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Project

Tottington Manor Farm, Turton Road, Tottington

Job Ref.

8674

Section

Sheet no./rev.

1

Calc. by

Date

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AP

17/11/2022

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17/11/2022

STRUCTURAL CALCULATIONS

FOR

ROOF STEEL

AT

TOTTINGTON MANOR FARM

TURTON ROAD

TOTTINGTON



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LOADINGS

ROOF

	Dead	Imposed
Metal sheeting; D1 =	0.20 kN/m ²	
purlins; D2 =	0.05 kN/m ²	
services; D3 =	0.20 kN/m ²	
soffite & insulation; D4 =	0.15 kN/m ²	
imposed; I1 =		0.60 kN/m ²

Total unfactored roof dead load;

Roof_{dead} = sum (D1, D2, D3, D4) = **0.600 kN/m²**

Total unfactored roof imposed load;

Roof_{imp} = I1 = **0.600 kN/m²**

SNOW LOAD

Site grid reference – SD 755 145

Site altitude – 223 m

SNOW LOADING TO BS6399:PART 3:1988

TEDDS calculation version 1.0.03

Site location

Location of site;

Turton

Site altitude;

A = 223 m

Calculate site snow load

From BS6399:Part 3: 1988 - Figure 1. Basic snow load on the ground

Basic snow load;

$s_b = 0.55 \text{ kN/m}^2$

$s_{alt} = 0.1 \times s_b + (0.09 \text{ kN/m}^2) = 0.14 \text{ kN/m}^2$

Site snow load;

$s_0 = \max(s_b, s_b + s_{alt} \times (A - (100 \text{ m})) / 100 \text{ m}) = 0.73 \text{ kN/m}^2$

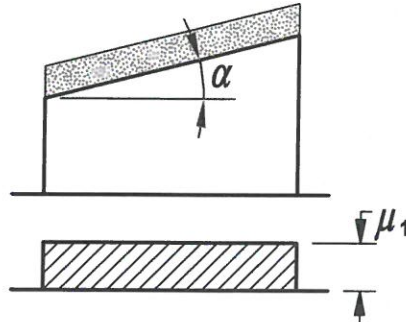
BS6399:Part3:1988 Cl.6.2



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Uniform loading

Roof geometry

Roof type; **Monopitch**
Distance on plan from gutter to ridge; **$b = 4.000$ m**
Angle of pitch of roof; **$\alpha = 5.0$ deg**

Calculate uniform snow load

From BS6399:Part 3: 1988 - Figure 2. Snow load shape coefficients for flat or monopitch roofs

Snow load shape coefficient; **$\mu_1 = 0.80$**
Uniform roof snow load; **$s_{d1} = \mu_1 \times s_0 = 0.58$ kN/m²**

BS6399:Part3:1988 Cl.5

Snow sliding down roof

Maximum uniform snow load on roof; **$s_{d_max} = 0.58$ kN/m²**
Force from sliding snow load; **$F_s = s_{d_max} \times b \times \sin(\alpha) = 0.20$ kN/m**

BS6399:Part3:1988 Cl.8



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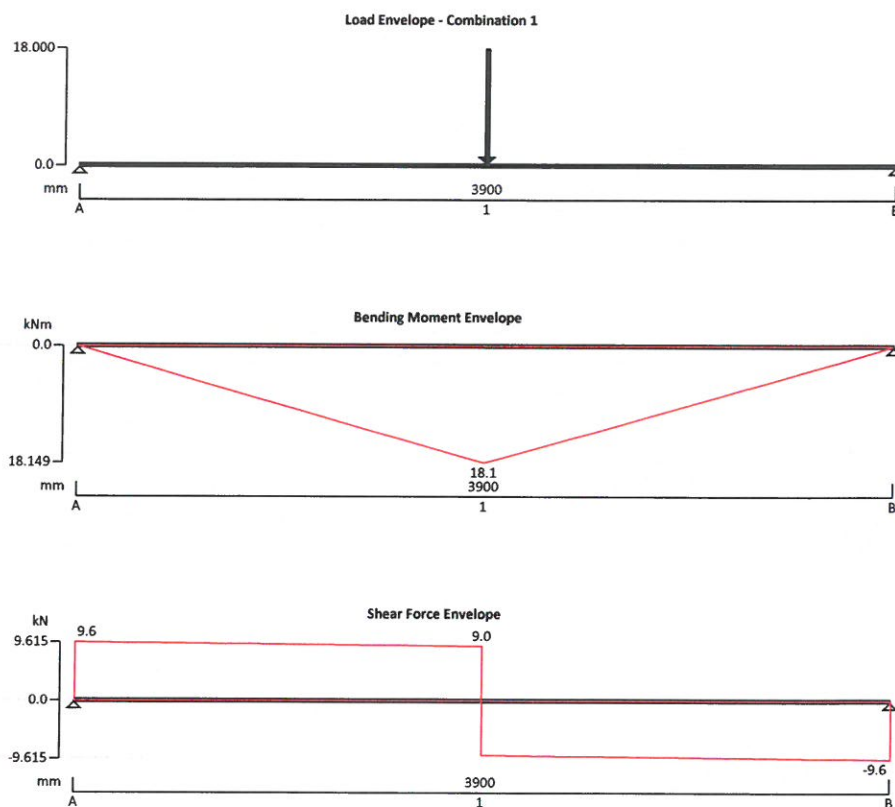
BEAM DESIGN - CASE 1

Beam spans 3900 mm and carries the following loads –
10 m² of roof from central purlin

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 $\times 1$

roof - Dead point load 6 kN at 1950 mm

roof - Imposed point load 6 kN at 1950 mm

Load combinations

Load combination 1 - dead+imp

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$



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Support B

Dead $\times 1.40$
Imposed $\times 1.60$

Analysis results

Maximum moment;

$M_{max} = 18.1 \text{ kNm}$;

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

Maximum shear;

$V_{max} = 9.6 \text{ kN}$;

$V_{min} = -9.6 \text{ kN}$

Deflection;

$\delta_{max} = 6.1 \text{ mm}$;

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

Maximum reaction at support A;

$R_{A_{max}} = 9.6 \text{ kN}$;

$R_{A_{min}} = 9.6 \text{ kN}$

Unfactored dead load reaction at support A;

$R_{A_{Dead}} = 3.4 \text{ kN}$

Unfactored imposed load reaction at support A;

$R_{A_{Imposed}} = 3 \text{ kN}$

Maximum reaction at support B;

$R_{B_{max}} = 9.6 \text{ kN}$;

$R_{B_{min}} = 9.6 \text{ kN}$

Unfactored dead load reaction at support B;

$R_{B_{Dead}} = 3.4 \text{ kN}$

Unfactored imposed load reaction at support B;

$R_{B_{Imposed}} = 3 \text{ kN}$

Section details

Section type;

UC 152x152x23 (BS4-1)

Steel grade;

S275

From table 9: Design strength p_y

Thickness of element;

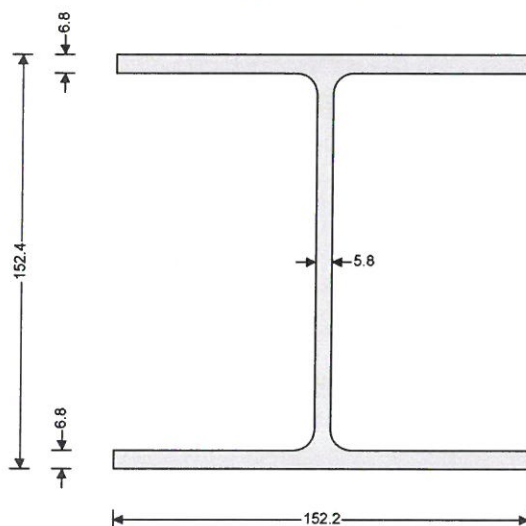
$\max(T, t) = 6.8 \text{ mm}$

Design strength;

$p_y = 275 \text{ N/mm}^2$

Modulus of elasticity;

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has 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;

$K_{LT,A} = 1.20; + 2 \times D$

$K_{LT,B} = 1.20; + 2 \times D$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section;

$d = 123.6 \text{ mm}$

$d / t = 21.3 \times \epsilon \leq 80 \times \epsilon$;

Class 1 plastic



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Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 76.1 \text{ mm}$$

$$b / T = 11.2 \times \epsilon \leq 15 \times \epsilon;$$

Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 9.6 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 884 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 145.8 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 18.1 \text{ kNm}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange;

$$\beta_{2f} = 10 \times \epsilon = 10$$

Limiting value for class 3 semi-compact flange;

$$\beta_{3f} = 15 \times \epsilon = 15$$

Limiting value for class 2 compact web;

$$\beta_{2w} = 100 \times \epsilon = 100$$

Limiting value for class 3 semi-compact web;

$$\beta_{3w} = 120 \times \epsilon = 120$$

Effective plastic modulus - cl.3.5.6.2

$$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min(\frac{((\beta_{3w} / (d / t))^2 - 1)}{((\beta_{3w} / \beta_{2w})^2 - 1)}, \frac{((\beta_{3f} / (b / T) - 1)}{(\beta_{3f} / \beta_{2f} - 1)})), S_{xx}) = 176245 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx}) = 48.5 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = 1.2 \times L_{s1} + 2 \times D = 4985 \text{ mm}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = 134.800$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;

$$u = 0.840$$

Torsional index;

$$x = 20.701$$

Slenderness factor;

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.752$$

Ratio - cl.4.3.6.9;

$$\beta_w = S_{eff} / S_{xx} = 0.968$$

Equivalent slenderness - cl.4.3.6.7;

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 83.799$$

Limiting slenderness - Annex B.2.2;

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

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

Bending strength - Section 4.3.6.5

Robertson constant;

$$\alpha_{LT} = 7.0$$

Perry factor;

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.346$$

Euler stress;

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 288.1 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 331.5 \text{ N/mm}^2$$

Bending strength - Annex B.2.1;

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 156.4 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment;

$$M_2 = 9.2 \text{ kNm}$$

Moment at centre-line of segment;

$$M_3 = 18.1 \text{ kNm}$$

Moment at three quarter point of segment;

$$M_4 = 9.2 \text{ kNm}$$

Maximum moment in segment;

$$M_{abs} = 18.1 \text{ kNm}$$

Maximum moment governing buckling resistance;

$$M_{LT} = M_{abs} = 18.1 \text{ kNm}$$



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Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.852$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{eff} = 27.6 \text{ kNm}$$

$$M_b / m_{LT} = 32.3 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 10.833 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 6.053 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

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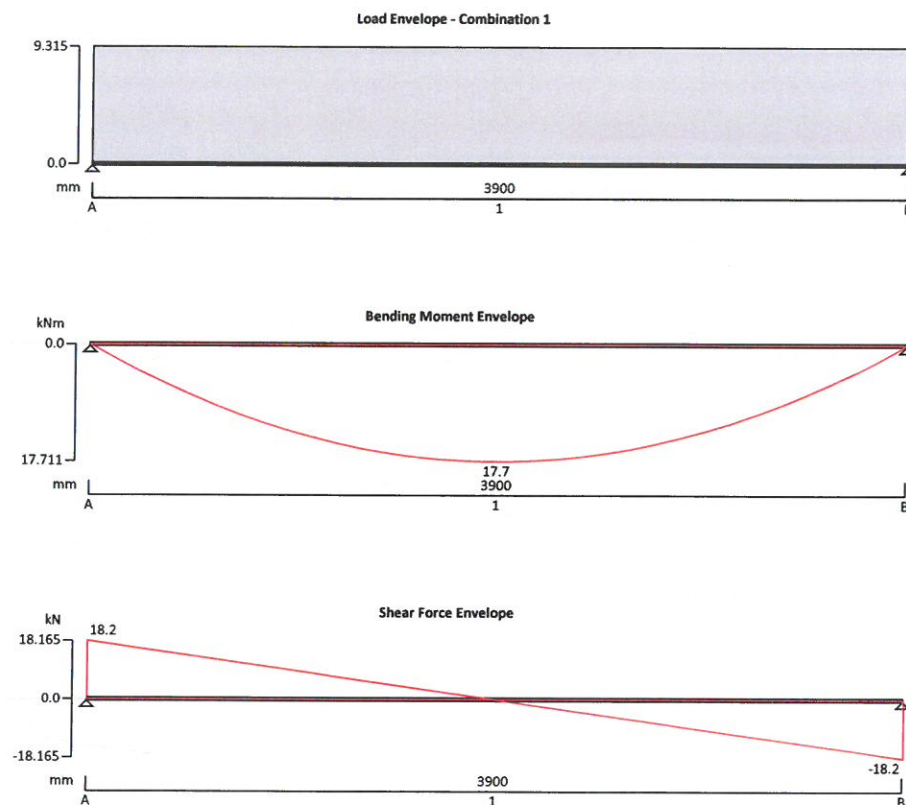
BEAM DESIGN - CASE 2

Beam spans 3900 mm and carries the following loads –
5000 mm of roof applied as udl

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 $\times 1$

roof - Dead full UDL 3 kN/m

roof - Imposed full UDL 3 kN/m

Load combinations

Load combination 1 - dead+imp

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$



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Support B

Dead $\times 1.40$

Imposed $\times 1.60$

Analysis results

Maximum moment;

$M_{max} = 17.7 \text{ kNm};$

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

Maximum moment span 1 segment 1;

$M_{s1_seg1_max} = 17.7 \text{ kNm};$

$M_{s1_seg1_min} = 0 \text{ kNm}$

Maximum moment span 1 segment 2;

$M_{s1_seg2_max} = 17.7 \text{ kNm};$

$M_{s1_seg2_min} = 0 \text{ kNm}$

Maximum shear;

$V_{max} = 18.2 \text{ kN};$

$V_{min} = -18.2 \text{ kN}$

Maximum shear span 1 segment 1;

$V_{s1_seg1_max} = 18.2 \text{ kN};$

$V_{s1_seg1_min} = 0 \text{ kN}$

Maximum shear span 1 segment 2;

$V_{s1_seg2_max} = 0 \text{ kN};$

$V_{s1_seg2_min} = -18.2 \text{ kN}$

Deflection segment 3;

$\delta_{max} = 7.3 \text{ mm};$

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

Maximum reaction at support A;

$R_{A_max} = 18.2 \text{ kN};$

$R_{A_min} = 18.2 \text{ kN}$

Unfactored dead load reaction at support A;

$R_{A_Dead} = 6.3 \text{ kN}$

Unfactored imposed load reaction at support A;

$R_{A_Imposed} = 5.9 \text{ kN}$

Maximum reaction at support B;

$R_{B_max} = 18.2 \text{ kN};$

$R_{B_min} = 18.2 \text{ kN}$

Unfactored dead load reaction at support B;

$R_{B_Dead} = 6.3 \text{ kN}$

Unfactored imposed load reaction at support B;

$R_{B_Imposed} = 5.9 \text{ kN}$

Section details

Section type;

UC 152x152x23 (BS4-1)

Steel grade;

S275

From table 9: Design strength p_y

Thickness of element;

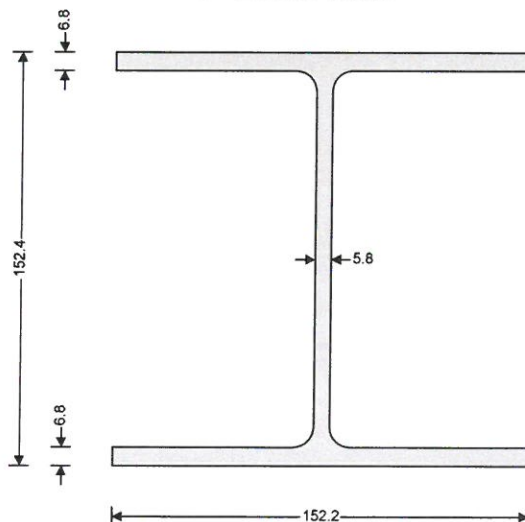
$\max(T, t) = 6.8 \text{ mm}$

Design strength;

$p_y = 275 \text{ N/mm}^2$

Modulus of elasticity;

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports plus midspan

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;

$K_{LT,A} = 1.20; + 2 \times D$

$K_{LT,B} = 1.20; + 2 \times D$

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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;

$$d = 123.6 \text{ mm}$$

$$d / t = 21.3 \times \varepsilon \leq 80 \times \varepsilon; \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 76.1 \text{ mm}$$

$$b / T = 11.2 \times \varepsilon \leq 15 \times \varepsilon; \quad \text{Class 3 semi-compact}$$

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 18.2 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 884 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 145.8 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 1 - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_seg1_max}), \text{abs}(M_{s1_seg1_min})) = 17.7 \text{ kNm}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange;

$$\beta_{2f} = 10 \times \varepsilon = 10$$

Limiting value for class 3 semi-compact flange;

$$\beta_{3f} = 15 \times \varepsilon = 15$$

Limiting value for class 2 compact web;

$$\beta_{2w} = 100 \times \varepsilon = 100$$

Limiting value for class 3 semi-compact web;

$$\beta_{3w} = 120 \times \varepsilon = 120$$

Effective plastic modulus - cl.3.5.6.2

$$S_{\text{eff}} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min(\frac{(\beta_{3w} / (d / t))^2 - 1}{(\beta_{3w} / \beta_{2w})^2 - 1}, \frac{[(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]}{S_{xx}}), S_{xx}) = 176245 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{\text{eff}}, 1.2 \times p_y \times Z_{xx}) = 48.5 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = ((1.2 + 1.0) \times L_{s1_seg1} + 2 \times D) / 2 = 2297 \text{ mm}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = 62.127$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;

$$u = 0.840$$

Torsional index;

$$x = 20.701$$

Slenderness factor;

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.911$$

Ratio - cl.4.3.6.9;

$$\beta_w = S_{\text{eff}} / S_{xx} = 0.968$$

Equivalent slenderness - cl.4.3.6.7;

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 46.774$$

Limiting slenderness - Annex B.2.2;

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

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

Bending strength - Section 4.3.6.5

Robertson constant;

$$\alpha_{LT} = 7.0$$

Perry factor;

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.087$$

Euler stress;

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 924.8 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 640.2 \text{ N/mm}^2$$

Bending strength - Annex B.2.1;

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 245.8 \text{ N/mm}^2$$



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Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = 7.7$ kNm

Moment at centre-line of segment; $M_3 = 13.3$ kNm

Moment at three quarter point of segment; $M_4 = 16.6$ kNm

Maximum moment in segment; $M_{abs} = 17.7$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = 17.7$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.781$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = 43.3$ kNm

$$M_b / m_{LT} = 55.4 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 10.833$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 7.319$ mm

PASS - Maximum deflection does not exceed deflection limit

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Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = 7.7 \text{ kNm}$
 Moment at centre-line of segment; $M_3 = 13.3 \text{ kNm}$
 Moment at three quarter point of segment; $M_4 = 16.6 \text{ kNm}$
 Maximum moment in segment; $M_{abs} = 17.7 \text{ kNm}$
 Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = 17.7 \text{ kNm}$
 Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.781$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = 43.3 \text{ kNm}$
 $M_b / m_{LT} = 55.4 \text{ kNm}$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 10.833 \text{ mm}$

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 7.319 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

152X152X23 UC SATISFACTORY



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Project Tottington Manor Farm, Turton Road, Tottington				Job Ref. 8674	
Section				Sheet no./rev. 12	
Calc. by AP	Date 17/11/2022	Chk'd by MP	Date 17/11/2022	App'd by	Date

SUMMARY

As part of the proposed conversion works the existing asbestos wall cladding sheets will be replaced with plastic coated steel sheets with insulation backing.

The weight of the asbestos sheets varies from 12 – 15 kg/m² depending on thickness and the proposed replacement panels have a weight less than 1.5 kg/m².

Calculations on the overall load bearing capacity of the steel framework confirm no increase in stresses or bearing pressures on the existing foundations or sub strata and the proposed changes have no adverse effect on the structural capability of the building.

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22 NOVEMBER 2022