

INTRODUCTION

The following structural calculations have been undertaken for the steel beams required to carry a new truss roof as part of the extension works at 21 Harrison Hey, Huyton. A single storey extension is to be constructed at the side of the property, linking the existing garage to the main house.

A steel member, Beam A, will span a distance of 6.04m across the centre of the extension and will support a 4.68m width of new truss roof. The ends of the beam will be supported by existing load-bearing masonry.

Two steel members, Beams B and C, will span across openings at the front and rear of the extension, supporting the new truss roof. Beam C has been selected for design as it is subject to the higher load, and then the same section size will be used for Beam B. Beam C will span 6.04m and will carry a 2.9m width of roof. The ends of these beams will be supported by a combination of existing load-bearing masonry and new steel posts.

Extension roof dead load : 0.85kN/m^2

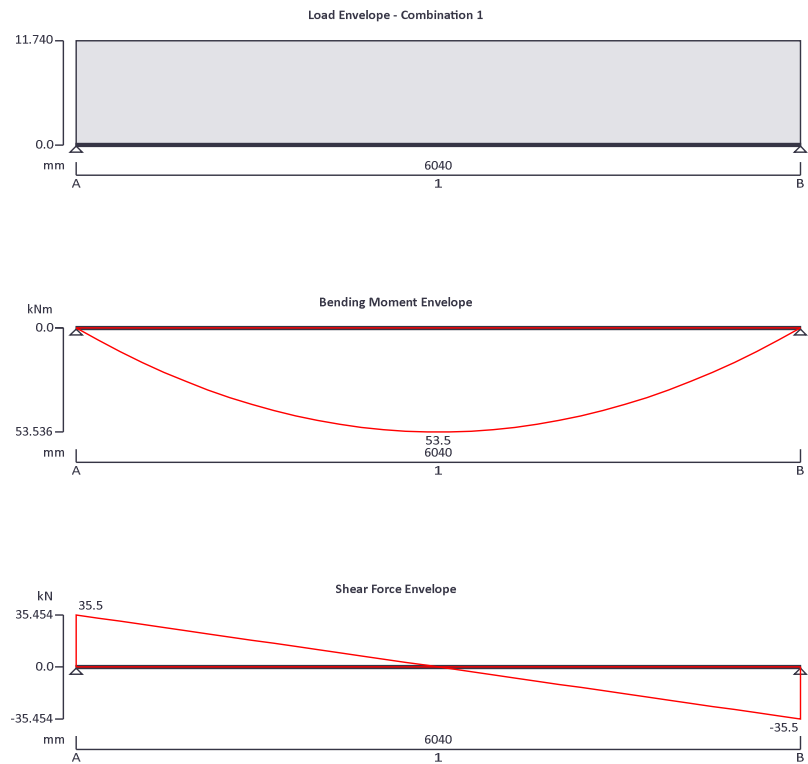
Extension roof imposed load : 0.75kN/m^2

BEAM A ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\gamma = 1$

Roof - Dead full UDL 4 kN/m

Roof - Imposed full UDL 3.52 kN/m

Load combinations

Load combination 1

Support A

Dead $\gamma = 1.40$

Imposed $\gamma = 1.60$

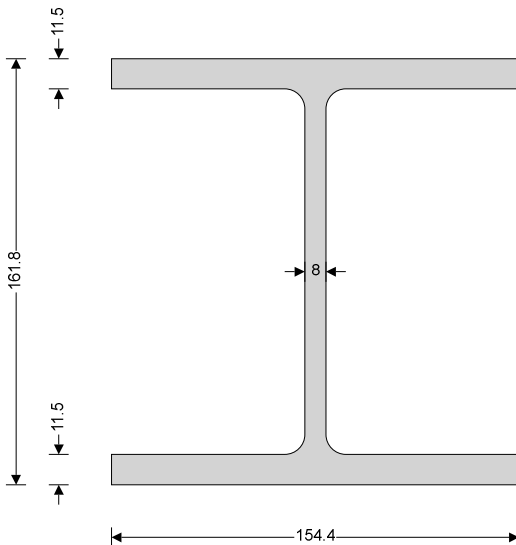
Dead $\hat{}$ 1.40
Imposed $\hat{}$ 1.60

Dead $\hat{}$ 1.40
Imposed $\hat{}$ 1.60

Support B

Analysis results		
Maximum moment	$M_{\max} = 53.5 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 35.5 \text{ kN}$	$V_{\min} = -35.5 \text{ kN}$
Deflection	$\delta_{\max} = 13.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 35.5 \text{ kN}$	$R_{A_{\min}} = 35.5 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 13.2 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 10.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 35.5 \text{ kN}$	$R_{B_{\min}} = 35.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 13.2 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 10.6 \text{ kN}$	

Section details	
Section type	UC 152x152x37 (BS4-1)
Steel grade	S355
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 11.5 \text{ mm}$
Design strength	$p_y = 355 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis $K_x = 1.00$

Effective length factor in minor axis $K_y = 1.00$

Effective length factor for lateral-torsional buckling $K_{LT,A} = 1.00$

$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section $d = 123.6 \text{ mm}$

$d / t = 17.6 < \varepsilon \leq 80 < \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = 77.2 \text{ mm}$

$b / T = 7.6 < \varepsilon \leq 9 < \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 35.5 \text{ kN}$

$d / t < 70 < \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \cdot D = 1294 \text{ mm}^2$

Design shear resistance $P_v = 0.6 \cdot p_y \cdot A_v = 275.7 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 53.5 \text{ kNm}$

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \cdot S_{xx}, 1.2 \cdot p_y \cdot Z_{xx}) = 109.6 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \cdot L_{s1} = 6040 \text{ mm}$

Slenderness ratio $\lambda = L_E / r_{yy} = 155.999$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = 0.848$

Torsional index $x = 13.334$

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.598}$

Ratio - cl.4.3.6.9 $\beta_w = \mathbf{1.000}$

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_w]} = \mathbf{79.065}$

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{30.198}$

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

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = \mathbf{7.0}$

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.342}$

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{323.7 \text{ N/mm}^2}$

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{394.7 \text{ N/mm}^2}$

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{192.5 \text{ N/mm}^2}$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = \mathbf{40.2 \text{ kNm}}$

Moment at centre-line of segment $M_3 = \mathbf{53.5 \text{ kNm}}$

Moment at three quarter point of segment $M_4 = \mathbf{40.2 \text{ kNm}}$

Maximum moment in segment $M_{abs} = \mathbf{53.5 \text{ kNm}}$

Maximum moment governing buckling resistance $M_{LT} = M_{abs} = \mathbf{53.5 \text{ kNm}}$

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = \mathbf{59.4 \text{ kNm}}$

$M_b / m_{LT} = \mathbf{64.3 \text{ kNm}}$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{16.778 \text{ mm}}$

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{13.461 \text{ mm}}$

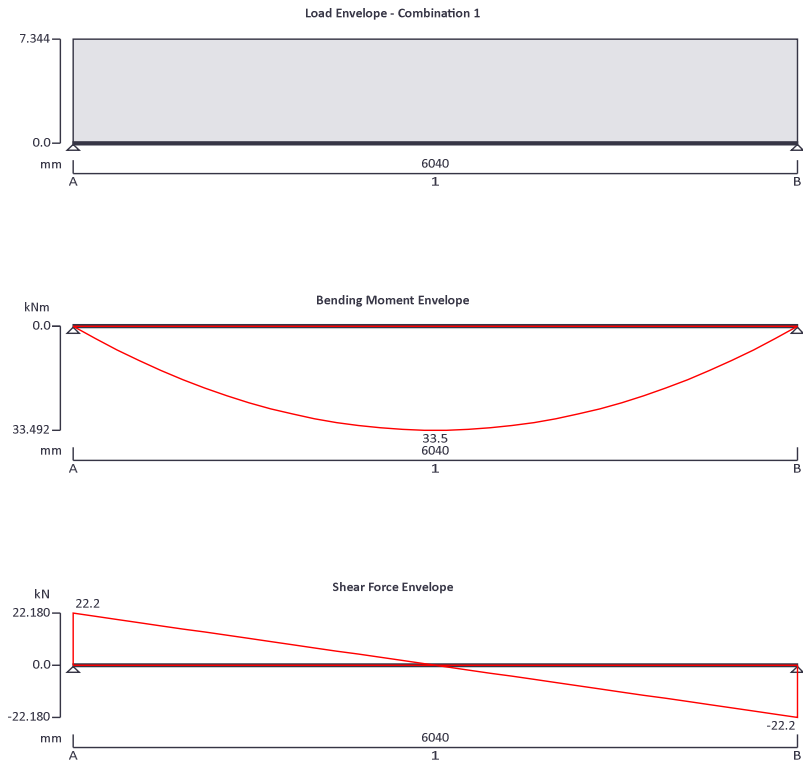
PASS - Maximum deflection does not exceed deflection limit

BEAMS B AND C ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\gamma_c = 1$

Roof - Dead full UDL 2.46 kN/m

Roof - Imposed full UDL 2.18 kN/m

Load combinations

Load combination 1

Support A

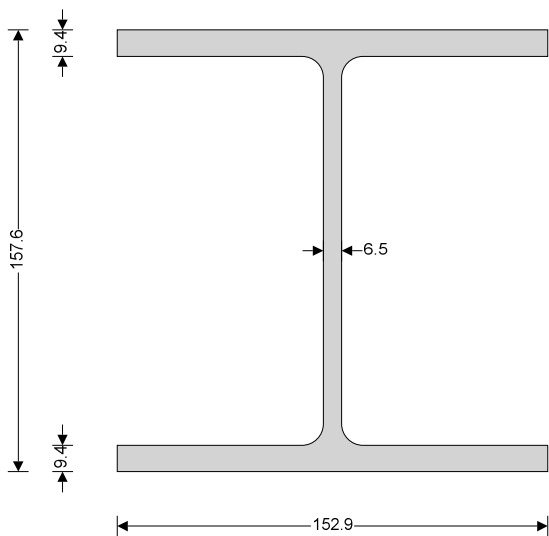
Dead $\gamma_c = 1.40$

Imposed $\gamma_c = 1.60$

Dead $\gamma_c = 1.40$

		Imposed $\gamma = 1.60$
	Support B	Dead $\gamma = 1.40$
		Imposed $\gamma = 1.60$
Analysis results		
Maximum moment	$M_{\max} = 33.5 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 22.2 \text{ kN}$	$V_{\min} = -22.2 \text{ kN}$
Deflection	$\delta_{\max} = 10.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 22.2 \text{ kN}$	$R_{A_{\min}} = 22.2 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 8.3 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 6.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 22.2 \text{ kN}$	$R_{B_{\min}} = 22.2 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 8.3 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 6.6 \text{ kN}$	

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



Lateral restraint	Span 1 has lateral restraint at supports only
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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		$K_{LT.A} = 1.00$
	$K_{LT.B} = 1.00$	

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

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

Outstand flanges - Table 11

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

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force	$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 22.2 \text{ kN}$
	$d / t < 70 \quad \varepsilon$

Web does not need to be checked for shear buckling

Shear area	$A_v = t \cdot D = 1024 \text{ mm}^2$
Design shear resistance	$P_v = 0.6 \cdot p_y \cdot A_v = 218.2 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment kNm	$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 33.5$
Moment capacity low shear - cl.4.2.5.2 kNm	$M_c = \min(p_y \cdot S_{xx}, 1.2 \cdot p_y \cdot Z_{xx}) = 87.9$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling	$L_E = 1.0 \cdot L_{s1} = 6040 \text{ mm}$
Slenderness ratio	$\lambda = L_E / r_{yy} = 157.817$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter	$u = 0.849$
Torsional index	$x = 15.999$
Slenderness factor	$v = 1 / [1 + 0.05 \times (\lambda / x)^{2.25}] = 0.643$
Ratio - cl.4.3.6.9	$\beta_w = 1.000$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_w]} = \mathbf{86.049}$$

Limiting slenderness - Annex B.2.2

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

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = \mathbf{7.0}$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.391}$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{273.3 \text{ N/mm}^2}$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{367.5 \text{ N/mm}^2}$$

Bending strength - Annex B.2.1
N/mm²

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{172.4}$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = \mathbf{25.1 \text{ kNm}}$$

Moment at centre-line of segment

$$M_3 = \mathbf{33.5 \text{ kNm}}$$

Moment at three quarter point of segment

$$M_4 = \mathbf{25.1 \text{ kNm}}$$

Maximum moment in segment

$$M_{abs} = \mathbf{33.5 \text{ kNm}}$$

Maximum moment governing buckling resistance
33.5 kNm

$$M_{LT} = M_{abs} =$$

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = \mathbf{42.7 \text{ kNm}}$$

$$M_b / m_{LT} = \mathbf{46.2 \text{ kNm}}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{16.778 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{10.542 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit