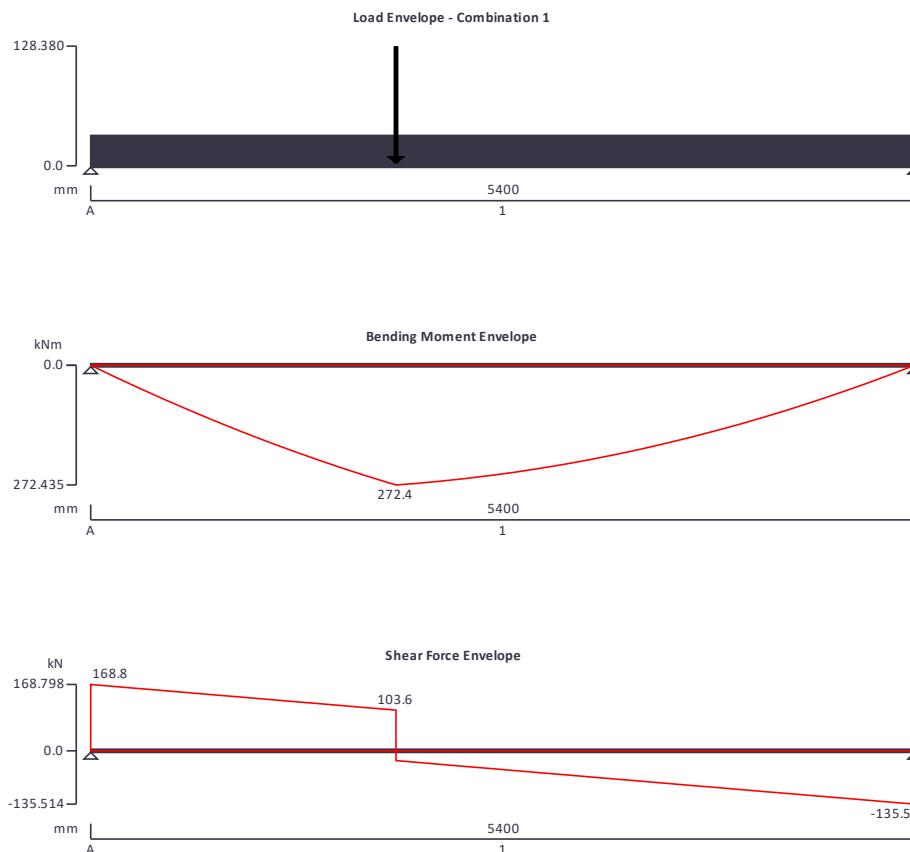
 CPS Architecture + Design Limited 7 Middleborough Colchester Essex	Project 97 Longmeadows Harwich				Job no. 17_861	
	Calcs for Beam A				Start page no./Revision 1	
	Calcs by AF	Calcs date 25/03/2020	Checked by AF	Checked date	Approved by AF	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

PL - Dead point load 91.7 kN at 2000 mm

roof - Dead full UDL 3.6 kN/m

roof - Imposed full UDL 2.6 kN/m

roof - Dead full UDL 1.5 kN/m

roof - Imposed full UDL 1.1 kN/m

wall - Dead full UDL 13 kN/m

Dead self weight of beam $\times 1$

Load combinations


Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

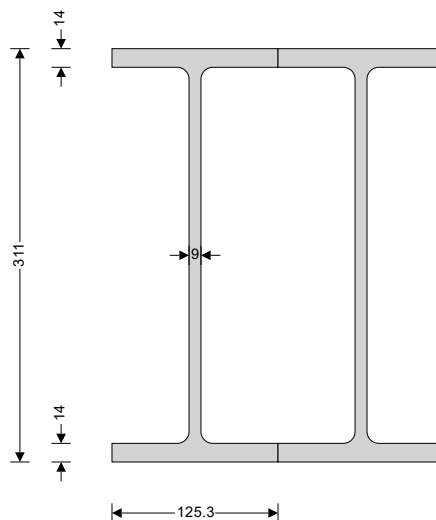
 CPS Architecture + Design Limited 7 Middleborough Colchester Essex	Project 97 Longmeadows Harwich				Job no. 17_861	
	Calcs for Beam A				Start page no./Revision 2	
	Calcs by AF	Calcs date 25/03/2020	Checked by AF	Checked date	Approved by AF	Approved date

Analysis results

Maximum moment	$M_{max} = 272.4 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 168.8 \text{ kN}$	$V_{min} = -135.5 \text{ kN}$
Deflection	$\delta_{max} = 13.4 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 168.8 \text{ kN}$	$R_{A_{min}} = 168.8 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 109.2 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 10 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 135.5 \text{ kN}$	$R_{B_{min}} = 135.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 85.4 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 10 \text{ kN}$	

Section details

Section type	2 x UKB 305x127x48 (Tata Steel Advance)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 14.0 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors


Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LTA} = 1.00$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 265.2 \text{ mm}$
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Outstand flanges - Table 11

Width of section

$$d / t = 29.5 \times \varepsilon \leq 80 \times \varepsilon$$

Class 1 plastic

$$b = B / 2 = 62.7 \text{ mm}$$

$$b / T = 4.5 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 168.8 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 2799 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times N \times p_y \times A_v = 923.7 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 272.4 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = N \times \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 390.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 15 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 13.399 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit