

# ***MARSH DESIGN LIMITED***

***STRUCTURAL DESIGN CONSULTANTS & SURVEY REPORTS***

***The Stable Block, Buckden Mount***

***8, Thornhill Road***

***Edgerton***

***Huddersfield***

***HD3 3AU***

***Company No: 457 2796***

***01484 - 341426***

**STRUCTURAL CALCULATIONS**

**FOR**

**INTERNAL LAYOUT ALTERATIONS**

**FOR**

**ANTHONY BENNETT**

**WOOLDALE METHODIST CHURCH**

**WOOLDALE ROAD**

**WOOLDALE**

**HD9 1QJ**

**OUR REF: MDL – 9098**

**REV.01: Issued for Building Control Approval Only**

**27<sup>th</sup> June 2024**

### **Structural Notes:**

- 1. These calculations are issued for building control approval only. Any structural construction works undertaken prior to approval being obtained are done so at risk by the client / contractor.**
- 2. Refer to Architect's drawings for setting out. Marsh Design's drawings are to be read in conjunction with all relevant Architects, subcontractors and engineers drawings and specifications. Final co-ordination of cladding, drainage, insulation, steelwork, and other elements is the responsibility of the contractor.**
- 3. If any queries arise on site or if there are any doubts, then ask the structural engineer immediately for further guidance.**
- 4. The sketches and marked up drawings contained within these structural calculations are NOT setting out or construction drawings, and they must NOT be used for setting out of steelwork or foundations. Existing structure to be verified by the contractor and discrepancies reported immediately to Engineer.**
- 5. The client must appoint the Principal Designer and Principal Contractor and comply with CDM. Regulations and that the Health & Safety File is issued at tender stage: Marsh Design are not the Principal Designer. The Contractor must have a Construction Phase Plan in Place prior to beginning works. Where applicable the contractor is to notify the local H.S.E. area office of the works using form F10 (rev.) and a copy to be displayed on site.**
- 6. It is the client's responsibility to ensure that all the requirements of the Party Wall Act are met during entire construction period of the project.**
- 7. Structural integrity and stability of all retained masonry piers or walls is to be confirmed after all openings have been broken out. This is generally undertaken by the building control inspector, but in certain instances approval by the appointed structural engineer may be necessary.**
- 8. Imposed load design to BS EN 1991: Structure Designed to: Typical Domestic 1.5kN/m<sup>2</sup>**
- 9. Contractor design and detail Items covers:**

- 9.1. **Hidden Steelwork connections, including those to foundations. Secondary Steelwork; balustrades, handrails, ladders plant decks, plant support and hanging etc.**
- 9.2. **Temporary works , scaffolds , trench & soil shoring etc**
- 9.3. **Glass, Glazing and curtain walling**
- 9.4. **Specialist propriety design items; Precast concrete (lintels, Beam and blocks), metal decks, cold rolled steel sections.**
- 9.5. **Timber trusses and timber frames**
- 9.6. **Other additional items identified separately on drawings**
10. **Manufacturer's details, requirements and installations methods to be used on Proprietary products. Approval of the CA must be sort when similar products are proposed by contractor**

### Concrete

11. **Concrete to be in accordance with BS EN 1992.**
  - 11.1. **Mass concrete foundations Concrete to be To FND 3 in accordance with BS8500 (minimum strength 30N/mm<sup>2</sup>, 20mm maximum aggregate size and 75mm slump).**

### Steelwork

12. **Structural steelwork to be in accordance with BS EN 10365; BS5950 for design detail and workmanship. Steelwork must be fabricated in accordance with BS EN 1090. All structural work and fire protection to the satisfaction of the Building Control Officer.**
  - 12.1. **All steel work must be minimum grade BS EN 10025 S355**
13. **Steel Corrosion Protection required to all steelwork**
  - 13.1. **Internal steelwork to be painted in accordance to BS EN ISO 12944: prepared by grit blasting in accordance with BS7079, the standard of surface cleanliness is to Swedish Standard SA2.5. Paint specification to be in accordance with BS5493. In shop applied high build Red zinc phosphate modified alkyd, to 75 microns. On site, degrease and touch up as necessary using high build zinc phosphate modified alkyd to 60 microns. Thicknesses are dry film thicknesses.**
  - 13.2. **External or Steelwork built into cavity or external solid walls to be galvanised in accordance to EN ISO 1461 & BS EN ISO 14713 with a minimum 120µm thickness achieving 460gms/m<sup>2</sup>. Site repairs to galvanising to be completed with Cold Galvafroid or similar.**
  - 13.3. **Concrete or masonry Encased steelwork to have 2 additional site coats of bitumen paint.**

#### **14. Steelwork Fabrication**

- 14.1. Contractor MUST provide fabrication drawings & connection calculations to the engineers two weeks prior to fabrication for approval, final appearance to be agreed with the architect. Fabricated Steelwork must be provided with a CE Mark, FPC, RWC and WQMS.**
- 14.2. Unless noted otherwise, steelwork welds to be minimum 6mm - maximum 12mm fillet welds, Penetration butt welds to be full depth and ground flush. All bolts to be grade 8.8 and Galvanised with minimum 16mm diameter. Welding to comply with BS EN 288. Site welding if essential to be tested in accordance with the national steelwork specification, site testing of all welds to be completed and results provided to the engineer.**
- 14.3. Overall lengths & Connection design is the responsibility of the contractor and details where shown are indicative.**
- 14.4. Connection Calculations & Fabrication details are to be provided by fabricator to the Engineer prior to fabrication for connection approval and to the Architect for setting out approval. Where loads are shown on the drawings, V = Factored Shear loads, M = Factored Moments. Minimum 2xM16 per connection and take 75kN tie force, 80kN shear unless noted otherwise.**
- 14.5. Bolt all double beams together with M16 at 600c/c with Spacer tubes. Where columns sit against masonry bolt back with M16 Resin Anchors at 450c/c staggered either side of flange.**

#### **Timber**

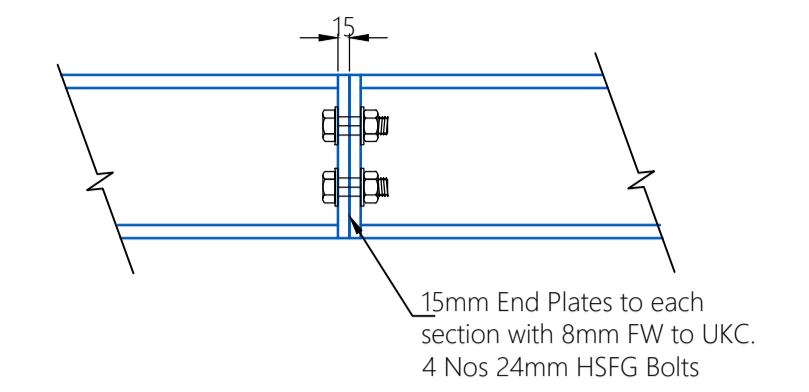
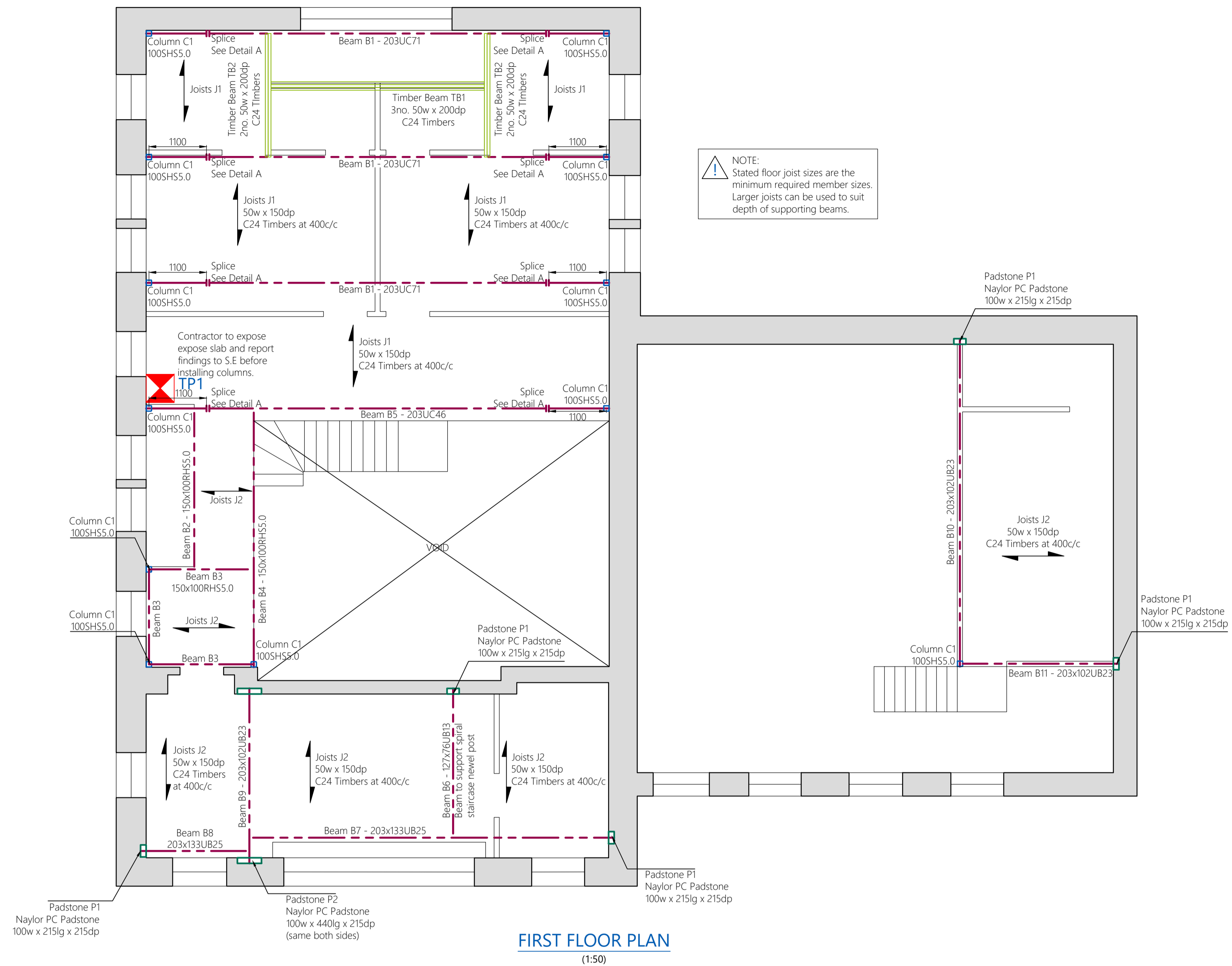
- 15. Structural timber to be strength class C24, unless noted otherwise, in accordance with EN 1995. All new timber in works to be tanalised with cut ends treated before fixing.**
- 16. New timber connections to have proprietary galvanised steel fixings in accordance to BS EN ISO 1461.**
- 17. Timber to masonry connections by joist hangers unless shown otherwise. Noggings, minimum 38mm width to extend at least 1/2 the depth of the joist, spaced at 1/3 points along joist span.**
- 18. Double up timber joists under all new partition walls and velux windows. All double & multiple timbers to be bolted with M10 at 450c/c.**

#### **Masonry**

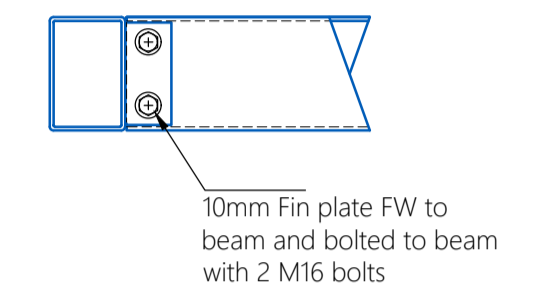
- 19. Masonry to be in accordance with EN 1996, Class (ii) above DPC and Class (i) to be used below DPC's and to chimneys. Below DPCs, all masonry to be Frost resistant. Block work below DPC to be sulphate resistant. Block Stresses: single Storey 7.3N/mm<sup>2</sup>, 2-3 stories 10N/mm<sup>2</sup> UNO**
- 20. All beams must be supported on reinforced concrete padstones unless noted otherwise in the calculations.**
- 21. Dry packing to be to be 2:1 Sand:Cement and to contain Fosroc CBex 100. Dry pack to be well rammed in. 48 hours to be left from drypacking to removal of any temporary supports.**
- 22. Existing masonry to be inspected. Where cracked or debonded, repairs are as specified. If repairs are not specified contact the engineer. Existing walls to be checked for lateral restraint: Install restraint straps (1200x30x3) at 1200centres to all floors.**
- 23. Provide Lateral Restraint straps (1200x30x3) at 1200centres to floors and roof. Provide Holding Down straps (1200x30x3) at 1200centres to roof sole plate.**
- 24. Existing lintels to be inspected and replaced if showing signs of deterioration.**

#### **Temporary Works and Sequencing**

- 25. Marsh Design have designed for the final permanent condition only. The main contractor is to be responsible for the temporary stability, Design, Detailing and Sequencing of existing structures and earthworks on the site and adjoining sites, and must take all necessary precautions to safeguard this stability.**
- 26. The contractor must follow the guidelines of BS5975. A Suitably qualified Temporary works Engineers employed by the contractor must design all temporary works.**
- 27. Details of propping/needling, with supporting calculations completed by a suitability qualified person & method statement to be provided to Engineer prior to commencing works for approval.**
  - 27.1. All needles to be design for Span/500 vertically deflection and height over 750 horizontal deflections.**
  - 27.2. All props to be founded on firm good bearing strata.**



**Detail A: 203UC71 Splice**  
1:10  
Moment 45kNm  
Shear 35kN



**150x100RH55.0 Simple Connection**  
Shear = 10kN  
Final Design by Fabricator

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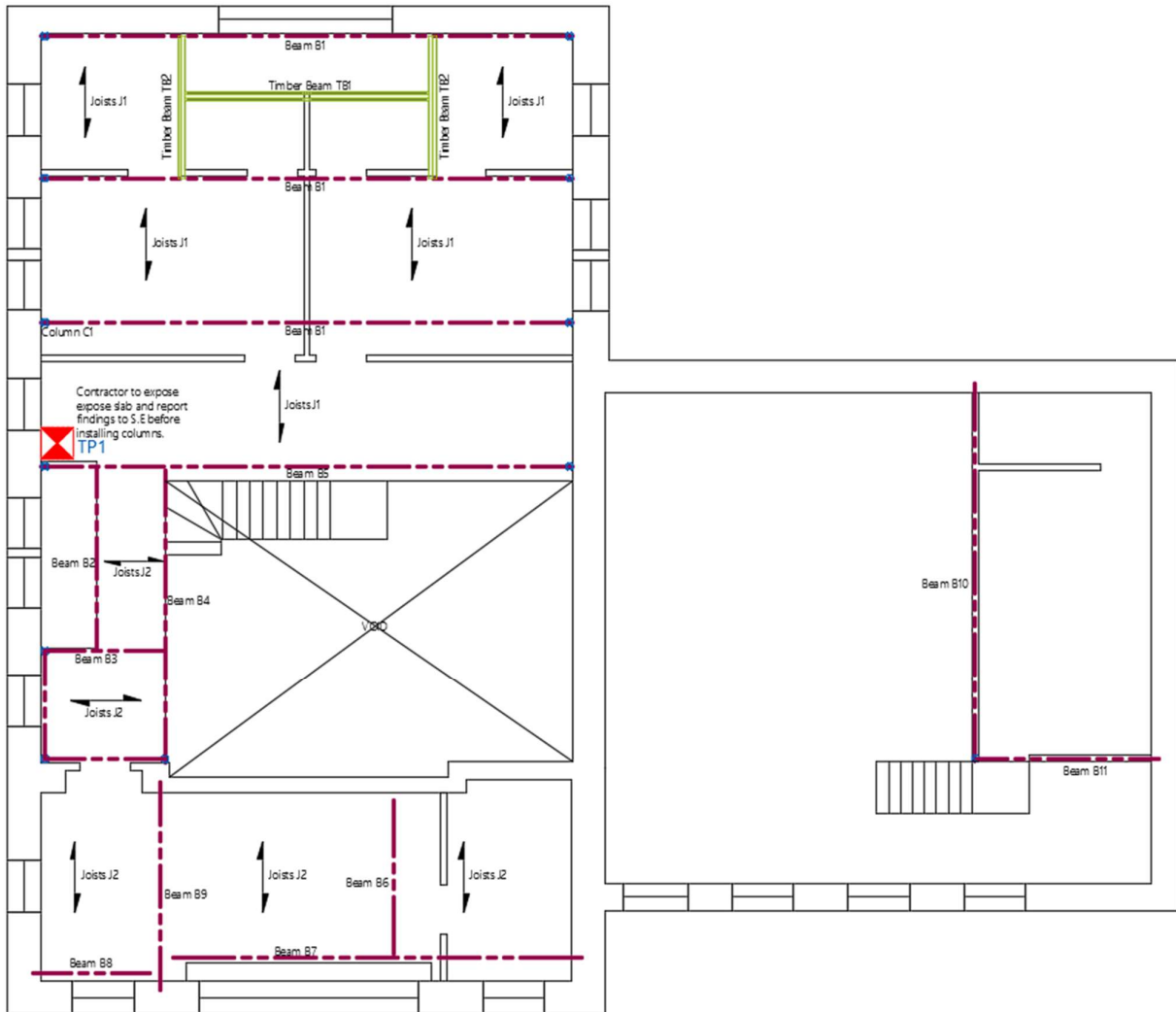
Rev	Date	By	First Issue
-	24/06/24	SC	First Issue

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**Anthony Bennett**  
Wooldale Methodist Church  
Structural Plans  
Issued for **INFORMATION ONLY**

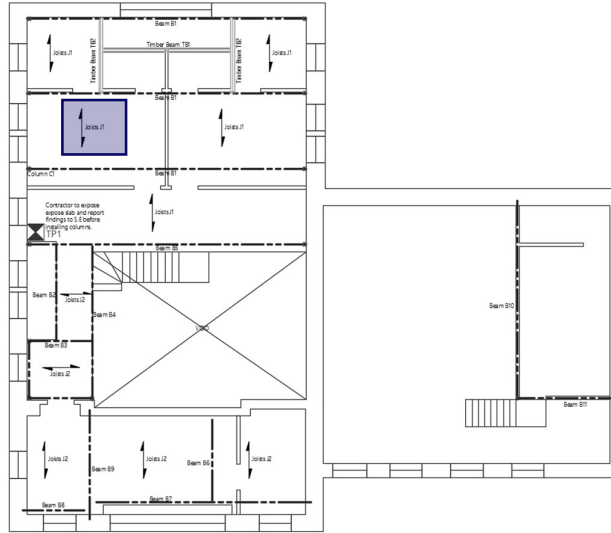
Job Number <b>9098</b>	Dwg Number <b>SL-10</b>	Rev -
By <b>SC</b>	Approved by <b>DH</b>	Scale <b>1:50 @A1</b>

### MEMBER LOCATION PLAN



FIRST FLOOR PLAN  
(1:50)

**JOISTS J1**

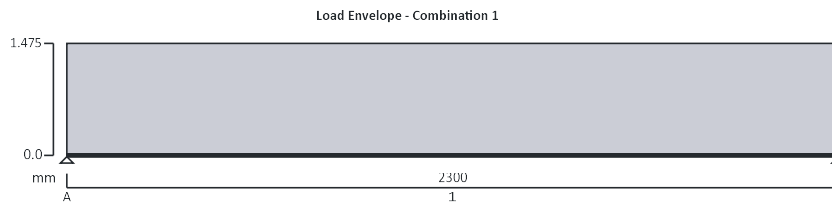


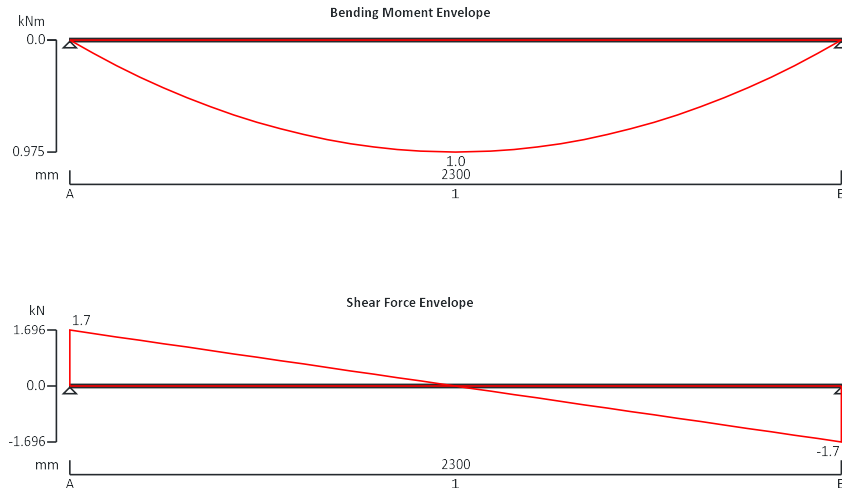
Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m				
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total	
<b>Joists J1</b>											
Floor	0.4	1	0.4	g <sub>k</sub>		0.88	0.4				
				q <sub>k</sub>		1.50			0.6		
							0.4	kN/m	0.6	kN/m	

**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  $\times 1$

Permanent full UDL 0.400 kN/m

Variable full UDL 0.600 kN/m

### Load combinations

Load combination 1

Support A

Permanent  $\times 1.35$

Variable  $\times 1.50$

Span 1

Permanent  $\times 1.35$

Variable  $\times 1.50$

Support B

Permanent  $\times 1.35$

Variable  $\times 1.50$

### Analysis results

Design moment

$M = 0.975$  kNm

Design shear

$F = 1.696$  kN

Total load on member

$W_{tot} = 3.392$  kN

Reactions at support A

$R_{A,max} = 1.696$  kN

$R_{A,min} = 1.696$  kN

Unfactored permanent load reaction at support A

$R_{A,Permanent} = 0.490$  kN

Unfactored variable load reaction at support A

$R_{A,Variable} = 0.690$  kN

Reactions at support B

$R_{B,max} = 1.696$  kN

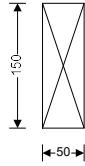
$R_{B,min} = 1.696$  kN

Unfactored permanent load reaction at support B

$R_{B,Permanent} = 0.490$  kN

Unfactored variable load reaction at support B

$R_{B,Variable} = 0.690$  kN

**Timber section details**

Breadth of section

 $b = 50$  mm

Depth of section

 $h = 150$  mm

Number of sections

 $N = 1$ 

Breadth of member

 $b_b = 50$  mm

Timber strength class

**C24****Member details**

Service class of timber

**1**

Load duration

**Long-term**

Length of span

 $L_{s1} = 2300$  mm

Length of bearing

 $L_b = 100$  mm**Compression perpendicular to grain - cl.6.1.4**Design compressive stress  
N/mm<sup>2</sup> $\sigma_{c,90,d} = 0.339$  N/mm<sup>2</sup>

Design compressive strength

 $f_{c,90,d} = 1.346$ **PASS - Design compressive strength exceeds design compressive stress at bearing****Bending - cl 6.1.6**

Design bending stress

 $\sigma_{m,d} = 5.201$  N/mm<sup>2</sup>Design bending strength  $f_{m,d} = 12.923$  N/mm<sup>2</sup>**PASS - Design bending strength exceeds design bending stress****Shear - cl.6.1.7**

Applied shear stress

 $\tau_d = 0.506$  N/mm<sup>2</sup>

Permissible shear stress

 $f_{v,d} = 2.154$  N/mm<sup>2</sup>**PASS - Design shear strength exceeds design shear stress****Deflection - cl.7.2**

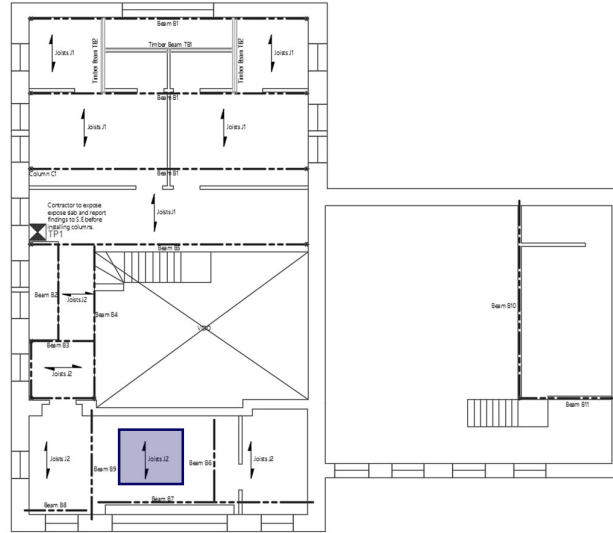
Deflection limit

 $\delta_{im} = 9.200$  mm

Total final deflection

 $\delta_{fin} = 3.485$  mm**PASS - Total final deflection is less than the deflection limit**

**JOISTS J2**

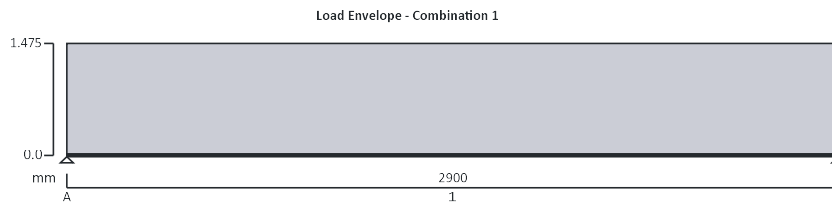


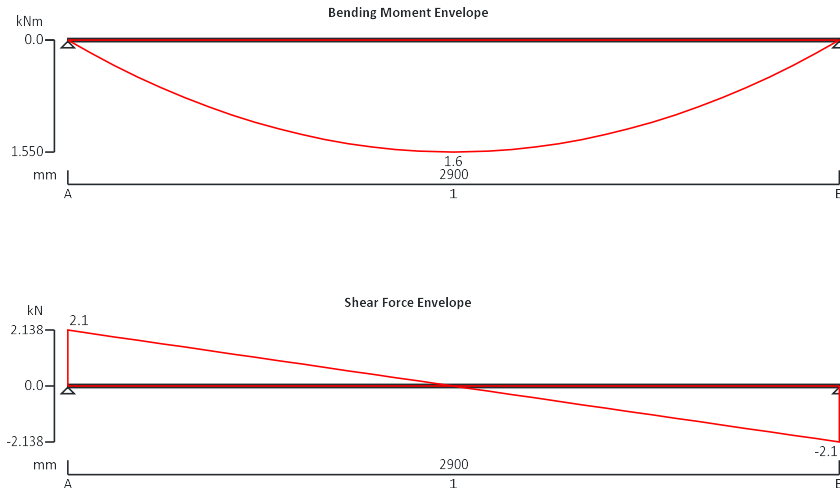
Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m				
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total	
<b>Joists J2</b>											
Floor	0.4	1	0.4	g <sub>k</sub>		0.88	0.4				
				q <sub>k</sub>		1.50			0.6		
							0.4	kN/m	0.6	kN/m	

**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

Teds calculation version 1.7.05





### Applied loading

#### Beam loads

Permanent self weight of beam  $\times 1$

Permanent full UDL 0.400 kN/m

Variable full UDL 0.600 kN/m

#### Load combinations

Load combination 1

Support A

Permanent  $\times 1.35$

Variable  $\times 1.50$

Span 1

Permanent  $\times 1.35$

Variable  $\times 1.50$

Support B

Permanent  $\times 1.35$

Variable  $\times 1.50$

#### Analysis results

Design moment

$M = 1.550$  kNm

Design shear

$F = 2.138$  kN

Total load on member

$W_{tot} = 4.277$  kN

Reactions at support A

$R_{A,max} = 2.138$  kN

$R_{A,min} = 2.138$  kN

Unfactored permanent load reaction at support A

$R_{A,Permanent} = 0.617$  kN

Unfactored variable load reaction at support A

$R_{A,Variable} = 0.870$  kN

Reactions at support B

$R_{B,max} = 2.138$  kN

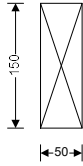
$R_{B,min} = 2.138$  kN

Unfactored permanent load reaction at support B

$R_{B,Permanent} = 0.617$  kN

Unfactored variable load reaction at support B

$R_{B,Variable} = 0.870$  kN

**Timber section details**

Breadth of section	$b = 50$ mm	Depth of section	$h = 150$ mm
Number of sections	$N = 1$	Breadth of member	$b_b = 50$ mm
Timber strength class	<b>C24</b>		

**Member details**

Service class of timber	<b>1</b>	Load duration	<b>Long-term</b>
Length of span	$L_{s1} = 2900$ mm		
Length of bearing	$L_b = 100$ mm		

**Compression perpendicular to grain - cl.6.1.4**

Design compressive stress $\sigma_{c,90,d}$	$= 0.428$ N/mm <sup>2</sup>	Design compressive strength $f_{c,90,d}$	$= 1.346$ N/mm <sup>2</sup>
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**PASS - Design compressive strength exceeds design compressive stress at bearing**

**Bending - cl 6.1.6**

Design bending stress $\sigma_{m,d}$	$= 8.268$ N/mm <sup>2</sup>	Design bending strength $f_{m,d}$	$= 12.923$ N/mm <sup>2</sup>
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**PASS - Design bending strength exceeds design bending stress**

**Shear - cl.6.1.7**

Applied shear stress $\tau_d$	$= 0.638$ N/mm <sup>2</sup>	Permissible shear stress $f_{v,d}$	$= 2.154$ N/mm <sup>2</sup>
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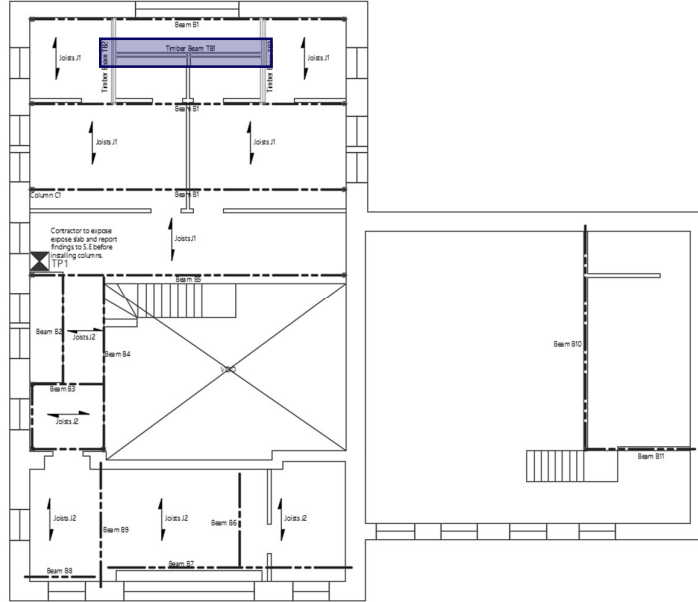
**PASS - Design shear strength exceeds design shear stress**

**Deflection - cl.7.2**

Deflection limit $\delta_{lim}$	$= 11.600$ mm	Total final deflection $\delta_{fin}$	$= 8.609$ mm
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**PASS - Total final deflection is less than the deflection limit**

**TIMBER BEAM TB 1**

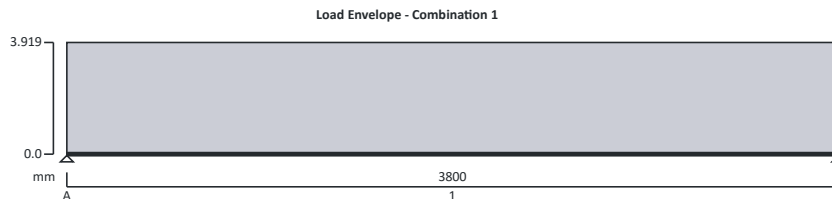


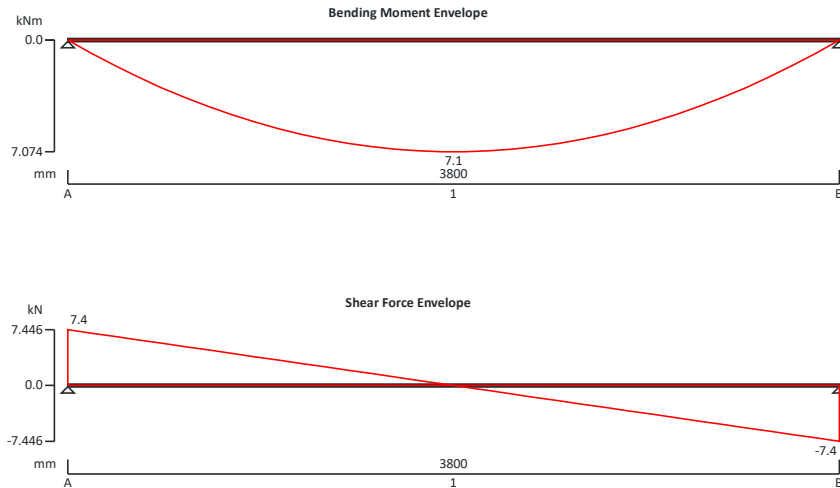
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<b>Timber Beam TB1</b>										
Floor	0.6	1	0.6	g <sub>k</sub>		0.88	0.5			
				q <sub>k</sub>		1.50			0.9	
Partition	2.5	1	2.5	g <sub>k</sub>		0.52	1.3			
							1.8	kN/m	0.9	kN/m

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Tedds calculation version 1.7.05





## Applied loading

### Beam loads

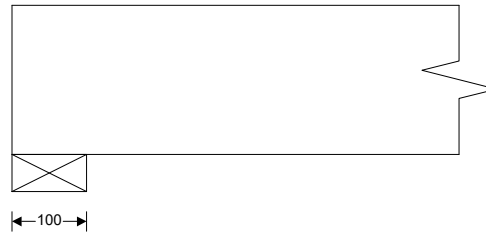
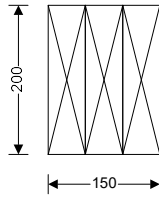
Permanent self weight of beam  $\times$  1  
 Permanent full UDL 1.800 kN/m  
 Variable full UDL 0.900 kN/m

### Load combinations

Load combination 1	Support A	Permanent $\times$ 1.35 Variable $\times$ 1.50
	Span 1	Permanent $\times$ 1.35 Variable $\times$ 1.50
	Support B	Permanent $\times$ 1.35 Variable $\times$ 1.50

### Analysis results

Design moment	$M = 7.074$ kNm	Design shear	$F = 7.446$ kN
Total load on member	$W_{tot} = 14.892$ kN		
Reactions at support A	$R_{A,max} = 7.446$ kN	$R_{A,min} = 7.446$ kN	
Unfactored permanent load reaction at support A			$R_{A,Permanent} = 3.616$ kN
Unfactored variable load reaction at support A			$R_{A,Variable} = 1.710$ kN
Reactions at support B	$R_{B,max} = 7.446$ kN	$R_{B,min} = 7.446$ kN	
Unfactored permanent load reaction at support B			$R_{B,Permanent} = 3.616$ kN
Unfactored variable load reaction at support B			$R_{B,Variable} = 1.710$ kN

**Timber section details**

Breadth of section	$b = 50$ mm	Depth of section	$h = 200$ mm
Number of sections	$N = 3$	Breadth of member	$b_b = 150$ mm
Timber strength class	<b>C24</b>		

**Member details**

Service class of timber	<b>1</b>	Load duration	<b>Long-term</b>
Length of span	$L_{s1} = 3800$ mm		
Length of bearing	$L_b = 100$ mm		

**Compression perpendicular to grain - cl.6.1.4**

Design compressive stress $\sigma_{c,90,d}$	$= 0.496$ N/mm <sup>2</sup>	Design compressive strength $f_{c,90,d}$	$= 1.346$ N/mm <sup>2</sup>
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**PASS - Design compressive strength exceeds design compressive stress at bearing**

**Bending - cl 6.1.6**

Design bending stress $\sigma_{m,d}$	$= 7.074$ N/mm <sup>2</sup>	Design bending strength $f_{m,d}$	$= 12.923$ N/mm <sup>2</sup>
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**PASS - Design bending strength exceeds design bending stress**

**Shear - cl.6.1.7**

Applied shear stress $\tau_d$	$= 0.556$ N/mm <sup>2</sup>	Permissible shear stress $f_{v,d}$	$= 2.154$ N/mm <sup>2</sup>
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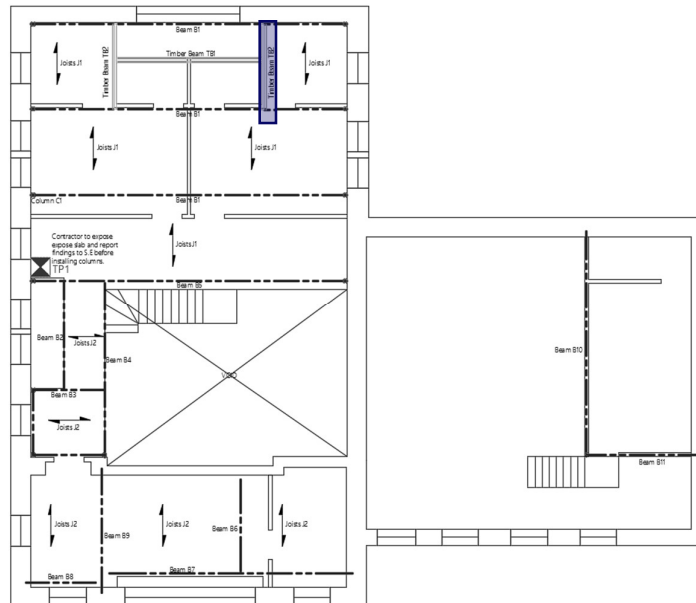
**PASS - Design shear strength exceeds design shear stress**

**Deflection - cl.7.2**

Deflection limit $\delta_{lim}$	$= 14.000$ mm	Total final deflection $\delta_{fin}$	$= 10.566$ mm
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**PASS - Total final deflection is less than the deflection limit**

## TIMBER BEAM TB2

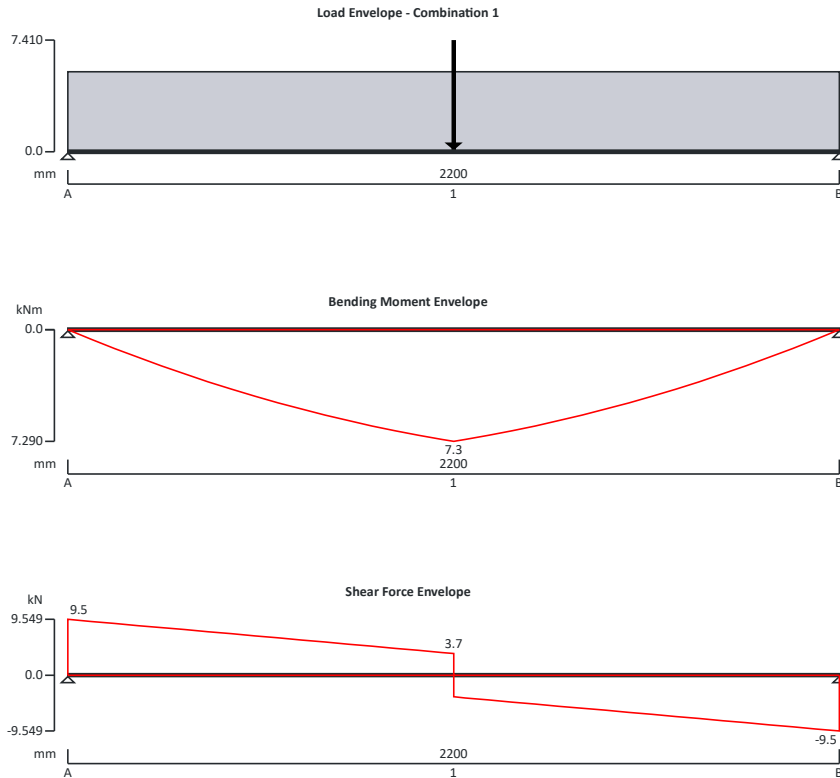


Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Timber Beam TB2</b>										
Floor	1	1	1	g <sub>k</sub>		0.88	0.9			
				q <sub>k</sub>		1.50			1.5	
Partition	2.5	1	2.5	g <sub>k</sub>		0.52	1.3			
							2.2	kN/m	1.5	kN/m
Timber Beam TB1				g <sub>k</sub>	3.6	kN				
				q <sub>k</sub>	1.7	kN				

### **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  $\times 1$   
 Permanent full UDL 2.200 kN/m  
 Variable full UDL 1.500 kN/m  
 Permanent point load 3.600 kN at 1100 mm  
 Variable point load 1.700 kN at 1100 mm

### Load combinations

Load combination 1

Support A	Permanent $\times 1.35$ Variable $\times 1.50$
Span 1	Permanent $\times 1.35$ Variable $\times 1.50$
Support B	Permanent $\times 1.35$ Variable $\times 1.50$

### Analysis results

Design moment

$M = 7.290$  kNm

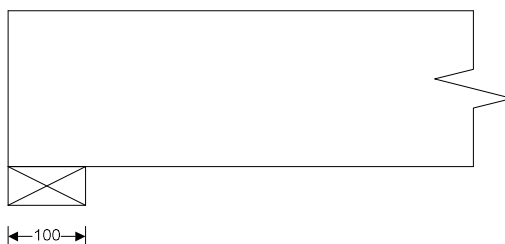
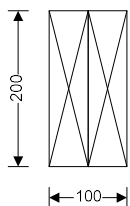
Design shear

$F = 9.549$  kN

Total load on member

$W_{\text{tot}} = 19.098$  kN

Reactions at support A	$R_{A\_max} = 9.549$ kN	$R_{A\_min} = 9.549$ kN	
Unfactored permanent load reaction at support A			$R_{A\_Permanent} = 4.296$ kN
Unfactored variable load reaction at support A			$R_{A\_Variable} = 2.500$ kN
Reactions at support B	$R_{B\_max} = 9.549$ kN	$R_{B\_min} = 9.549$ kN	
Unfactored permanent load reaction at support B			$R_{B\_Permanent} = 4.296$ kN
Unfactored variable load reaction at support B			$R_{B\_Variable} = 2.500$ kN

**Timber section details**

Breadth of section	$b = 50$ mm	Depth of section	$h = 200$ mm
Number of sections	$N = 2$	Breadth of member	$b_b = 100$ mm
Timber strength class	<b>C24</b>		

**Member details**

Service class of timber	<b>1</b>	Load duration	<b>Long-term</b>
Length of span	$L_{s1} = 2200$ mm		
Length of bearing	$L_b = 100$ mm		

**Compression perpendicular to grain - cl.6.1.4**

Design compressive stress $\sigma_{c,90,d}$	$0.955$ N/mm <sup>2</sup>	Design compressive strength $f_{c,90,d}$	$1.346$ N/mm <sup>2</sup>
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**PASS - Design compressive strength exceeds design compressive stress at bearing**

**Bending - cl 6.1.6**

Design bending stress $\sigma_{m,d}$	$10.935$ N/mm <sup>2</sup>	Design bending strength $f_{m,d}$	$12.923$ N/mm <sup>2</sup>
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**PASS - Design bending strength exceeds design bending stress**

**Shear - cl.6.1.7**

Applied shear stress $\tau_d$	$1.069$ N/mm <sup>2</sup>	Permissible shear stress $f_{v,d}$	$2.154$ N/mm <sup>2</sup>
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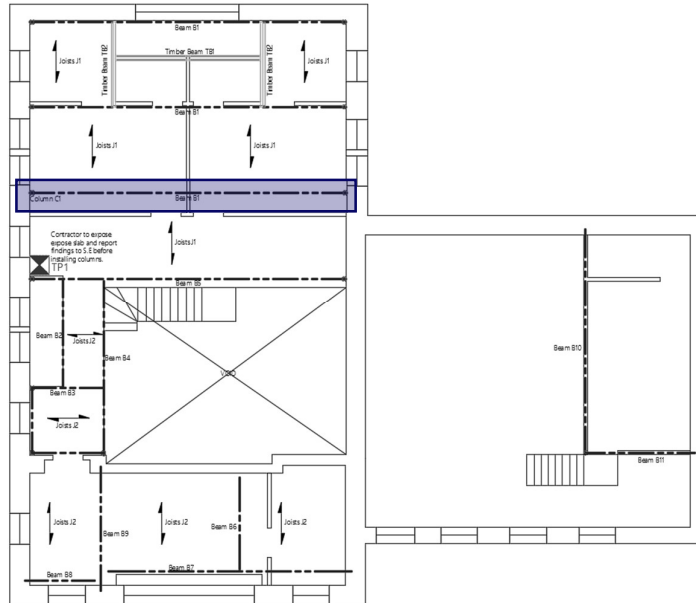
**PASS - Design shear strength exceeds design shear stress**

**Deflection - cl.7.2**

Deflection limit $\delta_{lim}$	$8.800$ mm	Total final deflection $\delta_{fin}$	$5.251$ mm
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**PASS - Total final deflection is less than the deflection limit**

**BEAM B1**

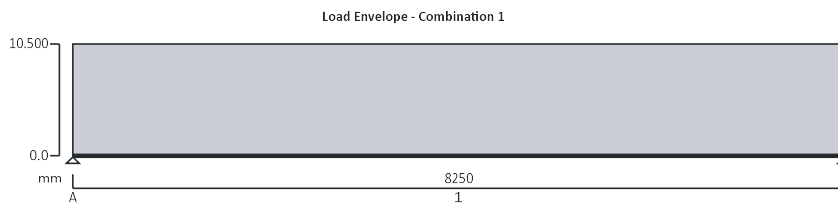


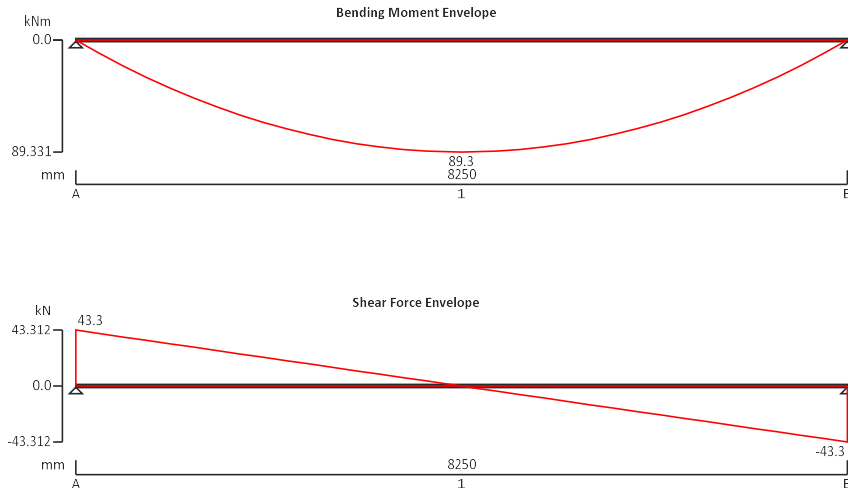
Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B1</b>										
Floor	2.25	1	2.25	g <sub>k</sub>		0.88	2.0			
				q <sub>k</sub>		1.50			3.4	
Timber partitions	2.5	1	2.5	g <sub>k</sub>		0.52	1.3			
							3.3	kN/m	3.4	kN/m

**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 3.0.14



**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

Permanent full UDL 3.3 kN/m

Variable full UDL 3.4 kN/m

**Load combinations**

Load combination 1

Support A

Permanent  $\times$  1.35Variable  $\times$  1.50Permanent  $\times$  1.35Variable  $\times$  1.50

Support B

Permanent  $\times$  1.35Variable  $\times$  1.50**Analysis results**

Maximum moment

 $M_{\max} = 89.3$  kNm $M_{\min} = 0$  kNm

Maximum shear

 $V_{\max} = 43.3$  kN $V_{\min} = -43.3$  kN

Deflection

 $\delta_{\max} = 27.8$  mm $\delta_{\min} = 0$  mm

Maximum reaction at support A

 $R_{A_{\max}} = 43.3$  kN $R_{A_{\min}} = 43.3$  kN

Unfactored permanent load reaction at support A

 $R_{A_{\text{Permanent}}} = 16.5$  kN

Unfactored variable load reaction at support A

 $R_{A_{\text{Variable}}} = 14$  kN

Maximum reaction at support B

 $R_{B_{\max}} = 43.3$  kN $R_{B_{\min}} = 43.3$  kN

Unfactored permanent load reaction at support B

 $R_{B_{\text{Permanent}}} = 16.5$  kN

Unfactored variable load reaction at support B

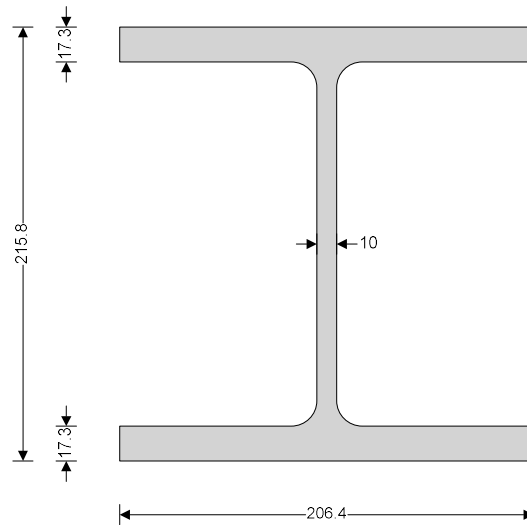
 $R_{B_{\text{Variable}}} = 14$  kN

**Section details**

Section type

**UC 203x203x71 (British Steel Section Range 2022 (BS4-1))**

Steel grade

**S355**

Section classification

**Class 1****Check shear - Section 6.2.6**

Design shear force

 $V_{Ed} = 43$  kNDesign shear resistance  $V_{c,Rd} = 510.5$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**

Design bending moment

 $M_{Ed} = 89.3$  kNm

Des. bending resist. moment

 $M_{c,Rd} = 277$  kNm**Slenderness ratio for lateral torsional buckling**

LTB slenderness ratio

 $\bar{\lambda}_{LT} = 0.914$ 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

 $M_{b,Rd} = 214.6$  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

 $\delta_{lim} = 33$  mm

Maximum deflection

 $\delta = 27.766$  mm**PASS - Maximum deflection does not exceed deflection limit**

**BEAM B1 - SPLICE**

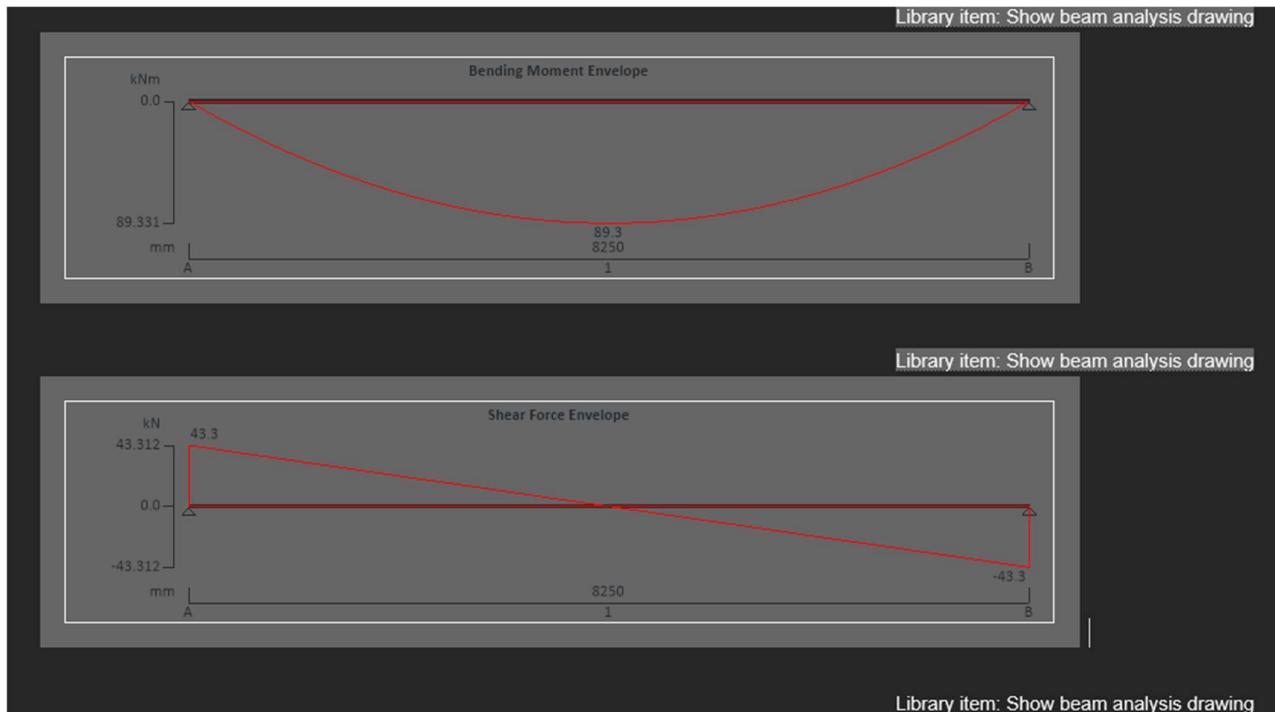
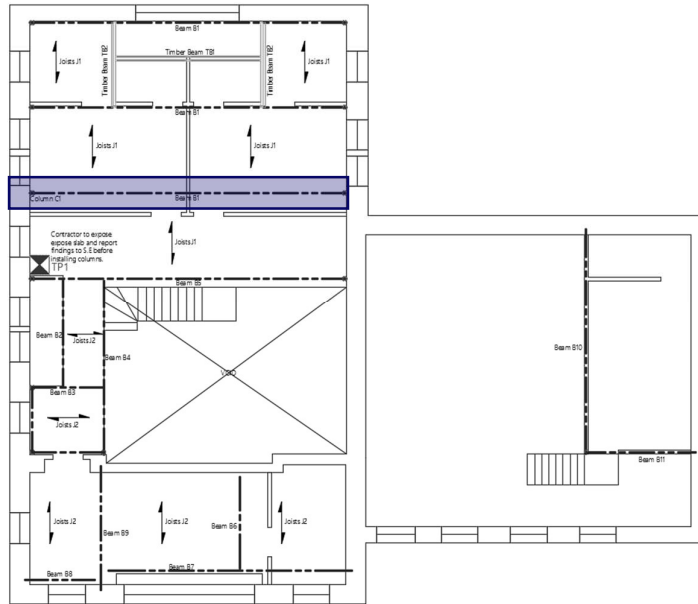


Figure 1 - Beam B1 Force Diagrams

**STEEL BEAM SPLICE DESIGN (EN1993)**

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

Teds calculation version 1.0.01

**Design summary**Overall design status **PASS**Overall design utilisation **0.823**

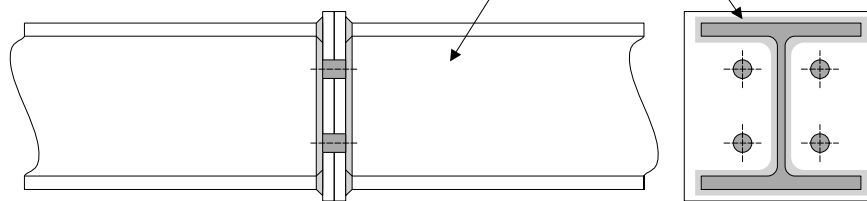
Description	Unit	Design	Resistance	Utilisation	Result
Moment resistance joint	kNm	41.2	50.0	0.823	PASS
Shear resist. bolt group	kN	32.3	347.0	0.093	PASS
Bearing resist. bolt group	kN	32.3	322.1	0.100	PASS
Tension flange weld	kN	340.0	1096.4	0.310	PASS
Web weld tension zone	kN/mm	1.45	3.34	0.435	PASS
Web weld shear zone	kN/mm	0.67	2.73	0.247	PASS
Comp. flange weld	mm	8.0	8.0	1.000	PASS

2No. M24 bolts in tension zone

2No. M24 bolts in compression zone

End plate

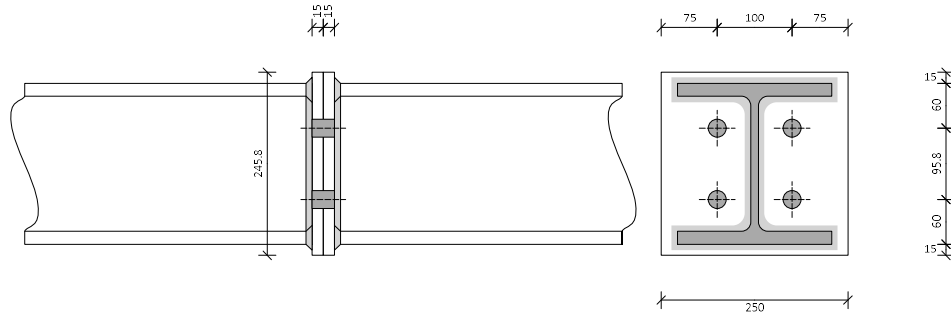
250 x 246

Beam sections:  
UC 203x203x71**End plate Splice****Design forces**Design moment  $M_{Ed} = 41.2$  kNmDesign shear force  $V_{Ed} = 32.3$  kNDesign axial force  $N_{Ed} = 0.0$  kN**Partial factors**Resistance of cross section instability  $\gamma_{M1} = 1.00$   $\gamma_{M0} = 1.00$  Resist. members toSections in tension to fracture  $\gamma_{M2} = 1.10$  Resistance of bolts $\gamma_{M2,b} = 1.25$ Slip resistance  $\gamma_{M3} = 1.25$  Slip resistance, servicability  $\gamma_{M3,ser} = 1.10$ Structural integrity  $\gamma_{M,u} = 1.10$ **Beam details**

Section name UC 203x203x71

Steel grade S355

Yield strength  $f_{y,bm} = 345$  N/mm<sup>2</sup>Ultimate strength  $f_{u,bm} = 470$  N/mm<sup>2</sup>Section height  $h = 215.8$  mmSection width  $b = 206.4$  mmFlange thickness  $t_f = 17.3$  mmWeb thickness  $t_w = 10.0$  mm

**End plate bolt details**Number of bolt columns  $n_{1,ep} = 2$ Number of upper bolt rows  
rows $n_{2,ep} = 1$  $n_{2u,ep} = 1$ 

Number of lower bolt

Bolt gauge  $p_{1,ep} = 100$  mm

Bolt pitch

 $p_{2,ep} = 50$  mmBolt size **M24**

Bolt grade

**8.8**Yield strength  $f_{yb,ep} = 640$  N/mm<sup>2</sup>

Ultimate strength

 $f_{ub,ep} = 800$  N/mm<sup>2</sup>Edge distance in plate  $e_{1,ep} = 75$  mm

Dist. int. bolt to face of flange

 $x_{2if,ep} = 60$  mm

Design tension resist. of bolt

 $F_{t,Rd} = 203.3$  kN**End plate details**Plate width  $b_{ep} = 250$  mm

Plate height

 $h_{ep} = 245.8$  mmPlate thickness  $t_{ep} = 15$  mm

Plate grade

**S355**Yield strength  $f_{y,ep} = 355$  N/mm<sup>2</sup>

Ultimate strength

 $f_{u,ep} = 470$  N/mm<sup>2</sup>**End plate bending resistance****Variables used for effective length (Figure 6.10)** $m_x = 30.4$  mm $m_2 = 33.1$  mm $w = 100$  mm $p = 50$  mm $e = 75$  mm $e_x = 50$  mm

Quarter of bolt washer dia.

 $e_w = 10$  mm**Bolt row 1 - other end tension flange of beam**

Design resist. T-stub - mode 1

 $F_{t,1,Rd,r1} = 532.1$  kN

Design resist. T-stub -

mode 2

 $F_{t,2,Rd,r1} = 340.0$  kN

Design resist. T-stub - mode 3

 $F_{t,3,Rd,r1} = 406.7$  kN

Resist of end plate to

bending

 $F_{t,ep,Rd,r1} = 340.0$  kN

Resist of beam web in tension

 $F_{t,wb,Rd,r1} = 717.5$  kN**Resistance of bolt row 1**Resistance of bolt row  $F_{t,Rd,r1} = 340.0$  kN

Job No.	Section Nos /Page No. /Revision	Calc By	SC	Calc Date
9098	/ 20			25/06/2024

### Minimum resistance of bolt rows

Effective resist. of bolt row 1  $F_{t,Rd,eff,r1} = 340.0$  kN

### Design resistance of beam flange and adjacent compression zone of web

Design bending resistance mnt  $M_{c,y,Rd} = 277.0$  kNm      Comp zone resist of flng & wb  $F_{c,fb,Rd} = 1395.7$  kN

### Lever arm of bolts from mid thickness of compression flange

#### Plastic distribution limit

Design tension resistance limit  $F_{t,Rd,lim} = 386.3$  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  $\Sigma F_{t,Rd} = 340.0$  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

Moment resistance of joint  $M_{j,Rd} = 50.0$  kNm

$$M_{Ed} / M_{j,Rd} = 0.823$$

**PASS - Moment resistance of joint greater than applied moment**

### Shear resistance of joint

Shear resistance of joint  $F_{v,ep,Rd} = 347.0$  kN

$$V_{Ed} / F_{v,ep,Rd} = 0.093$$

**PASS - Shear resistance of joint greater than applied shear**

### Bearing resistance of bolts

Bearing resistance of bolts  $F_{b,ep,Rd} = 322.1$  kN

$$V_{Ed} / F_{b,ep,Rd} = 0.100$$

**PASS - Bearing resistance of bolts greater than applied shear**

### Tension flange weld

Design force - local bolt forces  $F_{ff,w,Ed} = 340.0$  kN      Design resistance of weld  $F_{tn,w,Rd} = 1096.4$  kN

$$F_{ff,w,Ed} / F_{tn,w,Rd} = 0.310$$

**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  $F_{T,ww,Ed} = 1.454$  kN/mm      Design resistance of weld  $F_{T,ww,Rd} = 3.342$  kN/mm

$$F_{t,ww,Ed} / F_{t,ww,Rd} = 0.435$$

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

#### Web weld in shear zone

Tension force per unit length

$$F_{t,ww,Ed} = 0.675 \text{ kN/mm}$$

Design resistance of

weld

$$F_{t,ww,Rd} = 2.729 \text{ kN/mm}$$

$$F_{t,ww,Ed} / F_{t,ww,Rd} = 0.247$$

**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

$$S_{f,top} = 8.00 \text{ mm}$$

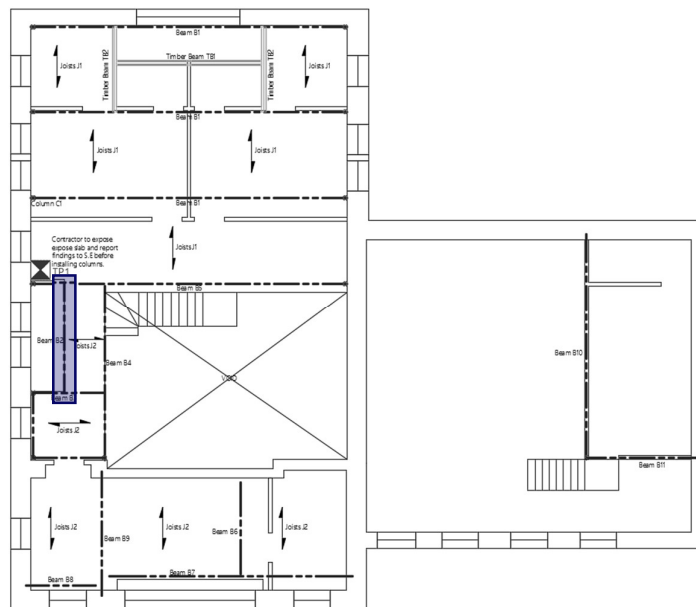
Leg length of weld required

$$S_{f,top,nom} = 8.00$$

mm

**PASS - Weld greater or equal to nominal weld size**

## BEAM B2

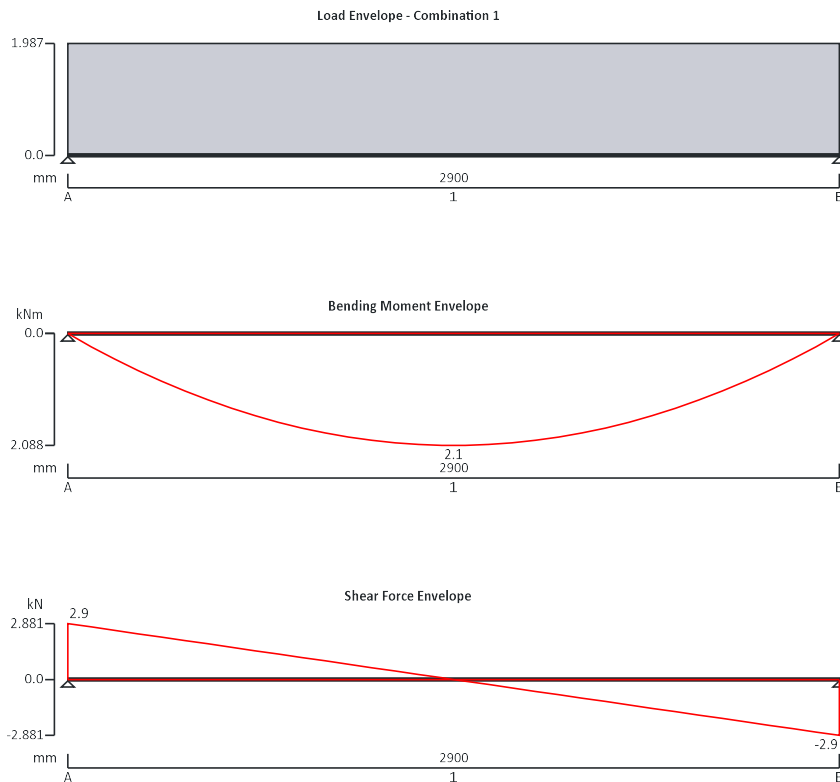


Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B2</b>										
Floor	0.5	1	0.5	g <sub>k</sub>		0.88	0.4			
				q <sub>k</sub>		1.50			0.8	
							0.4	kN/m	0.8	kN/m

**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 3.0.14



**Support conditions**

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

**Applied loading**

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 0.4 kN/m
	Variable full UDL 0.8 kN/m

**Load combinations**

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50

Permanent  $\times 1.35$ Variable  $\times 1.50$ 

Support B

Permanent  $\times 1.35$ Variable  $\times 1.50$ **Analysis results**

Maximum moment

 $M_{\max} = 2.1$  kNm $M_{\min} = 0$  kNm

Maximum shear

 $V_{\max} = 2.9$  kN $V_{\min} = -2.9$  kN

Deflection

 $\delta_{\max} = 0.8$  mm $\delta_{\min} = 0$  mm

Maximum reaction at support A

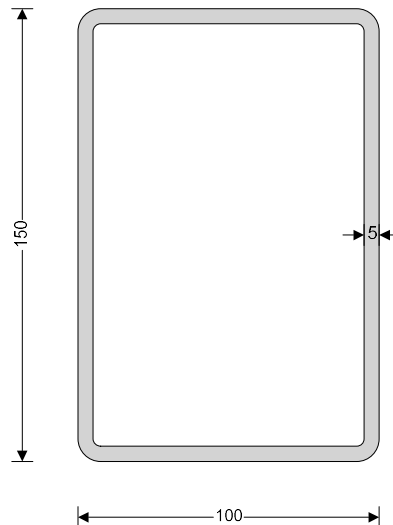
 $R_{A_{\max}} = 2.9$  kN $R_{A_{\min}} = 2.9$  kNUnfactored permanent load reaction at support A  $R_{A_{\text{Permanent}}} = 0.8$  kNUnfactored variable load reaction at support A  $R_{A_{\text{Variable}}} = 1.2$  kN

Maximum reaction at support B

 $R_{B_{\max}} = 2.9$  kN $R_{B_{\min}} = 2.9$  kNUnfactored permanent load reaction at support B  $R_{B_{\text{Permanent}}} = 0.8$  kNUnfactored variable load reaction at support B  $R_{B_{\text{Variable}}} = 1.2$  kN**Section details**

Section type

RHS 150x100x5.0 (Tata Steel Celsius (Gr355 Gr420))

Steel grade **S235H**Section classification **Class 1****Check shear - Section 6.2.6**Design shear force  $V_{Ed} = 3$  kNDesign shear resistance  $V_{c,Rd} = 193.2$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 2.1$  kNmDes.bending resist.moment  $M_{c,Rd} = 28.1$  kNm**PASS - Design bending resistance moment exceeds design bending moment**

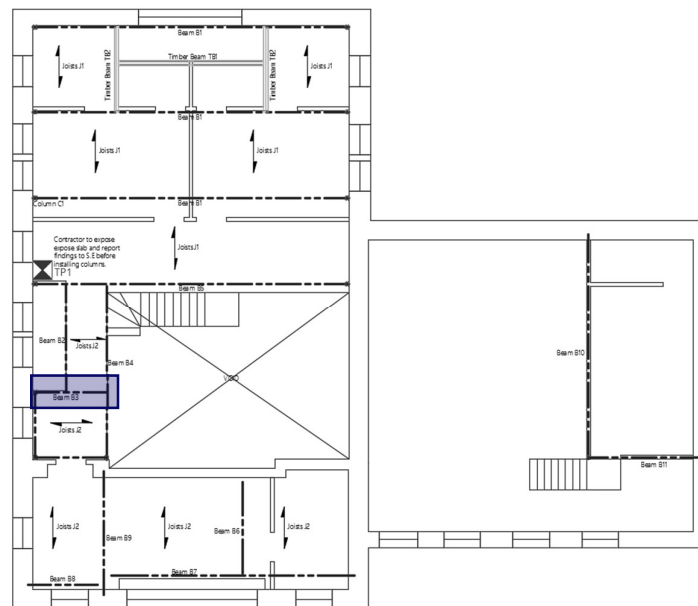
**Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection

 $\delta_{lim} = 11.6 \text{ mm}$ 

Maximum deflection

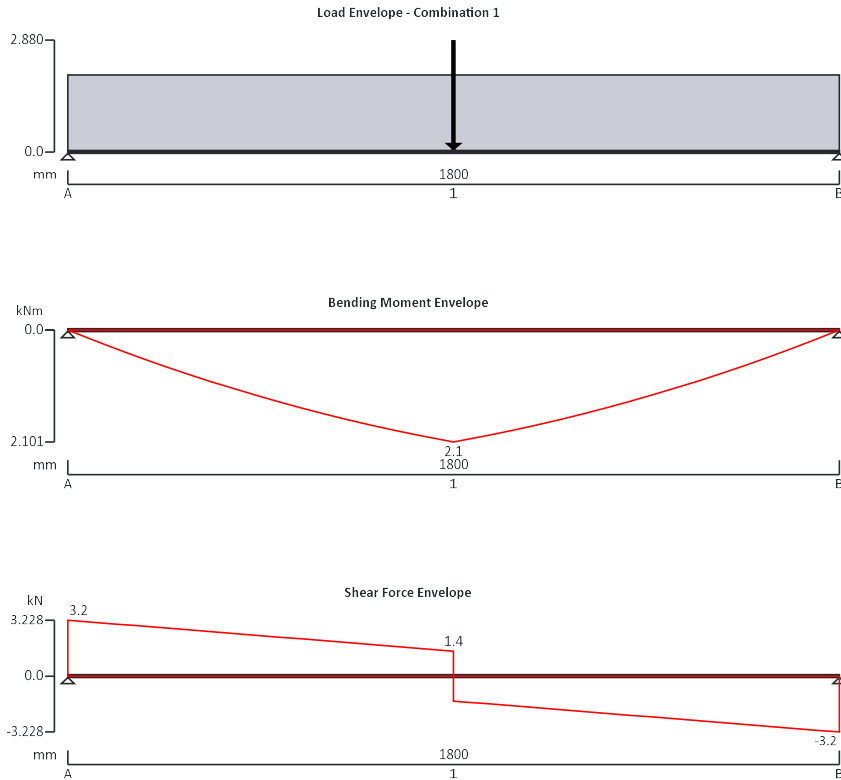
 $\delta = 0.821 \text{ mm}$ **PASS - Maximum deflection does not exceed deflection limit****BEAM B3**

Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B3</b>										
Floor	0.5	1	0.5	g <sub>k</sub>		0.88	0.4			
				q <sub>k</sub>		1.50			0.8	
							0.4	kN/m	0.8	kN/m
Beam B2				g <sub>k</sub>	0.8	kN				
				q <sub>k</sub>	1.2	kN				

**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 3.0.14



**Support conditions**

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

**Applied loading**

Beam loads	Permanent self weight of beam × 1 Permanent full UDL 0.4 kN/m Variable full UDL 0.8 kN/m Permanent point load 0.8 kN at 900 mm Variable point load 1.2 kN at 900 mm
------------	---

**Load combinations**

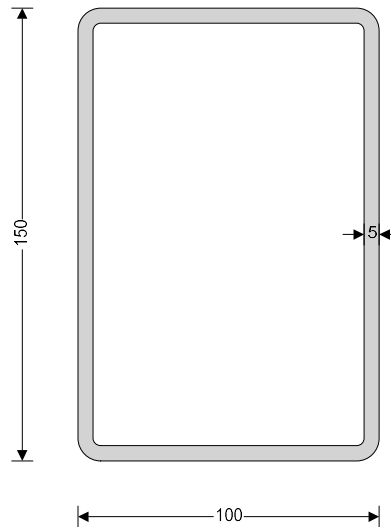
Load combination 1	Support A	Permanent × 1.35 Variable × 1.50
	Support B	Permanent × 1.35 Variable × 1.50

Variable  $\times 1.50$ **Analysis results**

Maximum moment	$M_{max} = 2.1$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 3.2$ kN	$V_{min} = -3.2$ kN
Deflection	$\delta_{max} = 0.3$ mm	$\delta_{min} = 0$ mm
Maximum reaction at support A	$R_{A,max} = 3.2$ kN	$R_{A,min} = 3.2$ kN
Unfactored permanent load reaction at support A	$R_{A,Permanent} = 0.9$ kN	
Unfactored variable load reaction at support A	$R_{A,Variable} = 1.3$ kN	
Maximum reaction at support B	$R_{B,max} = 3.2$ kN	$R_{B,min} = 3.2$ kN
Unfactored permanent load reaction at support B	$R_{B,Permanent} = 0.9$ kN	
Unfactored variable load reaction at support B	$R_{B,Variable} = 1.3$ kN	

**Section details**

Section type

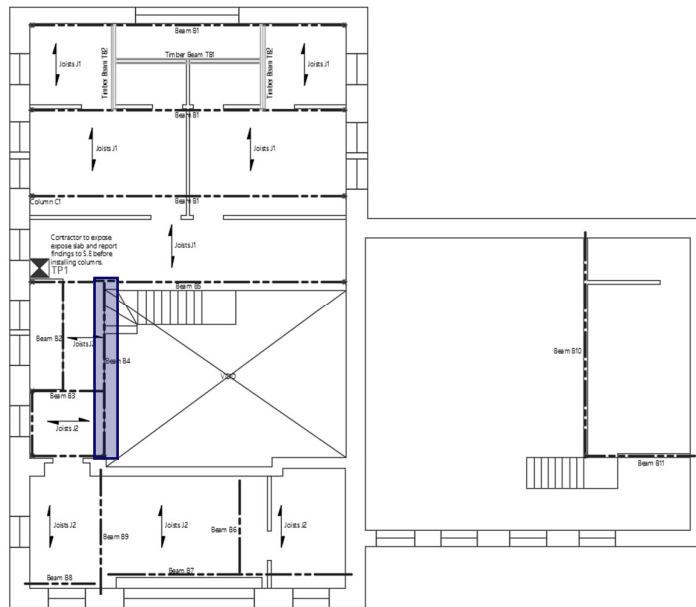
**RHS 150x100x5.0 (Tata Steel Celsius (Gr355 Gr420))**Steel grade **S235H**Section classification **Class 1****Check shear - Section 6.2.6**Design shear force  $V_{Ed} = 3$  kN      Design shear resistance  $V_{C,Rd} = 193.2$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 2.1$  kNm      Des.bending resist.moment  $M_{C,Rd} = 28.1$  kNm**PASS - Design bending resistance moment exceeds design bending moment****Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection  $\delta_{lim} = 7.2$  mm      Maximum deflection  $\delta = 0.278$  mm

**PASS - Maximum deflection does not exceed deflection limit**

**BEAM B4**

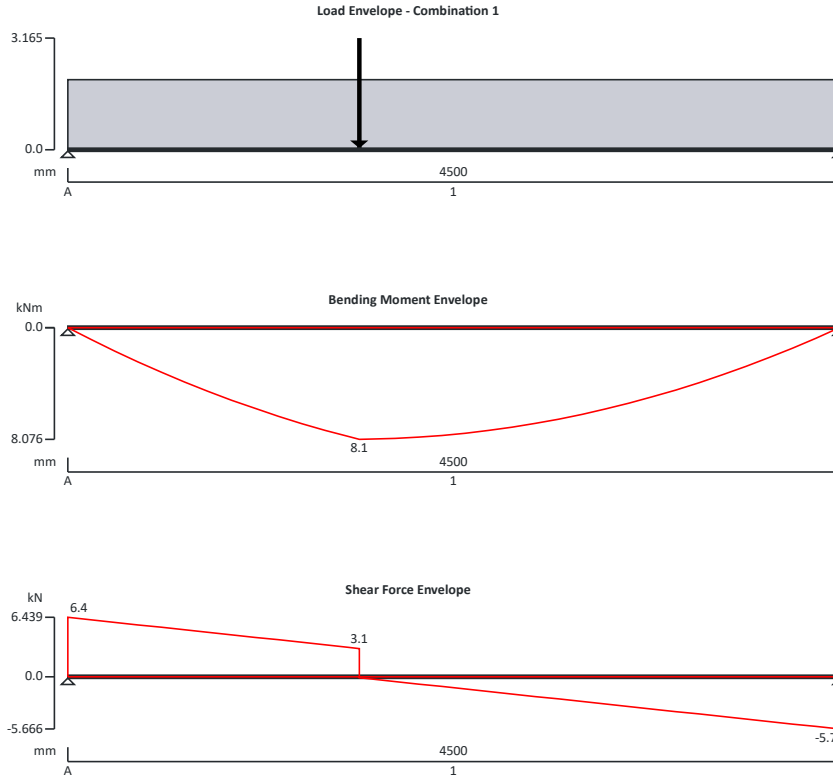


Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B4</b>										
Floor	0.5	1	0.5	g <sub>k</sub>		0.88	0.4			
				q <sub>k</sub>		1.50			0.8	
							0.4	kN/m	0.8	kN/m
Beam B3				g <sub>k</sub>	0.9	kN				
				q <sub>k</sub>	1.3	kN				

**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 3.0.14



**Support conditions**

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

**Applied loading**

Beam loads	Permanent self weight of beam × 1 Permanent full UDL 0.4 kN/m Variable full UDL 0.8 kN/m Permanent point load 0.9 kN at 1700 mm Variable point load 1.3 kN at 1700 mm
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**Load combinations**

Load combination 1	Support A	Permanent × 1.35 Variable × 1.50
	Support B	Permanent × 1.35 Variable × 1.50

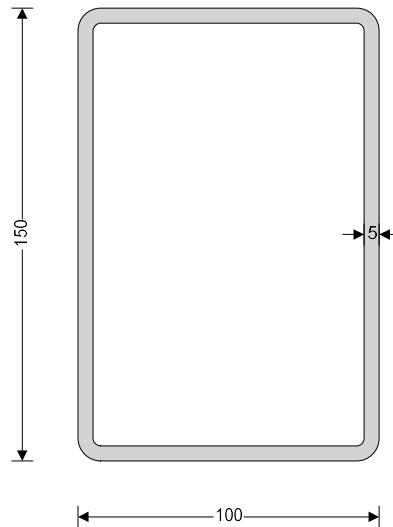
Variable  $\times 1.50$ **Analysis results**

Maximum moment	$M_{max} = 8.1$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 6.4$ kN	$V_{min} = -5.7$ kN
Deflection	$\delta_{max} = 7.2$ mm	$\delta_{min} = 0$ mm
Maximum reaction at support A	$R_{A,max} = 6.4$ kN	$R_{A,min} = 6.4$ kN
Unfactored permanent load reaction at support A	$R_{A,Permanent} = 1.9$ kN	
Unfactored variable load reaction at support A	$R_{A,Variable} = 2.6$ kN	
Maximum reaction at support B	$R_{B,max} = 5.7$ kN	$R_{B,min} = 5.7$ kN
Unfactored permanent load reaction at support B	$R_{B,Permanent} = 1.7$ kN	
Unfactored variable load reaction at support B	$R_{B,Variable} = 2.3$ kN	

**Section details**

Section type

RHS 150x100x5.0 (Tata Steel Celsius (Gr355 Gr420))

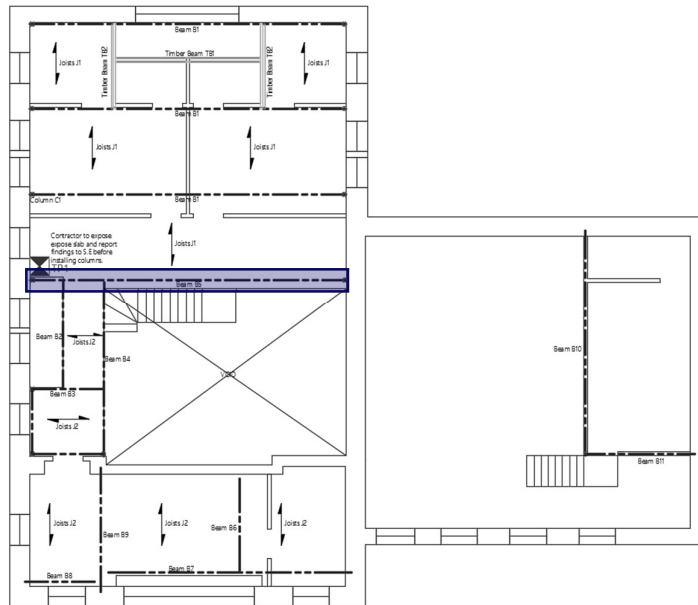
Steel grade **S355H**Section classification **Class 1****Check shear - Section 6.2.6**Design shear force  $V_{Ed} = 6$  kN      Design shear resistance  $V_{C,Rd} = 291.8$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 8.1$  kNm      Des.bending resist.moment  $M_{C,Rd} = 42.4$  kNm**PASS - Design bending resistance moment exceeds design bending moment****Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection  $\delta_{lim} = 18$  mm      Maximum deflection  $\delta = 7.235$  mm

**PASS - Maximum deflection does not exceed deflection limit**

**BEAM B5**

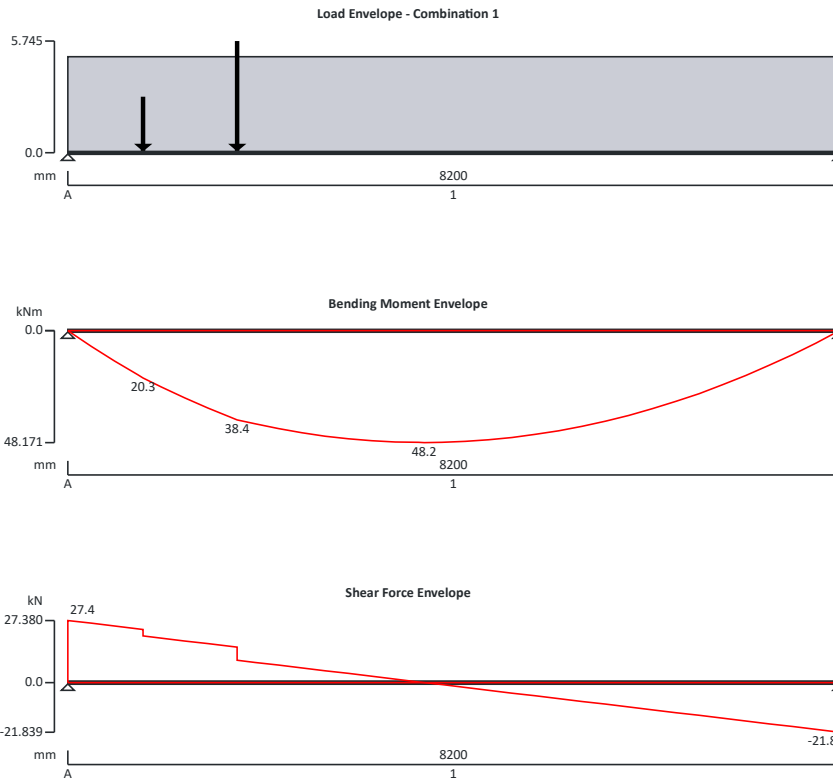


Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B5</b>										
Floor	1.25	1	1.25	g <sub>k</sub>		0.88	1.1			
				q <sub>k</sub>		1.50			1.9	
							1.1	kN/m	1.9	kN/m
Beam B2				g <sub>k</sub>	0.8	kN				
				q <sub>k</sub>	1.2	kN				
Beam B4				g <sub>k</sub>	1.7	kN				
				q <sub>k</sub>	2.3	kN				

**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 3.0.14



**Support conditions**

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

**Applied loading**

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 1.1 kN/m

Variable full UDL 1.9 kN/m

Permanent point load 0.8 kN at 800 mm

Variable point load 1.2 kN at 800 mm

Permanent point load 1.7 kN at 1800 mm

Variable point load 2.3 kN at 1800 mm

**Load combinations**

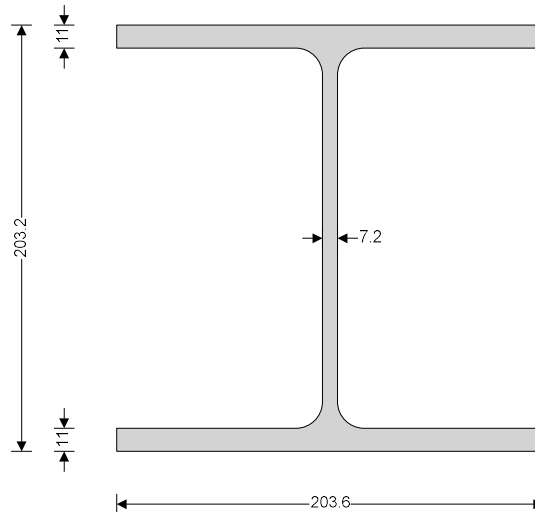
Load combination 1	Support A	Permanent × 1.35 Variable × 1.50
	Support B	Permanent × 1.35 Variable × 1.50

**Analysis results**

Maximum moment	$M_{max} = 48.2$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 27.4$ kN	$V_{min} = -21.8$ kN
Deflection	$\delta_{max} = 24.7$ mm	$\delta_{min} = 0$ mm
Maximum reaction at support A	$R_{A_{max}} = 27.4$ kN	$R_{A_{min}} = 27.4$ kN
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 8.4$ kN	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 10.7$ kN	
Maximum reaction at support B	$R_{B_{max}} = 21.8$ kN	$R_{B_{min}} = 21.8$ kN
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 6.8$ kN	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 8.4$ kN	

**Section details**

Section type **UC 203x203x46 (British Steel Section Range 2022 (BS4-1))** Steel grade **S355**



Section classification **Class 2**

**Check shear - Section 6.2.6**

Design shear force  $V_{Ed} = 27$  kN      Design shear resistance  $V_{c,Rd} = 369.3$  kN

**PASS - Design shear resistance exceeds design shear force**

**Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 48.2$  kNm  
kNm

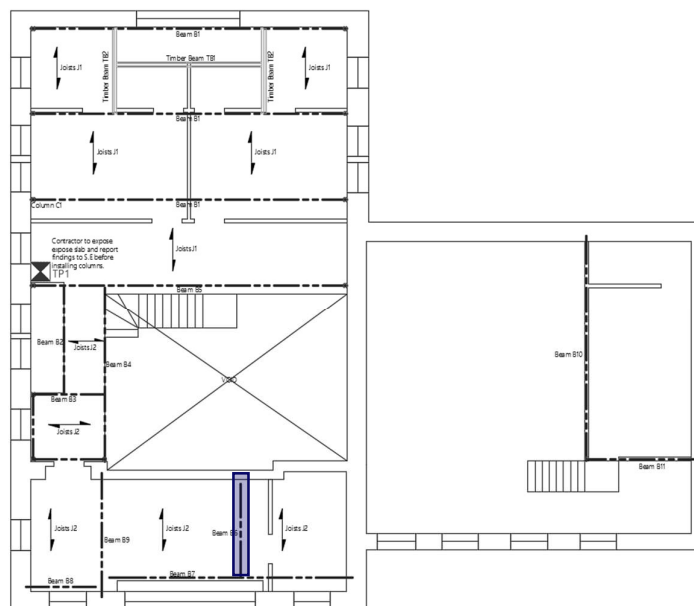
Des.bending resist.moment

 $M_{c,Rd} = 178.1$ **Slenderness ratio for lateral torsional buckling**LTB slenderness ratio  $\bar{\lambda}_{LT} = 1.103$ 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

 $M_{b,Rd} = 116.3$  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

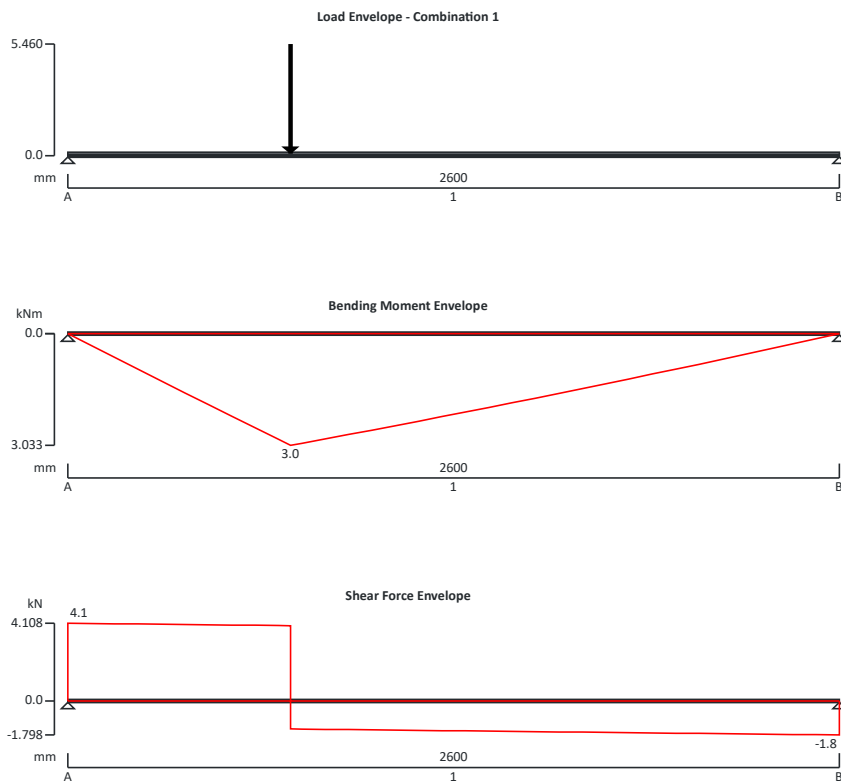
Limiting deflection  $\delta_{lim} = 32.8$  mmMaximum deflection  $\delta = 24.663$  mm**PASS - Maximum deflection does not exceed deflection limit****BEAM B6**

Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m				
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total	
<b>Beam B6</b>											
Stairs				g <sub>k</sub>	3.6	kN					
				q <sub>k</sub>	0.42	kN					

**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 3.0.14



**Support conditions**

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

**Applied loading**

- Beam loads
  - Permanent self weight of beam × 1
  - Permanent point load 3.6 kN at 750 mm

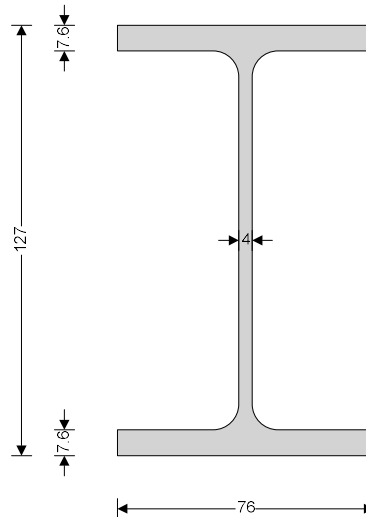
Variable point load 0.4 kN at 750 mm

**Load combinations**

Load combination 1	Support A	Permanent × 1.35 Variable × 1.50
	Support B	Permanent × 1.35 Variable × 1.50

**Analysis results**

Maximum moment	$M_{max} = 3 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 4.1 \text{ kN}$	$V_{min} = -1.8 \text{ kN}$
Deflection	$\delta_{max} = 1.2 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 4.1 \text{ kN}$	$R_{A_{min}} = 4.1 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{Permanent}} = 2.7 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{Variable}} = 0.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 1.8 \text{ kN}$	$R_{B_{min}} = 1.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{Permanent}} = 1.2 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{Variable}} = 0.1 \text{ kN}$	

**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  $V_{Ed} = 4 \text{ kN}$  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  $M_{Ed} = 3 \text{ kNm}$ 

Des.bending resist.moment

 $M_{C,Rd} = 29.9 \text{ kNm}$ **Slenderness ratio for lateral torsional buckling**LTB slenderness ratio  $\bar{\lambda}_{LT} = 1.054$ 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**

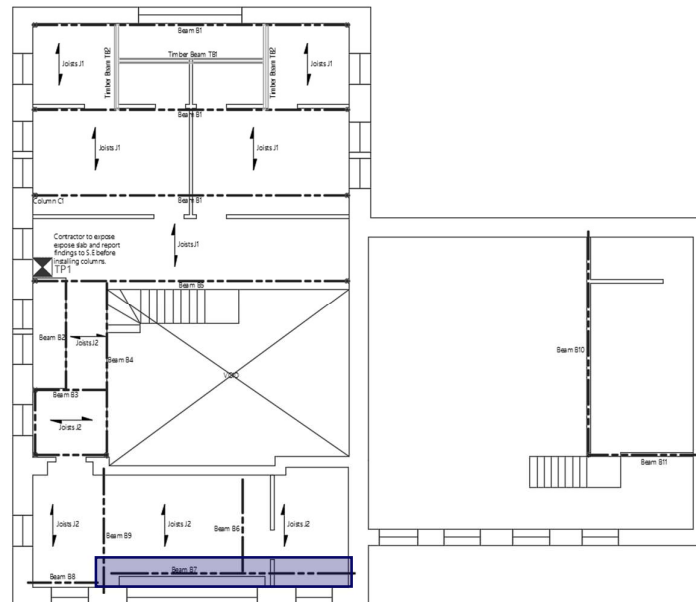
Des.buckling resist.moment

 $M_{b,Rd} = 20.4 \text{ kNm}$ **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  $\delta_{lim} = 10.4 \text{ mm}$ 

Maximum deflection

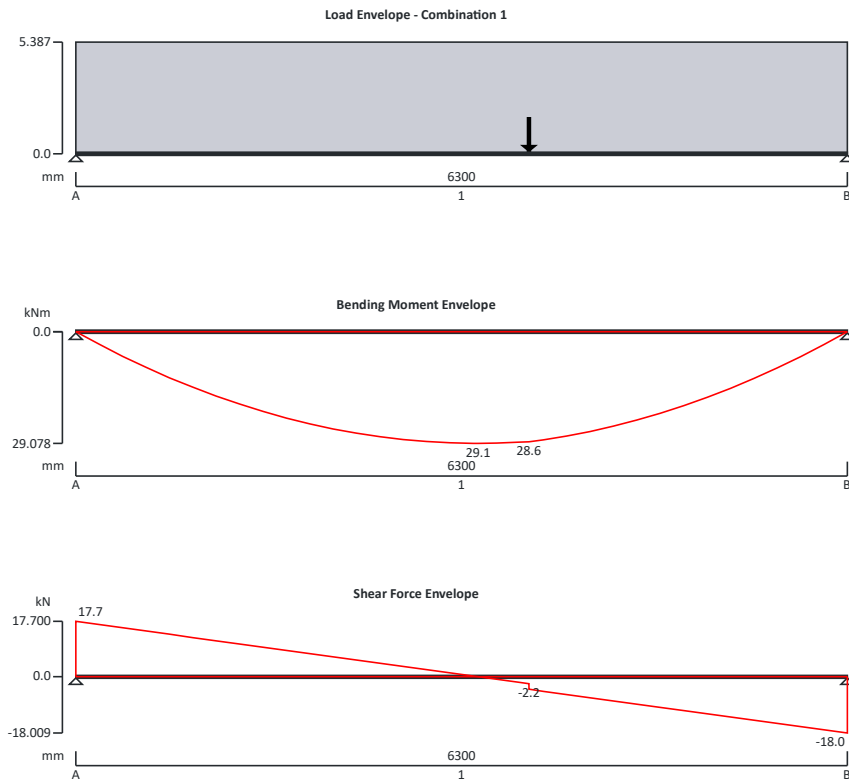
 $\delta = 1.224 \text{ mm}$ **PASS - Maximum deflection does not exceed deflection limit****BEAM B7**

Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B7</b>										
Floor	1.45	1	1.45	g <sub>k</sub>		0.88	1.3			
				q <sub>k</sub>		1.50			2.2	
							1.3	kN/m	2.2	kN/m
Beam B6				g <sub>k</sub>	1.2	kN				
				q <sub>k</sub>	0.1	kN				

**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 3.0.14



**Support conditions**

Support A

Vertically restrained  
Rotationally free

Job No.	Section Nos /Page No. /Revision	Calc By SC	Calc Date
9098	/ 38		25/06/2024

Support B

Vertically restrained

Rotationally free

**Applied loading**

Beam loads

Permanent self weight of beam  $\times 1$ 

Permanent full UDL 1.3 kN/m

Variable full UDL 2.2 kN/m

Permanent point load 1.2 kN at 3700 mm

Variable point load 0.1 kN at 3700 mm

**Load combinations**

Load combination 1

Support A

Permanent  $\times 1.35$ Variable  $\times 1.50$ Permanent  $\times 1.35$ Variable  $\times 1.50$ 

Support B

Permanent  $\times 1.35$ Variable  $\times 1.50$ **Analysis results**

Maximum moment

 $M_{max} = 29.1$  kNm $M_{min} = 0$  kNm

Maximum shear

 $V_{max} = 17.7$  kN $V_{min} = -18$  kN

Deflection

 $\delta_{max} = 17$  mm $\delta_{min} = 0$  mm

Maximum reaction at support A

 $R_{A,max} = 17.7$  kN $R_{A,min} = 17.7$  kN

Unfactored permanent load reaction at support A

 $R_{A,Permanent} = 5.4$  kN

Unfactored variable load reaction at support A

 $R_{A,Variable} = 7$  kN

Maximum reaction at support B

 $R_{B,max} = 18$  kN $R_{B,min} = 18$  kN

Unfactored permanent load reaction at support B

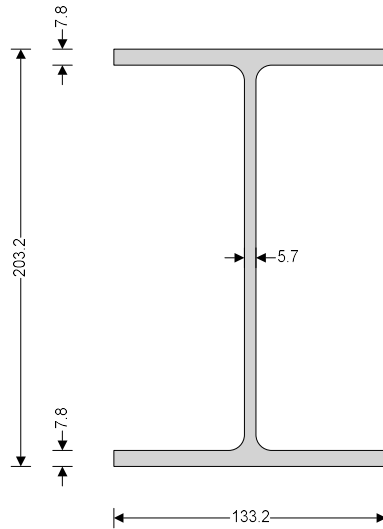
 $R_{B,Permanent} = 5.6$  kN

Unfactored variable load reaction at support B

 $R_{B,Variable} = 7$  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  $V_{Ed} = 18$  kN

Design shear resistance  $V_{C,Rd} = 262.7$  kN

**PASS - Design shear resistance exceeds design shear force**

**Check bending moment - Section 6.2.5**

Design bending moment  $M_{Ed} = 29.1$  kNm

Des.bending resist.moment  $M_{C,Rd} = 91.5$  kNm

**Slenderness ratio for lateral torsional buckling**

LTB slenderness ratio  $\bar{\lambda}_{LT} = 1.536$

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

$M_{b,Rd} = 37.7$  kNm

**PASS - Design buckling resistance moment exceeds design bending moment**

**Check vertical deflection - Section 7.2.1**

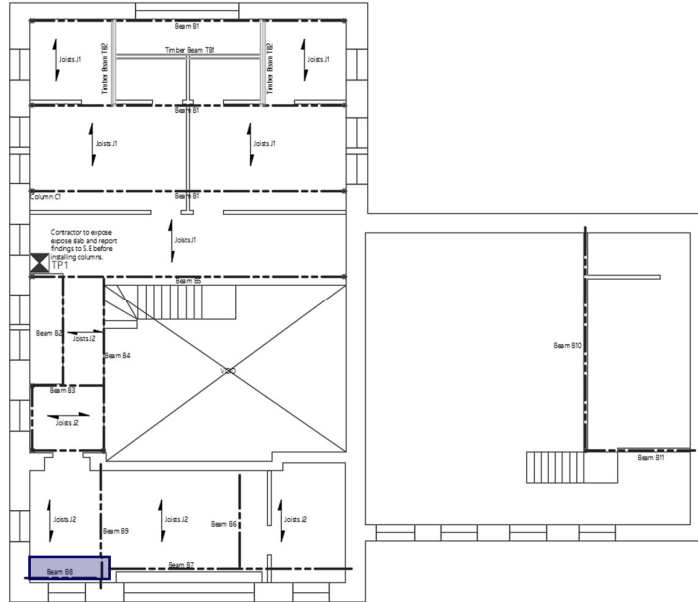
Consider deflection due to permanent and variable loads

Limiting deflection  $\delta_{lim} = 25.2$  mm

Maximum deflection  $\delta = 16.955$  mm

**PASS - Maximum deflection does not exceed deflection limit**

**BEAM B8**

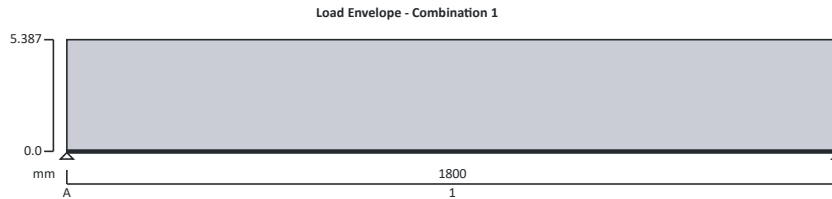


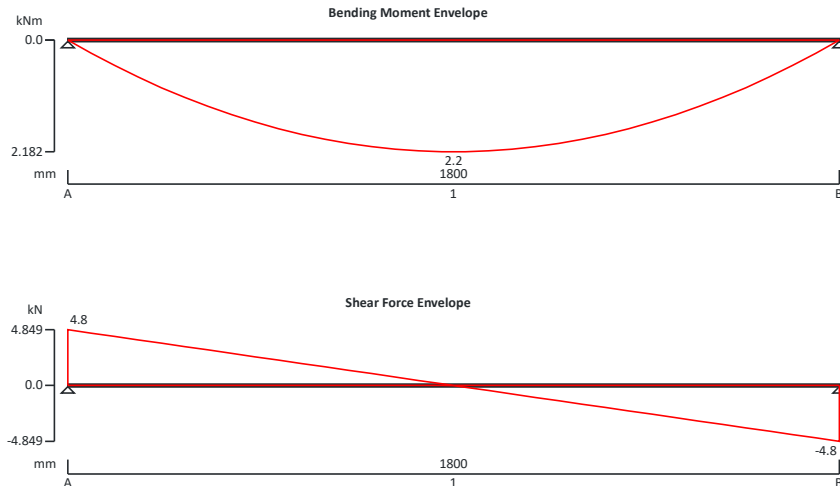
Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B8</b>										
Floor	1.45	1	1.45	g <sub>k</sub>		0.88	1.3			
				q <sub>k</sub>		1.50			2.2	
							1.3	kN/m	2.2	kN/m

**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 3.0.14



**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

Permanent full UDL 1.3 kN/m

Variable full UDL 2.2 kN/m

**Load combinations**

Load combination 1

Support A

Permanent  $\times$  1.35Variable  $\times$  1.50Permanent  $\times$  1.35Variable  $\times$  1.50

Support B

Permanent  $\times$  1.35Variable  $\times$  1.50**Analysis results**

Maximum moment

 $M_{max} = 2.2$  kNm $M_{min} = 0$  kNm

Maximum shear

 $V_{max} = 4.8$  kN $V_{min} = -4.8$  kN

Deflection

 $\delta_{max} = 0.1$  mm $\delta_{min} = 0$  mm

Maximum reaction at support A

 $R_{A_{max}} = 4.8$  kN $R_{A_{min}} = 4.8$  kN

Unfactored permanent load reaction at support A

 $R_{A_{Permanent}} = 1.4$  kN

Unfactored variable load reaction at support A

 $R_{A_{Variable}} = 2$  kN

Maximum reaction at support B

 $R_{B_{max}} = 4.8$  kN $R_{B_{min}} = 4.8$  kN

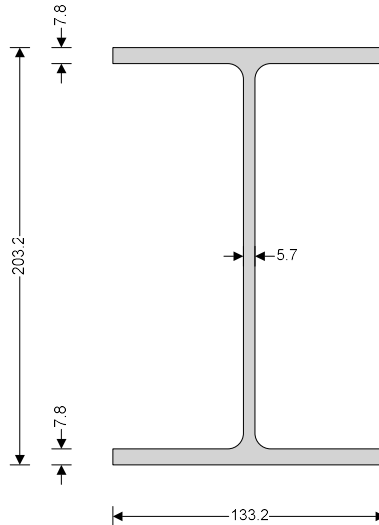
Unfactored permanent load reaction at support B  $R_{B\_Permanent} = 1.4$  kN

Unfactored variable load reaction at support B  $R_{B\_Variable} = 2$  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  $V_{Ed} = 5$  kN

Design shear resistance  $V_{C,Rd} = 262.7$  kN

**PASS - Design shear resistance exceeds design shear force**

### Check bending moment - Section 6.2.5

Design bending moment  $M_{Ed} = 2.2$  kNm

Des.bending resist.moment  $M_{C,Rd} = 91.5$  kNm

### Slenderness ratio for lateral torsional buckling

LTB slenderness ratio  $\bar{\lambda}_{LT} = 0.591$

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

$M_{b,Rd} = 86.6$  kNm

**PASS - Design buckling resistance moment exceeds design bending moment**

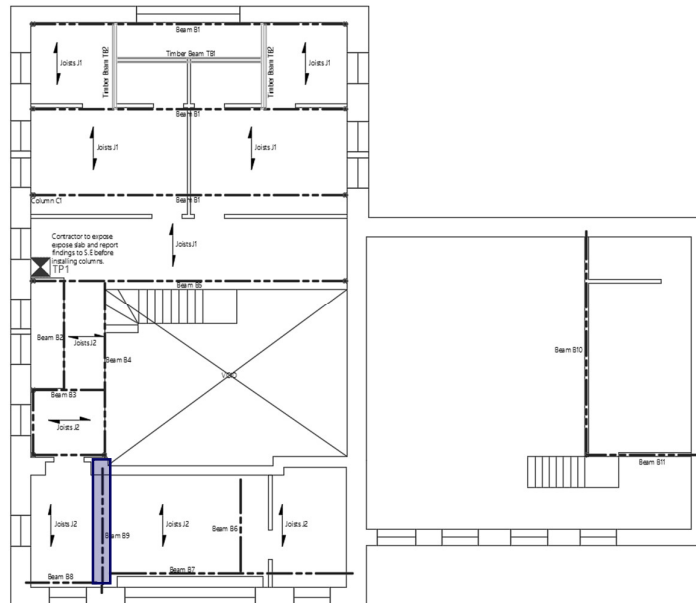
### Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection  $\delta_{lim} = 7.2$  mm

Maximum deflection  $\delta = 0.104$  mm

**PASS - Maximum deflection does not exceed deflection limit**

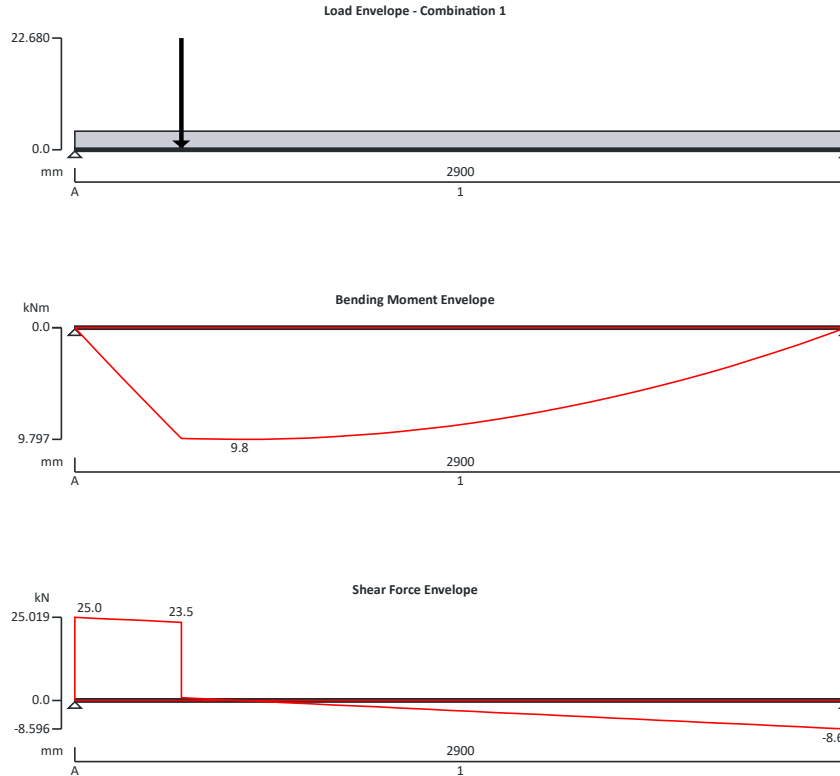
**BEAM B9**

Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B9</b>										
Floor	1	1	1	g <sub>k</sub>		0.88	0.9			
				q <sub>k</sub>		1.50			1.5	
							0.9	kN/m	1.5	kN/m
Beam B7				g <sub>k</sub>	5.4	kN				
				q <sub>k</sub>	7	kN				
Beam B8				g <sub>k</sub>	1.4	kN				
				q <sub>k</sub>	2	kN				

**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 3.0.14



**Support conditions**

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

**Applied loading**

Beam loads	Permanent self weight of beam × 1 Permanent full UDL 0.9 kN/m Variable full UDL 1.5 kN/m Permanent point load 5.4 kN at 400 mm Variable point load 7 kN at 400 mm Permanent point load 1.4 kN at 400 mm Variable point load 2 kN at 400 mm
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**Load combinations**

Load combination 1	Support A	Permanent × 1.35 Variable × 1.50 Permanent × 1.35
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Support B

Variable  $\times 1.50$ Permanent  $\times 1.35$ Variable  $\times 1.50$ **Analysis results**

Maximum moment

 $M_{\max} = 9.8$  kNm $M_{\min} = 0$  kNm

Maximum shear

 $V_{\max} = 25$  kN $V_{\min} = -8.6$  kN

Deflection

 $\delta_{\max} = 1.3$  mm $\delta_{\min} = 0$  mm

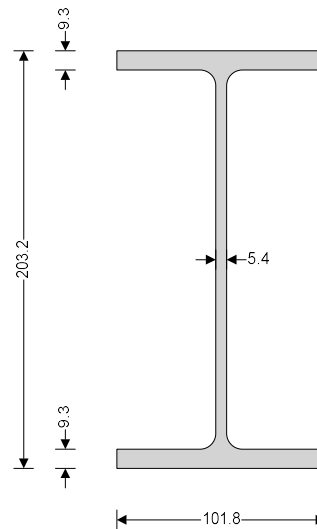
Maximum reaction at support A

 $R_{A_{\max}} = 25$  kN $R_{A_{\min}} = 25$  kNUnfactored permanent load reaction at support A  $R_{A_{\text{Permanent}}} = 7.5$  kNUnfactored variable load reaction at support A  $R_{A_{\text{Variable}}} = 9.9$  kN

Maximum reaction at support B

 $R_{B_{\max}} = 8.6$  kN $R_{B_{\min}} = 8.6$  kNUnfactored permanent load reaction at support B  $R_{B_{\text{Permanent}}} = 2.6$  kNUnfactored variable load reaction at support B  $R_{B_{\text{Variable}}} = 3.4$  kN**Section details**

Section type

**UB 203x102x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S355**Section classification **Class 1****Check shear - Section 6.2.6**Design shear force  $V_{Ed} = 25$  kNDesign shear resistance  $V_{C,Rd} = 253.7$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 9.8$  kNmDes. bending resist. moment  $M_{C,Rd} = 83.1$  kNm**Slenderness ratio for lateral torsional buckling**LTB slenderness ratio  $\bar{\lambda}_{LT} = 1.068$ 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

$M_{b,Rd} = 56.1$  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

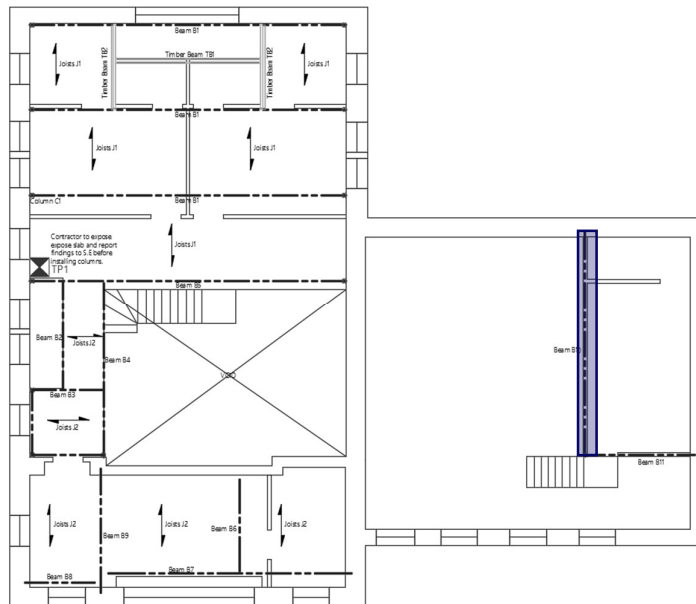
$\delta_{lim} = 11.6$  mm

Maximum deflection

$\delta = 1.289$  mm

**PASS - Maximum deflection does not exceed deflection limit**

**BEAM B10**

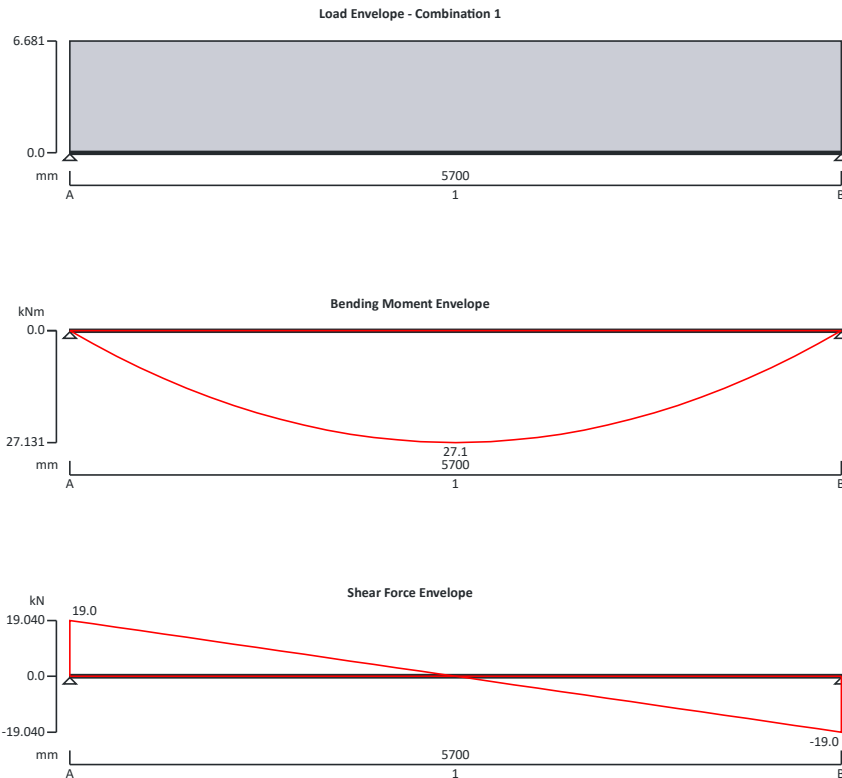


Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Beam B10</b>										
Floor	1.35	1	1.35	g <sub>k</sub>		0.88	1.2			
				q <sub>k</sub>		1.50			2.0	
Partition	2.5	1	2.5	g <sub>k</sub>		0.52	1.3			
							2.5	kN/m	2.0	kN/m

**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 3.0.14

**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

Permanent full UDL 2.5 kN/m

Variable full UDL 2 kN/m

**Load combinations**

Load combination 1

Support A

Permanent  $\times$  1.35Variable  $\times$  1.50Permanent  $\times$  1.35

Support B

Variable × 1.50

Permanent × 1.35

Variable × 1.50

**Analysis results**

Maximum moment

 $M_{max} = 27.1$  kNm $M_{min} = 0$  kNm

Maximum shear

 $V_{max} = 19$  kN $V_{min} = -19$  kN

Deflection

 $\delta_{max} = 14.7$  mm $\delta_{min} = 0$  mm

Maximum reaction at support A

 $R_{A,max} = 19$  kN $R_{A,min} = 19$  kN

Unfactored permanent load reaction at support A

 $R_{A,Permanent} = 7.8$  kN

Unfactored variable load reaction at support A

 $R_{A,Variable} = 5.7$  kN

Maximum reaction at support B

 $R_{B,max} = 19$  kN $R_{B,min} = 19$  kN

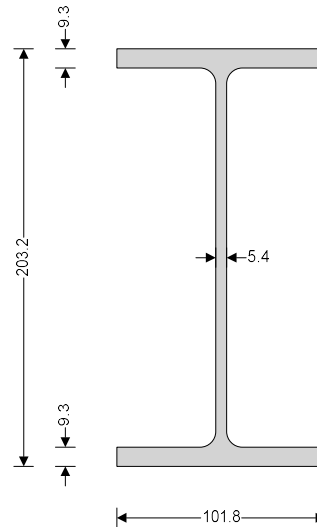
Unfactored permanent load reaction at support B

 $R_{B,Permanent} = 7.8$  kN

Unfactored variable load reaction at support B

 $R_{B,Variable} = 5.7$  kN**Section details**

Section type

**UB 203x102x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S355**

Section classification

**Class 1****Check shear - Section 6.2.6**Design shear force  $V_{Ed} = 19$  kNDesign shear resistance  $V_{C,Rd} = 253.7$  kN**PASS - Design shear resistance exceeds design shear force****Check bending moment - Section 6.2.5**Design bending moment  $M_{Ed} = 27.1$  kNm

Des. bending resist. moment

 $M_{C,Rd} = 83.1$  kNm**Slenderness ratio for lateral torsional buckling**

LTB slenderness ratio

 $\bar{\lambda}_{LT} = 1.635$ 

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

$M_{b,Rd} = 31.1$  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

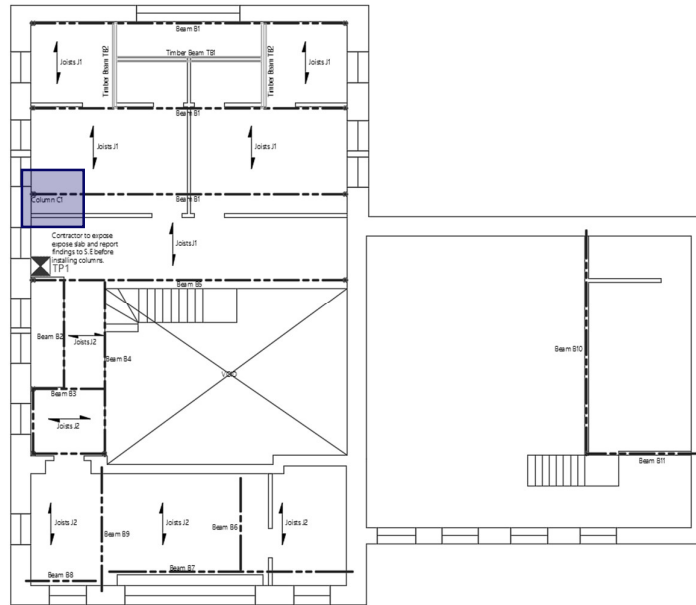
$\delta_{lim} = 22.8$  mm

Maximum deflection

$\delta = 14.696$  mm

**PASS - Maximum deflection does not exceed deflection limit**

**COLUMN C1**



Location	Area			Type	L	Action kN/m <sup>2</sup>	Actions, kN or kN/m			
	L	W	m <sup>2</sup>				Perm., g <sub>k</sub>	%	Var., q <sub>k</sub>	Total
<b>Column C1</b>										
Beam B1				N	44	kN				

**STEEL COLUMN DESIGN (EN1993-1-1:2005)**

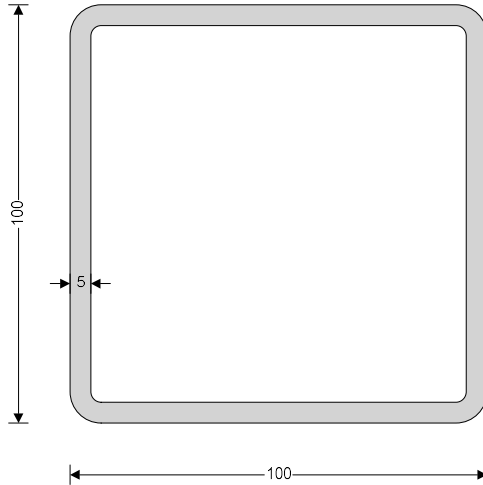
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 1.1.08

**Design summary**

Description	Unit	Provided	Required	Utilisation	Result
Axial compression	kN	665	44	0.066	PASS
Bending resistance (z-z)	kNm	24	4	0.187	PASS

Description	Unit	Provided	Required	Utilisation	Result
Combined bending & axial	kNm	24	4	0.187	PASS
Buckling in compression	kN	200	44	0.220	PASS
Combined buckling				0.351	PASS



SHS 100x100x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))

Section depth, h, 100 mm  
 Section breadth, b, 100 mm  
 Mass of section, Mass, 14.7 kg/m  
 Section thickness, t, 5 mm  
 Area of section, A, 1873 mm<sup>2</sup>  
 Radius of gyration about y-axis,  $i_y$ , 38.623 mm  
 Radius of gyration about z-axis,  $i_z$ , 38.623 mm  
 Elastic section modulus about y-axis,  $W_{el,y}$ , 55886 mm<sup>3</sup>  
 Elastic section modulus about z-axis,  $W_{el,z}$ , 55886 mm<sup>3</sup>  
 Plastic section modulus about y-axis,  $W_{pl,y}$ , 66358 mm<sup>3</sup>  
 Plastic section modulus about z-axis,  $W_{pl,z}$ , 66358 mm<sup>3</sup>  
 Second moment of area about y-axis,  $I_y$ , 2794323 mm<sup>4</sup>  
 Second moment of area about z-axis,  $I_z$ , 2794323 mm<sup>4</sup>

### Column details

Column section	<b>SHS 100x100x5.0</b>	Steel grade	<b>S355H</b>
Yield strength	$f_y = 355 \text{ N/mm}^2$	Ultimate strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210 \text{ kN/mm}^2$	Shear modulus	$G = 80.8 \text{ kN/mm}^2$

### Column geometry

Major axis buckling length  $L_y = 2500 \text{ mm}$       Minor axis buckling length  $L_z = 2500 \text{ mm}$

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

### Column loading

Axial load	$N_{Ed} = 44 \text{ kN}$ (Compression)		
Major axis moment at end 1	$M_{y,Ed1} = 0.0 \text{ kNm}$	Major axis moment at end 2	$M_{y,Ed2} = 0.0 \text{ kNm}$
Minor axis moment at end 1	$M_{z,Ed1} = 0.0 \text{ kNm}$	Minor axis moment at end 2	$M_{z,Ed2} = 4.4 \text{ kNm}$
		<b>Minor axis bending is single curvature</b>	
Major axis shear force	$V_{y,Ed} = 0 \text{ kN}$	Minor axis shear force	$V_{z,Ed} = 0 \text{ kN}$

### Buckling length for flexural buckling - Major axis

End restraint factor  $K_y = 1.000$       Buckling length  $L_{cr,y} = 2500 \text{ mm}$

### Buckling length for flexural buckling - Minor axis

End restraint factor  $K_z = 2.000$       Buckling length  $L_{cr,z} = 5000 \text{ mm}$

### Section classification (Table 5.2)

Web classification	<b>1</b>	Flange classification	<b>1</b>
			<b>The section is class 1</b>

### Resistance of cross section (cl. 6.2)

#### Shear - Minor axis (cl. 6.2.6)

Design shear force  $V_{z,Ed} = 0.0 \text{ kN}$       Plastic shear resistance  $V_{pl,z,Rd} = 192.0 \text{ kN}$

**PASS - Shear resistance exceeds the design shear force**

**$V_{z,Ed} \leq 0.5 \times V_{pl,z,Rd}$  - No reduction in  $f_y$  required for bending/axial force**

**Shear - Major axis (cl. 6.2.6)**

Design shear force

$V_{y,Ed} = 0.0 \text{ kN}$

Plastic shear resistance  $V_{pl,y,Rd} = 192.0 \text{ kN}$ **PASS - Shear resistance exceeds the design shear force**

**$V_{y,Ed} \leq 0.5 \times V_{pl,y,Rd}$  - No reduction in  $f_y$  required for bending/axial force**

**Compression (cl. 6.2.4)**

Design force

$N_{Ed} = 44 \text{ kN}$

Design resistance

$N_{c,Rd} = 665 \text{ kN}$

**PASS - The compression design resistance exceeds the design force****Bending - Minor axis (cl. 6.2.5)**

Design bending moment

$M_{z,Ed} = 4.4 \text{ kNm}$

Design resistance

$M_{c,z,Rd} = 23.6 \text{ kNm}$

**PASS - The bending design resistance exceeds the design moment****Combined bending and axial force (cl. 6.2.9)****Bending about z axis (cl. 6.2.9.1)**

Design bending moment

$M_{z,Ed} = 4.4 \text{ kNm}$

Modified design resistance

$M_{N,z,Rd} = 23.6$

kNm

**PASS - Bending resistance about z axis in presence of axial load exceeds design moment****Buckling resistance (cl. 6.3)****Axial buckling resistance**Flexural buck resist about y  
kN

$N_{b,y,Rd} = 510.5 \text{ kN}$

Flexural buck resist about z

$N_{b,z,Rd} = 200.3$

Min. buckling resistance

$N_{b,Rd} = 200.3 \text{ kN}$

**PASS - The axial load buckling resistance exceeds the design axial load****Combined bending and axial compression (cl. 6.3.3)**

Section utilisation

$UR_{B,1} = 0.165$

Section utilisation

$UR_{B,2} = 0.351$

**PASS - The buckling resistance is adequate****PADSTONE P1**

Brick compressive strength

$p_{unit} = 1 \text{ N/mm}^2$

Reaction at support

$V = 43 \text{ kN}$

Padstone area

$A_{padstone} = 100\text{mm} \times 430\text{mm} = 43000.000 \text{ mm}^2$

Masonry compressive stress

$p = V / A_{padstone} = 1.000 \text{ N/mm}^2$

**Provide a 100x430x215dp Naylor PC padstone.**