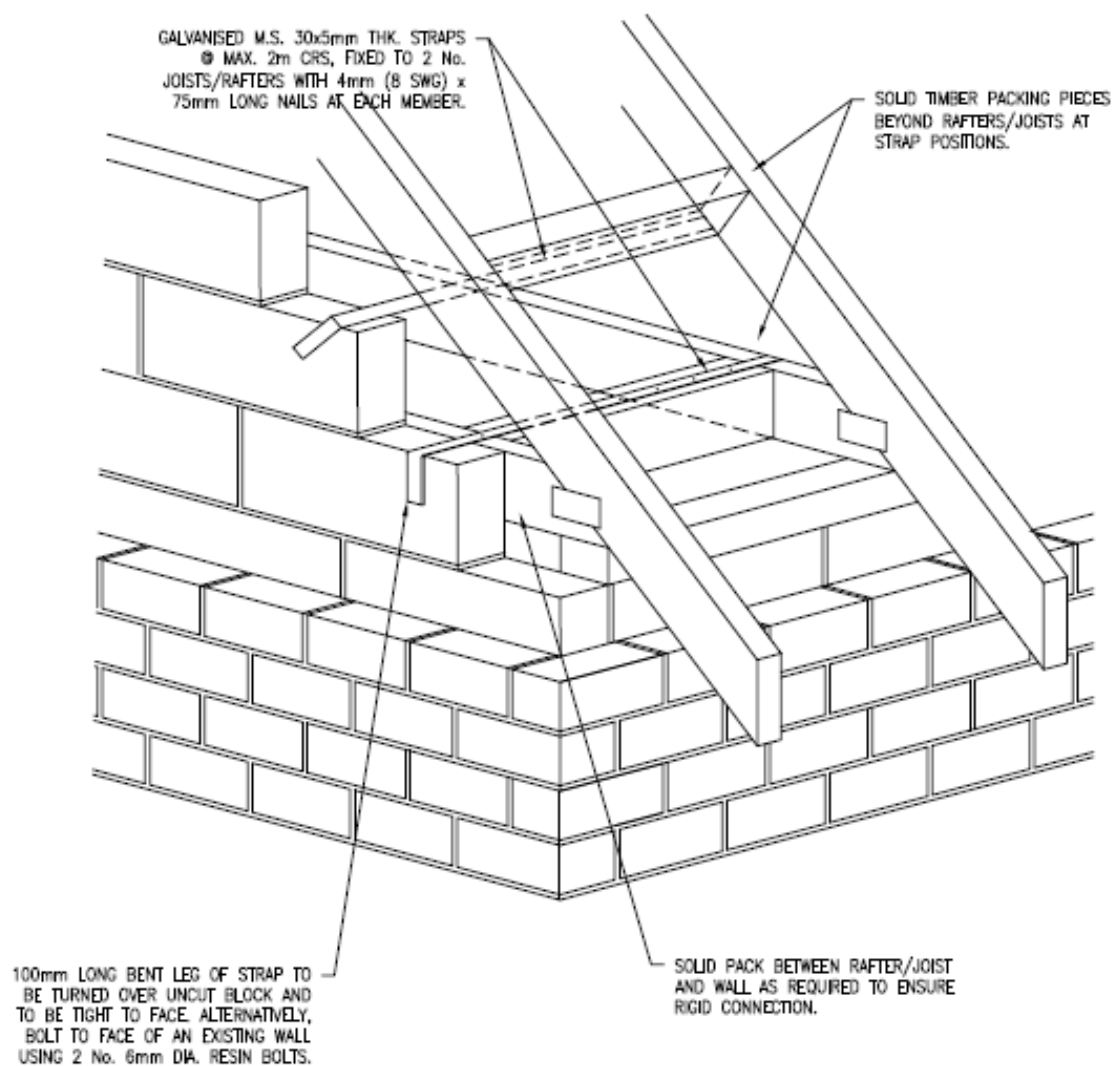
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STD-5

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**LATERAL SUPPORT TO BRICK/BLOCK GABLE CAVITY WALL
AT ROOF LEVEL**

N.T.S.