

vision | DESIGN | group

**4 Ellesmere Road South
Chorlton**

STRUCTURAL CALCULATIONS
VDG-D-100

Document Revision Status

Issue	Date	Comments
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Contents

1 Description of project 1

2 Elements to be designed: 1

3 Design standards 1

4 Loadings 2

5 Plans of design 3

6 Designs 4

6.1 B1..... 4

6.2 T1..... **Error! Bookmark not defined.**

6.3 Lintel Design Doors **Error! Bookmark not defined.**

General Notes

Calculations are based on the use of workmanship and material complying with the latest edition of the Building Regulation.

The Client and Contractor are entirely responsibly for the quality of the workmanship and materials.

Dimensions where shown in these calculations are for design purposes only and the Contractor must check all details and dimension for compliance prior to ordering any material.

1 Description of project

The proposed works are taken place at 4 Ellesmere Road, Chorlton, Manchester.

The works consist of the following:

- Single steel extensions
- Removal of load bearing walls
- Bi-fold steelwork.

We have been instructed to undertake the structural calculations and design.

2 Elements to be designed:

- Steel Beam over bi-fold
- Steel Beams

3 Design standards

The building and associated works will be designed in accordance with Eurocodes and Codes of Practice as listed and as appropriate to the form of construction used, but not limited to.

- The Building Regulations
- British Standards
- Eurocode 3: Design of steel structures
- Eurocode 5: Design of Timber Structures
- Eurocode 1:Part 1-1: General actions-Densities, self-weight and imposed loads.
- Eurocode 1:Part 1-3: General actions-Snow loads
- Eurocode 1:Part 1-4: General actions-Wind loads

- Eurocode 0: Basis for Design

4 Loadings

Pitch roof Permanent loading Timber floor

Tiles=0.4

Timber battens and felt=0.15

Roof Timber=0.2

Ceiling and services=0.15

Total Load=0.9

Roof 30 degrees load= 1.20kN/m² .

Pitch Roof Variable Loading

=0.6kN/m²

First floor Timber floor

Timber floor joist= 0.2

18mm chipboard=0.15

Ceiling and services=0.25

Total load= 0.60kN/m²

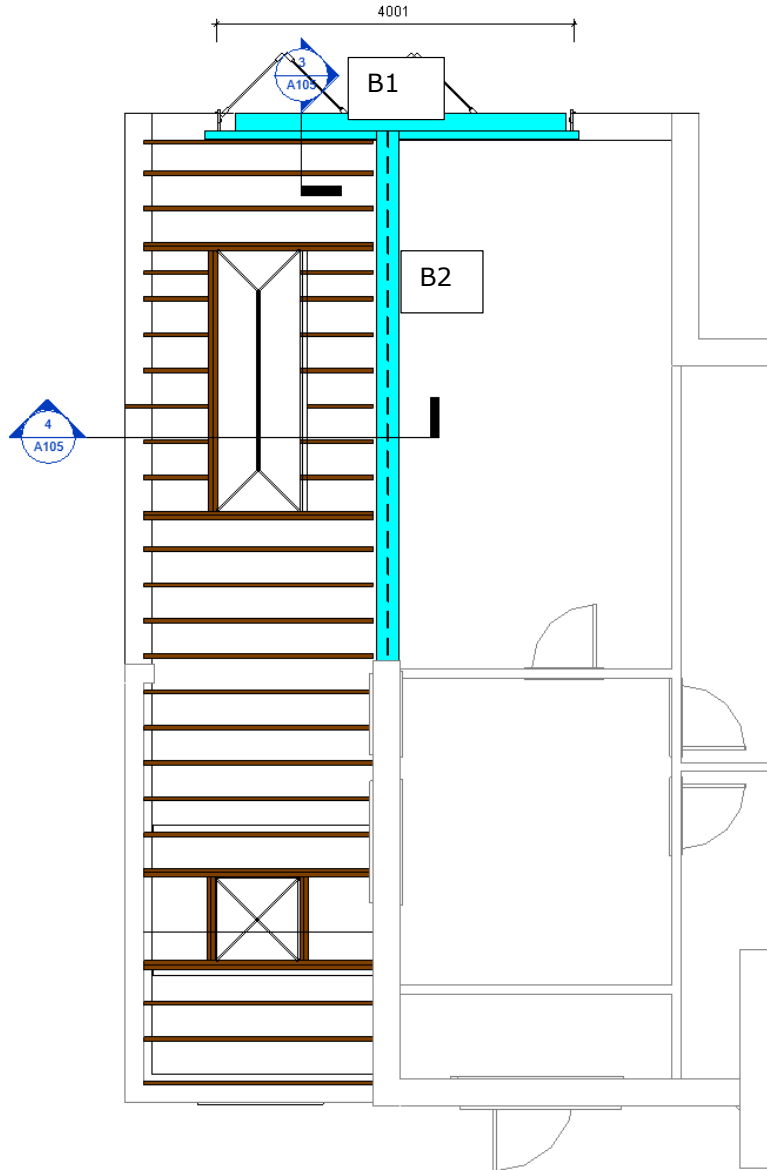
First Floor Variable Loading

=1.5kN/m²

5 Plans of design

The following snapshots have been taken from LHG Project architectural drawings to aid the scope of the calculations:

Proposed Sketches



6 Designs

6.1 B1

Max span=4300

Higher Roof- 2m contributed area

2m contributed area x 0.6=1.2Qk

2m contributed area x 1.2=2.4Gk

Udl to half of beam

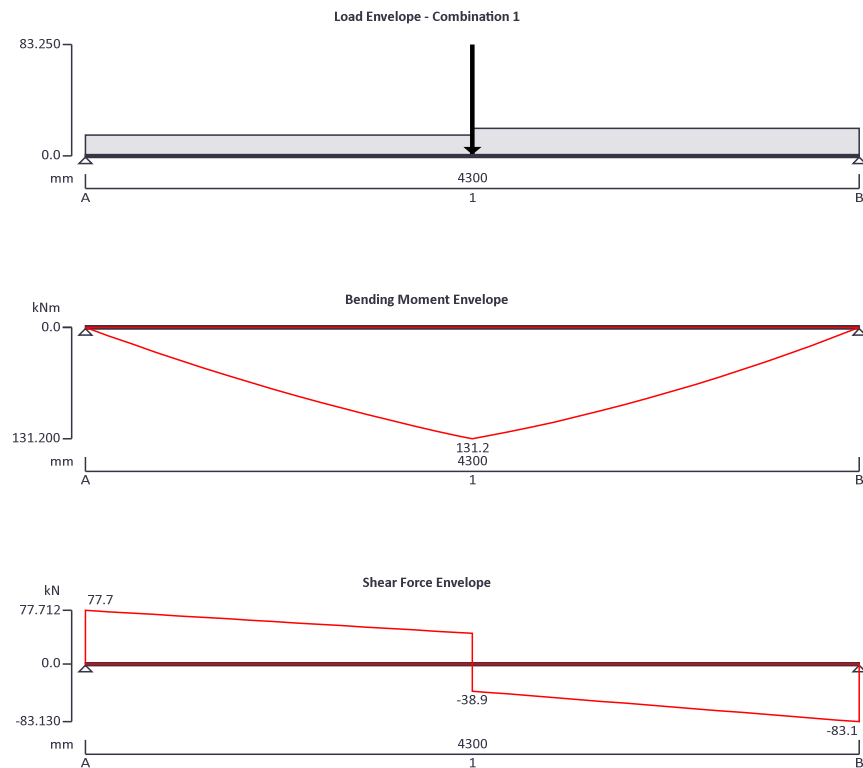
Point load from B2- 45GK, 15QK

Brickwork- 11kN/m run

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 partial UDL 2.4 kN/m from 2150 mm to 4300 mm
Variable partial UDL 1.2 kN/m from 2150 mm to 4300 mm
Permanent point load 45 kN at 2150 mm
Variable point load 15 kN at 2150 mm
Permanent full UDL 11 kN/m

Load combinations

Load combination 1

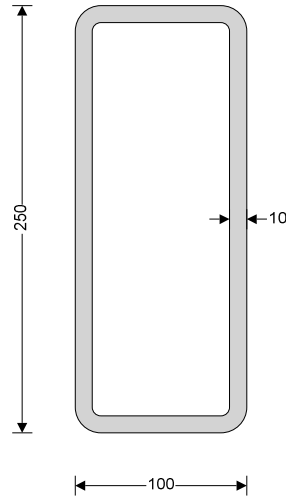
Support A
Permanent $\times 1.35$
Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$
Support B
Permanent $\times 1.35$
Variable $\times 1.50$

Analysis results

Maximum moment;	$M_{\max} = 131.2 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 77.7 \text{ kN};$	$V_{\min} = -83.1 \text{ kN}$
Deflection;	$\delta_{\max} = 16 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 77.7 \text{ kN};$	$R_{A_{\min}} = 77.7 \text{ kN}$
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 48.5 \text{ kN}$	
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 8.1 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 83.1 \text{ kN};$	$R_{B_{\min}} = 83.1 \text{ kN}$
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 51.1 \text{ kN}$	
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 9.4 \text{ kN}$	

Section details

Section type;	RHS 250x100x10.0 (Tata Steel Celsius (Gr355 Gr420
Gr460))	
Steel grade;	S355H
EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels	
Nominal thickness of element;	$t = 10.0 \text{ mm}$
Nominal yield strength;	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength;	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity;	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections;	$\gamma_{M0} = 1.00$
Resistance of members to instability;	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture;	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	$K_y = 1.000$
Effective length factor in minor axis;	$K_z = 1.000$
Effective length factor for torsion;	$K_{LT,A} = 1.400; + 2 \times h$
	$K_{LT,B} = 1.400; + 2 \times h$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section;	$c = h - 3 \times t = 220 \text{ mm}$	
	$c / t = 27.0 \times \varepsilon \leq 72 \times \varepsilon;$	Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section;	$c = b - 3 \times t = 70 \text{ mm}$	
	$c / t = 8.6 \times \varepsilon \leq 33 \times \varepsilon;$	Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web;	$h_w = h - 2 \times t = 230 \text{ mm}$
Shear area factor;	$\eta = 1.000$
	$h_w / t < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force;	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 83.1 \text{ kN}$
Shear area - cl 6.2.6(3);	$A_v = A \times h / (b + h) = 4638 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2);	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 950.5 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment;	$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 131.2 \text{ kNm}$
Design bending resistance moment - eq 6.13;	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 174.2 \text{ kNm}$

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; $\delta_{lim} = L_{s1} / 200 = 21.5 \text{ mm}$

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 15.957 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

6.2 B2

Max span=6100

Higher Roof- 1.5m contributed area

1.5m contributed area x 0.6=0.9Qk

1.5m contributed area x 1.2=1.8Gk

Floor Joist- 1.5m contributed area

1.5m contributed area x 1.5=2.25Qk

1.5m contributed area x 0.6=0.9Gk

Lower Roof- 1.4m contributed area

1.4m contributed area x 0.6=0.84Qk

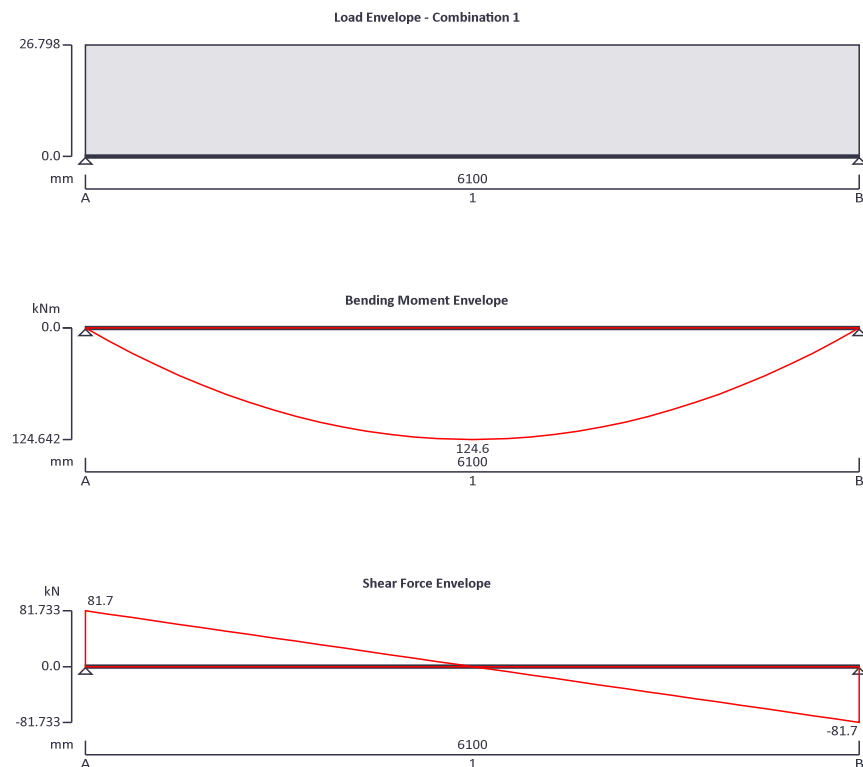
1.4m contributed area x 0.7=1.0Gk

Brickwork- 11kN/m run

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

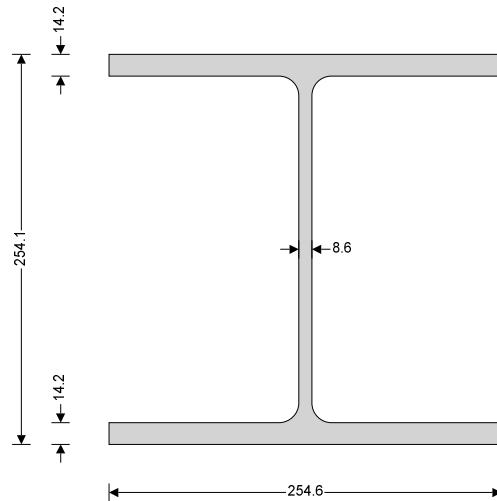
Vertically restrained

Rotationally free

Support B

Vertically restrained

	Rotationally free	
Applied loading		
Beam loads	Permanent self weight of beam $\times 1$ Permanent full UDL 1.8 kN/m Variable full UDL 0.9 kN/m Permanent full UDL 0.9 kN/m Variable full UDL 2.25 kN/m Permanent full UDL 11 kN/m Permanent full UDL 1 kN/m Variable full UDL 0.84 kN/m	
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;	$M_{\max} = 124.6$ kNm;	$M_{\min} = 0$ kNm
Maximum shear;	$V_{\max} = 81.7$ kN;	$V_{\min} = -81.7$ kN
Deflection;	$\delta_{\max} = 14.6$ mm;	$\delta_{\min} = 0$ mm
Maximum reaction at support A;	$R_{A_{\max}} = 81.7$ kN;	$R_{A_{\min}} = 81.7$ kN
Unfactored permanent load reaction at support A;	$R_{A_{\text{Permanent}}} = 47$ kN	
Unfactored variable load reaction at support A;	$R_{A_{\text{Variable}}} = 12.2$ kN	
Maximum reaction at support B;	$R_{B_{\max}} = 81.7$ kN;	$R_{B_{\min}} = 81.7$ kN
Unfactored permanent load reaction at support B;	$R_{B_{\text{Permanent}}} = 47$ kN	
Unfactored variable load reaction at support B;	$R_{B_{\text{Variable}}} = 12.2$ kN	
Section details		
Section type;	UC 254x254x73 (BS4-1)	
Steel grade;	S355	
EN 10025-2:2004 - Hot rolled products of structural steels		
Nominal thickness of element;	$t = \max(t_r, t_w) = 14.2$ mm	
Nominal yield strength;	$f_y = 355$ N/mm ²	
Nominal ultimate tensile strength;	$f_u = 470$ N/mm ²	
Modulus of elasticity;	$E = 210000$ N/mm ²	



Partial factors - Section 6.1

Resistance of cross-sections;	$\gamma_{M0} = 1.00$
Resistance of members to instability;	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture;	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	$K_y = 1.000$
Effective length factor in minor axis;	$K_z = 1.000$
Effective length factor for torsion;	$K_{LT,A} = 1.400; + 2 \times h$
	$K_{LT,B} = 1.400; + 2 \times h$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section;	$c = d = 200.3 \text{ mm}$	
	$c / t_w = 28.6 \times \varepsilon \leq 72 \times \varepsilon;$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section;	$c = (b - t_w - 2 \times r) / 2 = 110.3 \text{ mm}$	
	$c / t_f = 9.5 \times \varepsilon \leq 10 \times \varepsilon;$	Class 2
		Section is class 2

Check shear - Section 6.2.6

Height of web;	$h_w = h - 2 \times t_f = 225.7 \text{ mm}$
Shear area factor;	$\eta = 1.000$
	$h_w / t_w < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force;	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 81.7 \text{ kN}$
Shear area - cl 6.2.6(3);	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$
2562 mm²	
Design shear resistance - cl 6.2.6(2);	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 525.2 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment;	$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 124.6 \text{ kNm}$
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Design bending resistance moment - eq 6.13;

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{352.2 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6;

$$k_c = \mathbf{1}$$

$$C_1 = 1 / k_c^2 = \mathbf{1}$$

Destabilised load condition factor;

$$D = \mathbf{1.2}$$

Curvature factor;

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.811}$$

Poissons ratio;

$$\nu = \mathbf{0.3}$$

Shear modulus;

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length;

$$L = 1.4 \times L_{s1} + 2 \times h = \mathbf{9048 \text{ mm}}$$

Elastic critical buckling moment;

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{302.4 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling;

$$\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{1.295}$$

Limiting slenderness ratio;

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5;

$$b$$

Imperfection factor - Table 6.3;

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections;

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor;

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.281}$$

LTB reduction factor - eq 6.57;

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$$

$$\mathbf{0.526}$$

Modification factor;

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$$

$$\mathbf{1.000}$$

Modified LTB reduction factor - eq 6.58;

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.526}$$

Design buckling resistance moment - eq 6.55;

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{185.4 \text{ kNm}}$$

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;

$$\delta_{lim} = L_{s1} / 200 = \mathbf{30.5 \text{ mm}}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{14.605 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit