STRUCTURAL ENGINEERS REPORT

Subject	Spring Lane Farm Shop Mapperley Plains Nottingham NG3 5RQ
Client	Spring Lane Farm Shop Ltd
Our Ref.	P21229
Inspection Date	14 September 2023
Report Date	19 September 2023
Prepared by	Giles Ward MA CEng MICE Director giles.ward@hwaconsulting.co.uk For and on behalf of Howard Ward Associates Ltd

Version History

	Date	Reason	By
1.0	19/09/2023	First issue	GRW

Contents

Summary

Purpose of report: To advise on the structural condition of an existing agricultural building, with a view to its possible conversion to a retail coffee shop

Conclusion: Subject to some minor improvements as noted below, the building is structurally sound and capable of conversion as proposed.

1. General/Background

1.1 For the avoidance of doubt, this report relates to the building outlined in red on the extract from the Mapmatic topographical survey of the site dated 12 October 2021, attached as Appendix A . Surrounding/attached buildings have not been inspected in detail and are excluded from our scope.

1.2 The age of the building is not known, although it does not appear to be especially recent.

1.3 In describing the building, all references to front, rear, left and right are as viewed from Spring Lane.

2. Topography/Geology

2.1 The site slopes downwards from front to rear. The gradient is however insufficient to give rise to any concerns relating to slope stability and there are no other relevant topographical features nearby.

2.2 Data from the British Geological Survey indicates the underlying bedrock is from the Gunthorpe Member, consisting of mudstone from the Triassic period. No superficial strata are recorded and it is therefore probable that material close to the surface consists of weathered mudstone. Such material would be expected to include a substantial clay content and to have a satisfactory bearing capacity for foundations.

3. Inspection

3.1 The building superstructure comprises a series of steel portal frames spaced at approximately 4.6m centres and spanning around 13.5m. Timber purlins span between the steel frames and support fibre cement sheeting. Given the age of the building, the sheeting is assumed to contain asbestos fibres which are hazardous to health, although it is stressed that no testing has been undertaken to confirm this.

3.2 The steelwork appears generally straight and true, with columns vertical (within normal tolerances) where accessible. There are occasional very minor and localised areas of impact damage to the steel rafters and some slight surface corrosion, but nothing that might compromise structural integrity.

3.3 Most of the timber purlins exhibit a degree of sagging. This is typical of agricultural construction where timbers are unlikely to be generously sized and hence will be relatively highly stressed, with sagging progressing over time due to the natural “creep” of timber, especially at times of high loading due to snow etc. None however appeared to be in any immediate danger of collapse.

3.4 The concrete floor was only partially visible at the time of inspection due to the materials and other items stored in the building. It appeared to be somewhat uneven, consistent with agricultural usage, but to be reasonably intact with no significant cracking.

3.5 Towards the rear, floor level within the building is above that of the surrounding ground due to the sloping site. Along the rear part of the side elevation, the floor and fill material beneath it is constrained by brickwork between ground level and floor level. At the rear, block work walls fulfil the same function, some parts of which extend above ground with only a small section of fibre cement cladding above them. There is a slight outwards lean on the central section of the blockwork of around 1:60.

3.6 The attached photographs illustrate the above.

4. Numerical Appraisal

4.1 In order to confirm the adequacy of both the steel frames and the purlins, check calculations have been prepared based on measurements and observations taken on site. These are attached as Appendix B.

5. Discussion/Recommendations

5.1 It is assumed that a new roof would be required as part of the conversion, in conjunction with which it would be straightforward to replace and upgrade the existing purlins, hence eliminating any concerns in relation to sagging.

5.2 The outwards lean on the rear blockwork is relatively minor. It might nonetheless be prudent to re-clad this localised section of the building such that the affected blockwork could be taken down to avoid any residual concerns in this regard.

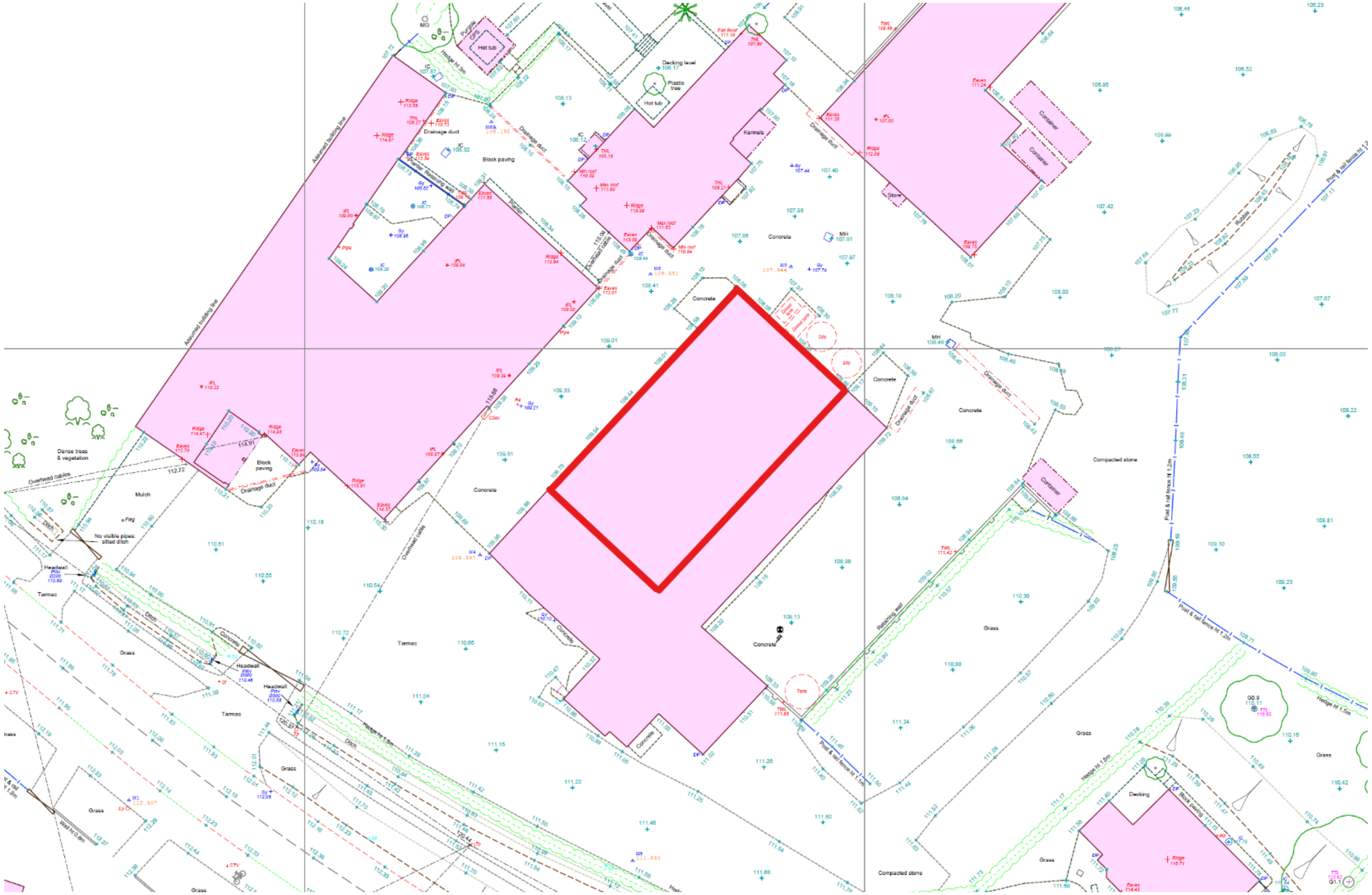
5.3 All other aspects of the building are structurally suitable to provide a sound basis for the proposed conversion, although it is assumed that other changes/upgrades would in any event be required as part of the conversion process.

1.– General view from rear left

2.– General internal view

3.– Underside of roof, note purlin distortions

Appendix A – Building Identification



Appendix B - Calculations

ROOF LOADING (PITCHED SHEETED ROOF)

Roof slope θ = **15.0** °

Dead load

Sheeting — $Roof_{D1}$ = **0.20** kN/m^2

Purlins — $Roof_{D4}$ = **0.08** kN/m^2

Steelwork — $Roof_{D5}$ = **0.05** kN/m^2

Services — $Roof_{D6}$ = **0.10** kN/m^2

Total dead load on slope

$Roof_{DL\_sroof}$ = sum($Roof_{D1}$,$Roof_{D4}$,$Roof_{D5}$,$Roof_{D6}$) = **0.43** kN/m^2

Total dead load on plan — $Roof_{DL}$ = $Roof_{DL\_sroof}$ / cos(θ) = **0.44** kN/m^2

SNOW LOADING (EN1991)

SNOW LOADING

In accordance with EN1991-1-3:2003+A1:2015 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex NA+A1:2015 to BS EN 1991-1-3:2003+A1:2015 incorporating Corrigendum No.1

Tedds calculation version 1.0.13

Characteristic ground snow load

Location — Nottingham

Site altitude above sea level (user modified value) — A = **108** m

Zone number — Z = **3.0**

Density of snow — γ = **2.00** kN/m^3

Characteristic ground snow load — s_k = ((0.15 + (0.1 $\times$ Z + 0.05)) + ((A - 100m) / 525m)) $\times$ 1kN/m^2 = **0.52** kN/m^2

Exposure coefficient (Normal) — C_e = **1.0**

Thermal coefficient — C_t = **1.0**

Snow fence — Not present

Building details

Roof type — Duopitch

Width of roof (left on elevation) — b_1 = **7.00** m

Width of roof (right on elevation) — b_2 = **7.00** m

Slope of roof (left on elevation) — α_1 = **15.00** deg

Slope of roof (right on elevation) — α_2 = **15.00** deg

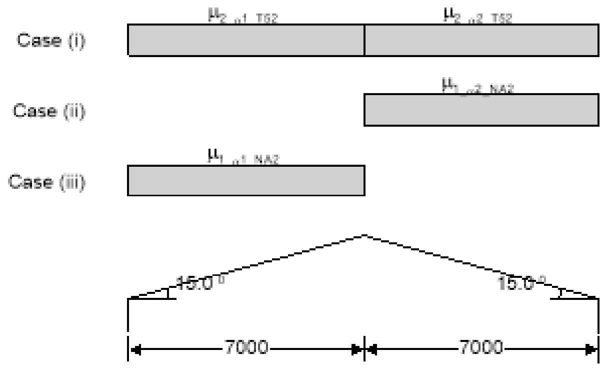
Shape coefficients

Shape coefficient roof (Table 5.2) — $\mu_{2\_\alpha1\_T52}$ = **0.80**

Shape coefficient roof (Table 5.2) — $\mu_{2\_\alpha2\_T52}$ = **0.80**

Shape coefficient roof (Table UK NA.2) — $\mu_{1\_\alpha1\_NA2}$ = **0.80**

Shape coefficient roof (Table UK NA.2) — $\mu_{1\_\alpha2\_NA2}$ = **0.80**



Elevation on gable roof

Shape coef	Coef	Loading (kN/m^2)
$\mu_{2\_\alpha1\_T52}$	0.800	0.41
$\mu_{2\_\alpha2\_T52}$	0.800	0.41
$\mu_{1\_\alpha1\_NA2}$	0.800	0.41
$\mu_{1\_\alpha2\_NA2}$	0.800	0.41

Loadcase 1 Table 5.2

Loading to roof 1 (LHS) $s_{1\_1} = \mu_{2\_\alpha1\_T52} \times C_e \times C_t \times s_k =$ **0.41** kN/m^2

Loading to roof 2 (RHS) $s_{2\_1} = \mu_{2\_\alpha2\_T52} \times C_e \times C_t \times s_k =$ **0.41** kN/m^2

Loadcase 2 UK NA.2

Loading to roof 1 (LHS) $s_{1\_2} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

Loading to roof 2 (RHS) $s_{2\_2} = \mu_{1\_\alpha2\_NA2} \times C_e \times C_t \times s_k =$ **0.41** kN/m^2

Loadcase 3 UK NA.2

Loading to roof 1 (LHS) $s_{1\_3} = \mu_{1\_\alpha1\_NA2} \times C_e \times C_t \times s_k =$ **0.41** kN/m^2

Loading to roof 2 (RHS) $s_{2\_3} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

PURLIN

Spacing estimated on site at 1.35m, check for 1.4m:

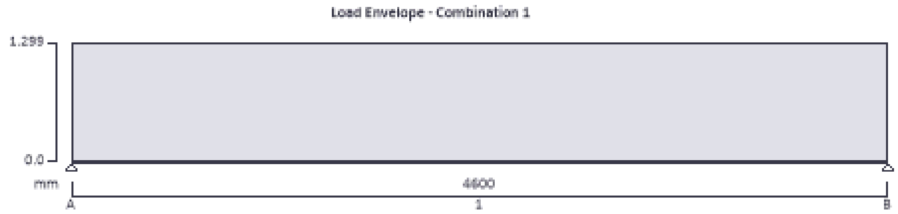
$Roof_{D1} \times$ 1.4m = **0.28** kN/m

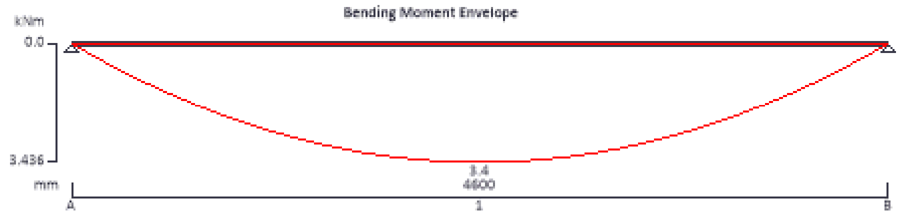
$s_{1\_1} \times$ 1.4m = **0.58** kN/m

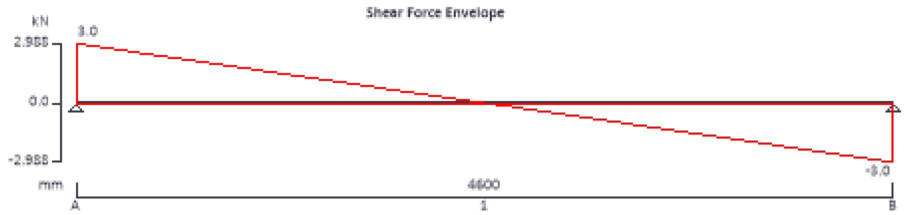
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

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 1.7.04







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 0.280 kN/m

Variable full UDL 0.580 kN/m

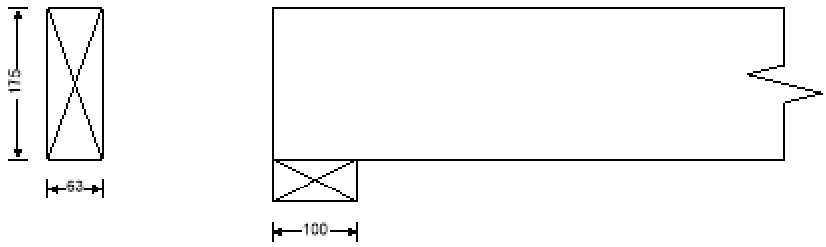
Load combinations

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

	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **3.436** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.436** kNm	
Maximum shear	F_{max} = **2.988** kN	F_{min} = **-2.988** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.988** kN	
Total load on beam	W_{tot} = **5.976** kN	
Reactions at support A	R_{A_max} = **2.988** kN	R_{A_min} = **2.988** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.731** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1.334** kN	
Reactions at support B	R_{B_max} = **2.988** kN	R_{B_min} = **2.988** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.731** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.334** kN	



Timber section details

Breadth of timber sections	b = **63** mm
Depth of timber sections	h = **175** mm
Number of timber sections in member	N = **1**
Overall breadth of timber member	b_b = N × b = **63** mm
Inclination of section	θ = **15.0** deg
Timber strength class - EN 338:2016 Table 1	**C24**

Member details

Load duration - cl.2.3.1.2	**Short-term**
Service class of timber - cl.2.3.1.3	**1**
Length of span	L_{s1} = **4600** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **11025** mm^2
Section modulus	W_y = N × b × h^2 / 6 = **321563** mm^3
	W_z = h × (N × b)^2 / 6 = **115763** mm^3
Second moment of area	I_y = N × b × h^3 / 12 = **28136719** mm^4
	I_z = h × (N × b)^3 / 12 = **3646519** mm^4
Radius of gyration	r_y = √(I_y / A) = **50.5** mm
	r_z = √(I_z / A) = **18.2** mm
Second moment of area of rotated section	$I_θ$ = (I_y + I_z) / 2 + (I_y - I_z) × cos(2 × θ) / 2 = **26496186** mm^4

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3	γ_M = **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.900**

Deformation factor for service classes - Table 3.2 k_{def} = **0.600**

Depth factor for bending - exp.3.1 $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.1 $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.670**

Load configuration factor - exp.6.4 $k_{c.90}$ = **1.000**

System strength factor - cl.6.6 k_{sys} = **1.000**

Lateral buckling factor - cl.6.3.3(5) k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{B\_max} / (N \times b \times L_b)$ = **0.474** N/mm²

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.731** N/mm²

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.274**

PASS - Design compressive strength exceeds design compressive stress at bearing

Biaxial bending - cl 6.1.6

Design bending stress in major (y-y) axis $\sigma_{m.y.d} = M \times \cos(\theta) / W_y$ = **10.321** N/mm²

Design bending stress in minor (z-z) axis $\sigma_{m.z.d} = M \times \sin(\theta) / W_z$ = **7.682** N/mm²

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **16.615** N/mm²

Combined bending checks - eq.6.11 & eq.6.12 $\sigma_{m.y.d} / f_{m.d} + k_m \times \sigma_{m.z.d} / f_{m.d}$ = **0.945**

$k_m \times \sigma_{m.y.d} / f_{m.d} + \sigma_{m.z.d} / f_{m.d}$ = **0.897**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.607** N/mm²

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.769** N/mm²

$\tau_d / f_{v.d}$ = **0.219**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 0.005 \times L_{s1}$ = **23.000** mm

Instantaneous deflection due to permanent load δ_{instG} = **6.490** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **10.385** mm

Instantaneous deflection due to variable load δ_{instQ} = **11.844** mm

Factor for quasi-permanent variable action ψ_2 = **0**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **11.844** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **22.228** mm

$\delta_{fin} / \delta_{lim}$ = **0.966**

PASS - Total final deflection is less than the deflection limit

PORTAL FRAME ANALYSIS & DESIGN (EN1993)

STEEL MEMBER 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

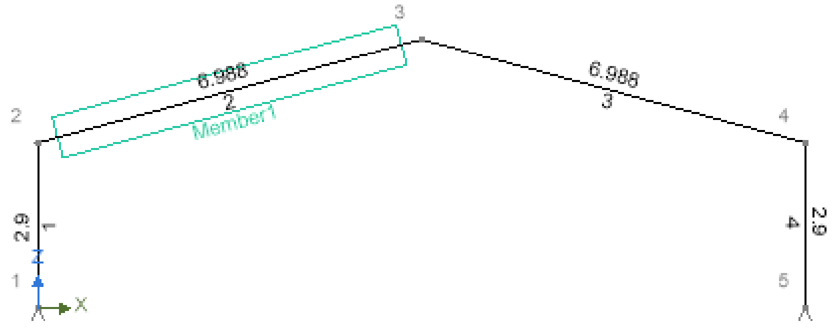
Tedds calculation version 4.4.08

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - Steel (EC3)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area	Moment of inertia		Shear area parallel to	
		Major	Minor	Minor	Major
	(cm²)	(cm⁴)	(cm⁴)	(cm²)	(cm²)
UB 254x102x22	28	2841	119.3	14.5	12.4
UB 254x102x25	32	3414.6	148.7	15.4	15.4

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. kNm/°
1	0	0	Fixed	Fixed	Free		0	0	0	0
2	0	2.9	Free	Free	Free		0	0	0	0
3	6.75	4.709	Free	Free	Free		0	0	0	0
4	13.5	2.9	Free	Free	Free		0	0	0	0
5	13.5	0	Fixed	Fixed	Free		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	2.9	1	2	UB 254x102x22	Steel (EC3)	Fixed	Fixed	Fixed	
2	6.988	2	3	UB 254x102x25	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.988	3	4	UB 254x102x22	Steel (EC3)	Fixed	Fixed	Fixed	

Element	Length (m)	Nodes Start	Nodes End	Section	Material	Releases Start moment	Releases End moment	Releases Axial	Rotated
4	2.9	4	5	UB 254x102x22	Steel (EC3)	Fixed	Fixed	Fixed	

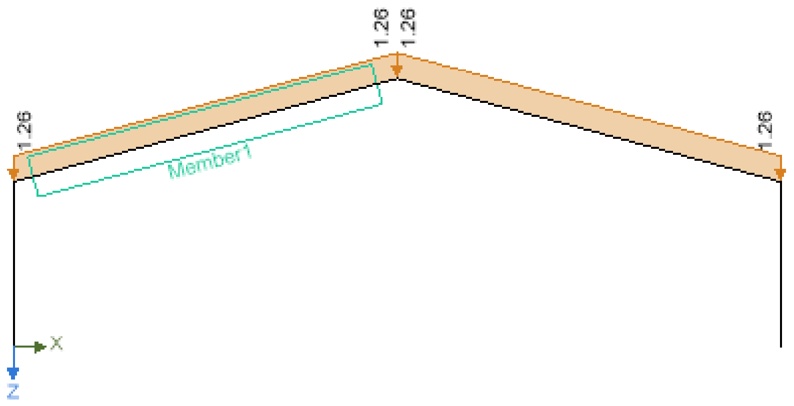
Members

Name	Elements Start	Elements End
Member1	2	2

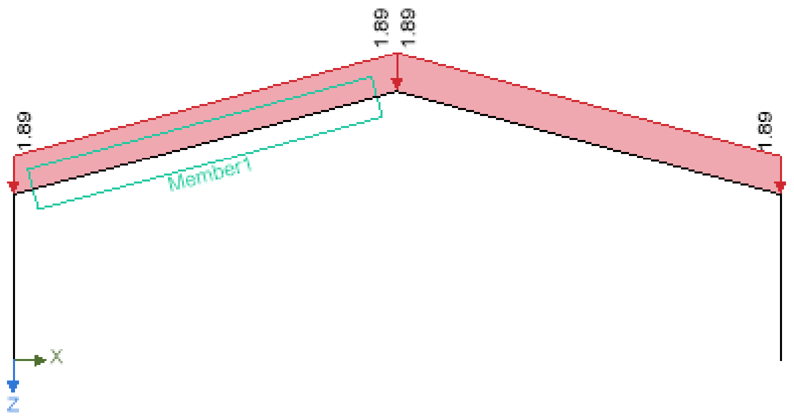
Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

Element Loads

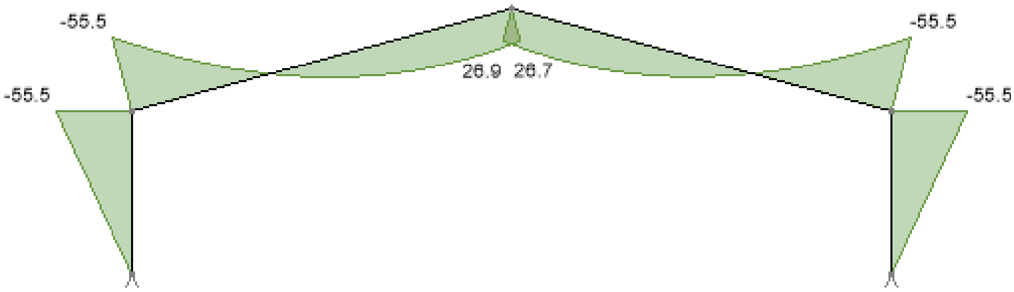
Element	Load case	Load Type	Orientation	Description
2	Permanent	UDL	GlobalZ	1.26 kN/m

Element	Load case	Load Type	Orientation	Description
3	Permanent	UDL	GlobalZ	1.26 kN/m
2	Imposed	UDL	GlobalZ	1.89 kN/m
3	Imposed	UDL	GlobalZ	1.89 kN/m

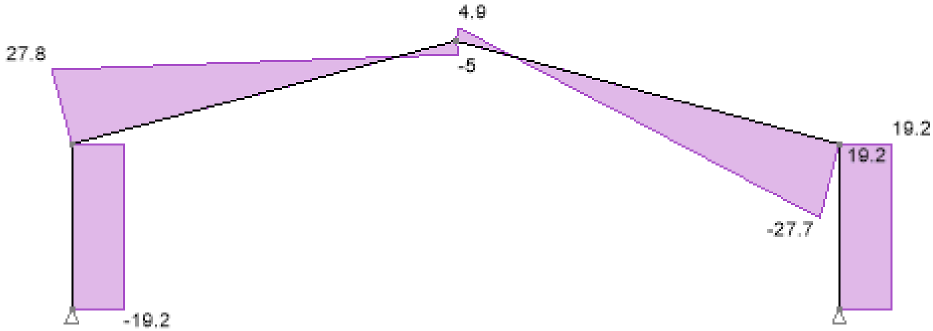
Results

Forces

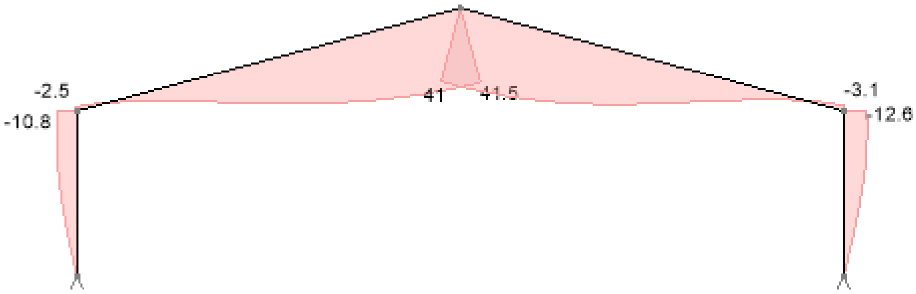
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

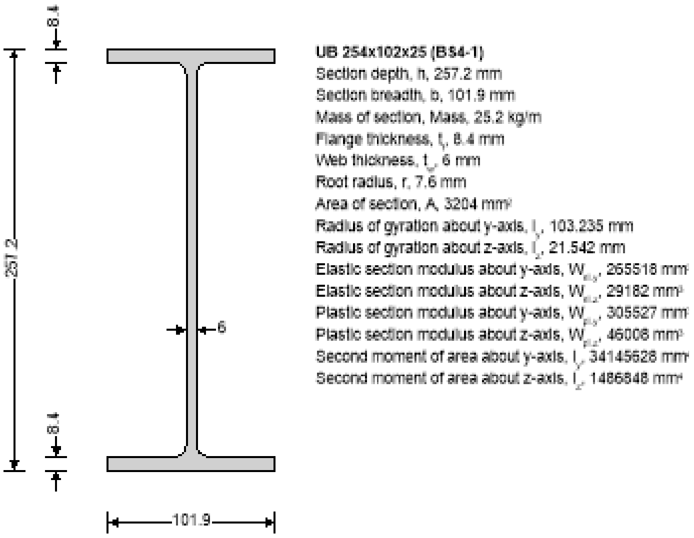
Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

<u>Member1 design</u>

Section details

Section type	UB 254x102x25 (BS4-1)
Steel grade - EN 10025-2:2004	S235
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) = \mathbf{8.4}$ mm
Nominal yield strength	$f_y = \mathbf{235}$ N/mm^2
Nominal ultimate tensile strength	$f_u = \mathbf{360}$ N/mm^2
Modulus of elasticity	$E = \mathbf{210000}$ N/mm^2



Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports only

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{1.00}$

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

Width of section — $c = d = \mathbf{225.2}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.543}$

$c / t_w = 37.5 = 37.5 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{40.3}$ mm

$c / t_f = 4.8 = 4.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force	$N_{Ed} = \mathbf{27.3}$ kN
Design resistance of section - eq 6.10	$N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = \mathbf{752.9}$ kN
	$N_{Ed} / N_{c,Rd} = \mathbf{0.036}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length	$L_{cr,y} = L_{m1\_s1} = \mathbf{6988}$ mm
Critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{1449.2}$ kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.721}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_y = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.814**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.838**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **630.8** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.043**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m1\_s1\_seg1\_R}$ = **6988** mm

Distance from shear centre to centroid in y axis y_0 = **0.0** mm

Distance from shear centre to centroid in z axis z_0 = **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **105.5** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **554** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2$ = **1**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **554** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **554** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **1.166**

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_T = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.344**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.497**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **374.2** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.073**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **240.4** mm η = **1.000**

$h_w / t_w = 40.1 = 40.1 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **27.8** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1670** mm²

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **226.6** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.123**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **55.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **71.8** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.774**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.588**

$C_1 = 1 / k_c^2 =$ **2.889**

Poissons ratio $\nu =$ **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)] =$ **80769** N/mm²

Unrestrained effective length $L = 0.5 \times L_{m1\_s1\_seg1\_B} =$ **3494** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **138.4** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.72**

Limiting slenderness ratio $\bar{\lambda}_{LT,0} =$ **0.4**

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 c

Imperfection factor - Table 6.3 $\alpha_{LT} =$ **0.49**

Correction factor for rolled sections $\beta =$ **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.773**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.813**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.797**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **1.000**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **71.8** kNm

$M_{y,Ed} / M_{b,y,Rd} =$ **0.774**

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **169.5** kN

$N_{Ed} / N_{y,lim} =$ **0.161**

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 $\psi_y = 24.174$ kNm / -55.54 kNm = **-0.435**

$\alpha_y = 13.03$ kNm / -55.54 kNm = **-0.235**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**

$\psi_{LT} = 24.174$ kNm / -55.54 kNm = **-0.435**

$\alpha_{LT} = 13.03$ kNm / -55.54 kNm = **-0.235**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y =$ **71.8** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y =$ **10.8** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **752.9** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.409**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.976**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.432**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.964**

PASS - Combined bending and compression checks are satisfied

Check design 5919 mm along span

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} =$ **26.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **71.8** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.374**

PASS - Design bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 ψ_y = 24.174 kNm / -55.54 kNm = **-0.435**

α_y = 13.03 kNm / -55.54 kNm = **-0.235**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**

ψ_{LT} = 24.174 kNm / -55.54 kNm = **-0.435**

α_{LT} = 13.03 kNm / -55.54 kNm = **-0.235**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y =$ **71.8** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y =$ **10.8** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **752.9** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.407**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.982**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.184**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.394**

PASS - Combined bending and compression checks are satisfied