The Old School
Kirkgate

Structural Calculations
Replacement Roof Structure
Document No: WA638-CAL01

May 2023

General Notes

- All setting out and finishes to architects details.
- All proprietary fixings/items to be installed in strict accordance with manufacturers guidance.
- All steel to steel connections to fabricator design and detail to loading information provided on LACE LTD sketches unless noted otherwise.
- All fabrication undertaken to execution class 2 (EXC-2).
- All external steelwork to be galvanised.
- Any steelwork below ground level to be bitumastic painted and encased in concrete with a minimum 50 mm cover to all sides.

Design Codes

BS 6399-1:1996	Loading for buildings. Code of practice for dead and imposed loads
BS 6399-2:1997	Loading for buildings. Code of practice for wind loads
BS 6399-3:1998	Loading for buildings. Code of practive for imposed roof loads
BS 5268-2:2002	Structural use of timber. Code of practice for permissible stress design
BS 5268-6.1:1996	Structural use of timber. Code of practice for racking resistance
BS 5268-4.1:1978	Structural use of timber. Recommendations for calculations fire resistance of timber
BS 5628-1:2005	Code of practice for the use of masonry. Structural use of unreinforced masonry
BS 5950-1:2000	Structural use of steelwork in building
BS 8004:1986	Code of practice for foundations

Cover Sheet

A1	Loading
B1 – B12	Structure Design
SK01	Scheme Sketch

Loadings

Roof (30^{O})

Dead

Roof Finishes	1.20
Battens, Felt, Rafters, Insulation	0.35
Ceilings	0.25
	Sum (plan) = 1.80 kN/m²
	Therefore Pitch = 2.10 kN/m²

Imposed

Roof Imposed Load- No Access **= 0.75 kN/m²**

Floors

Dead

Timber Floor Construction **= 0.50 kN/m²**

Imposed

Domestic Floor Load = **1.50 kN/m²**

Walls

Dead

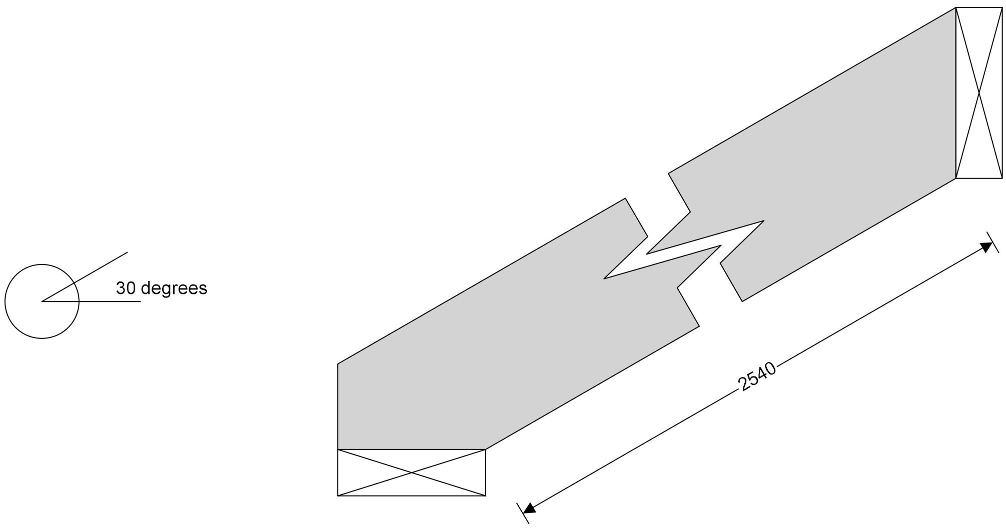
100mm stonework = **2.40 kN/m²**

100mm blockwork = **1.80 kN/m²**

PITCH RAFTERS

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	b = **47** mm
Depth of timber sections;	h = **150** mm
Rafter spacing;	s = **400** mm
Rafter slope;	α = **30.0** deg
Clear span of rafter on horizontal;	L_{clh} = **2200** mm
Clear span of rafter on slope;	$L_{cl} = L_{clh} / \cos(\alpha)$ = **2540** mm
Rafter span;	**Single span**
Timber strength class;	**C24**

Section properties

Cross sectional area of rafter;	$A = b \times h$ = **7050** mm^2
Section modulus;	$Z = b \times h^2 / 6$ = **176250** mm^3
Second moment of area;	$I = b \times h^3 / 12$ = **13218750** mm^4
Radius of gyration;	$r = \sqrt{(I / A)}$ = **43.3** mm

Loading details

Rafter self weight;	$F_j = b \times h \times \rho_{char} \times g_{acc}$ = **0.02** kN/m
Dead load on slope;	F_d = **1.80** kN/m^2
Imposed load on plan;	F_u = **0.75** kN/m^2
Imposed point load;	F_p = **0.90** kN

Modification factors

Section depth factor;	$K_7 = (300\text{ mm} / h)^{0.11}$ = **1.08**
Load sharing factor;	K_8 = **1.10**

Consider long term load condition

Load duration factor;	K_3 = **1.00**
Total UDL perpendicular to rafter;	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.644** kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] =$ **7** mm

Effective span; $L_{eff} = L_{cl} + L_b =$ **2547** mm

Check bending stress

Bending stress parallel to grain; $\sigma_m =$ **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.904** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) =$ **2.965** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c =$ **7.900** N/mm^2

Minimum modulus of elasticity; $E_{min} =$ **7200** N/mm^2

;;;Compression member factor; $K_{12} =$ **0.67**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} =$ **5.865** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) =$ **0.403** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **20.539** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) =$ **0.980**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} =$ **0.409**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau =$ **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.781** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) =$ **0.175** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} =$ **7.641** mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) =$ **2.474** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) =$ **0.132** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s =$ **2.605** mm

PASS - Total deflection within permissible limits

<u>**Consider medium term load condition**</u>

Load duration factor; $K_3 =$ **1.25**

Total UDL perpendicular to rafter; $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) =$ **0.869** kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] =$ **9** mm

Effective span; $L_{eff} = L_{cl} + L_b =$ **2549** mm

Check bending stress

Bending stress parallel to grain; $\sigma_m =$ **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **11.130** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) =$ **4.008** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c =$ **7.900** N/mm^2

Minimum modulus of elasticity; $E_{min} =$ **7200** N/mm^2

;;;Compression member factor; $K_{12} =$ **0.64**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **7.003** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.545** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **20.502** N/mm^2

Euler coefficient; $K_{eu} = 1 – (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.974**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.447**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.976** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.236** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **7.648** mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **3.349** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.178** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **3.528** mm

PASS - Total deflection within permissible limits

<u>Consider short term load condition</u>

Load duration factor; K_3 = **1.50**

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.644** kN/m

Notional bearing length; $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **10** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **2550** mm

Check bending stress

Bending stress parallel to grain; σ_m = **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **13.355** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z)$ = **5.792** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; σ_c = **7.900** N/mm^2

Minimum modulus of elasticity; E_{min} = **7200** N/mm^2

;;;Compression member factor; K_{12} = **0.61**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **7.999** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha)+3 \times \tan(\alpha))/(2 \times A) + F_p \times \sin(\alpha)/A$ = **0.468** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **20.488** N/mm^2

Euler coefficient; $K_{eu} = 1 – (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.979**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.501**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **1.172** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) =$ **0.341** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} =$ **7.650** mm

Bending deflection; $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) =$ **4.372** mm

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) =$ **0.257** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s =$ **4.630** mm

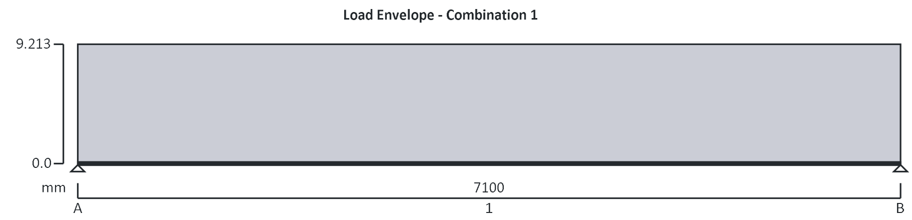
PASS - Total deflection within permissible limits

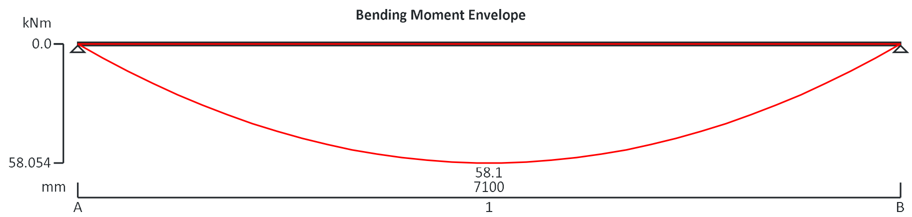
BEAM A (RIDGE)

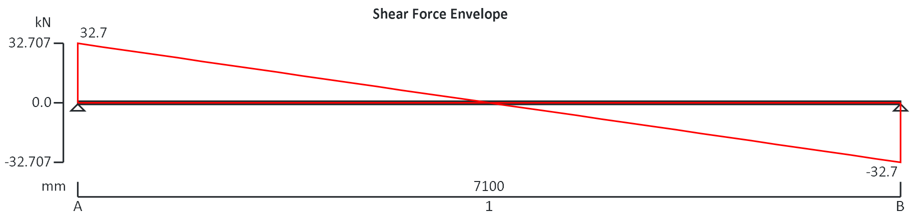
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 × 1
	2.2m Roof - Dead full UDL 4.62 kN/m
	2.2m Roof - Imposed full UDL 1.32 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **58.1** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **32.7** kN;	V_{min} = **-32.7** kN
Deflection;	δ_{max} = **10.4** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **32.7** kN;	R_{A_min} = **32.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **18** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **4.7** kN	
Maximum reaction at support B;	R_{B_max} = **32.7** kN;	R_{B_min} = **32.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **18** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **4.7** kN	

Section details

Section type; **UKB 305x165x46 (Tata Steel Advance)**

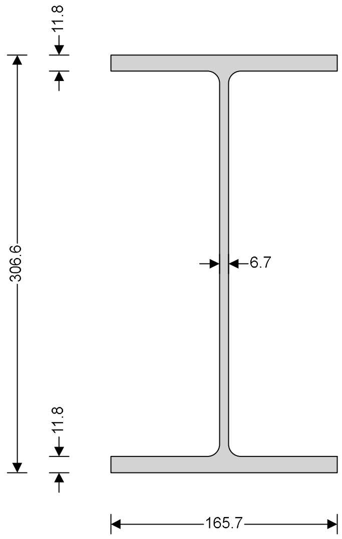
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **11.8** mm

Design strength; p_y = **355** N/mm²

Modulus of elasticity; E = **205000** N/mm²



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 × D

$K_{LT.B}$ = **1.20**; + 2 × D

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 = **265.2** mm

$d / t = 45.0 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 =$ **82.9** mm

$b / T = 8.0 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **32.7** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D =$ **2054** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v =$ **437.5** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **58.1** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **255.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.2 \times L_{s1} + 2 \times D =$ **9133** mm

Slenderness ratio; $\lambda = L_E / r_{yy} =$ **233.905**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u =$ **0.891**

Torsional index; $x =$ **27.145**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.679**

Ratio - cl.4.3.6.9; $\beta_W =$ **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **141.375**

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

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

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} =$ **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.778**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **101.2** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **267.5** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **78.8** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 =$ **43.5** kNm

Moment at centre-line of segment; $M_3 =$ **58.1** kNm

Moment at three quarter point of segment; $M_4 =$ **43.5** kNm

Maximum moment in segment; $M_{abs} =$ **58.1** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} =$ **58.1** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} =$ **56.7** kNm

$M_b / m_{LT} =$ **61.3** 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; $\delta_{lim} = L_{s1} / 360 = \mathbf{19.722}$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{10.423}$ mm

PASS - Maximum deflection does not exceed deflection limit

Padstone Check

Maximum beam reaction = 33 kN (ULS)

Permissible bearing stress = 1.90 N/mm^2

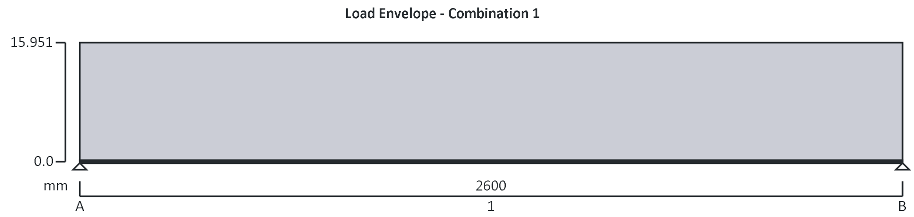
Provide 100 mm bearing onto a 440 x 102 x 215 mm concrete padstone to masonry bearings.

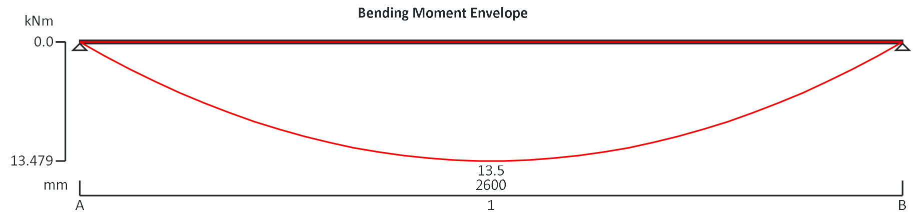
BEAM B (REPLACEMENT LINTEL)

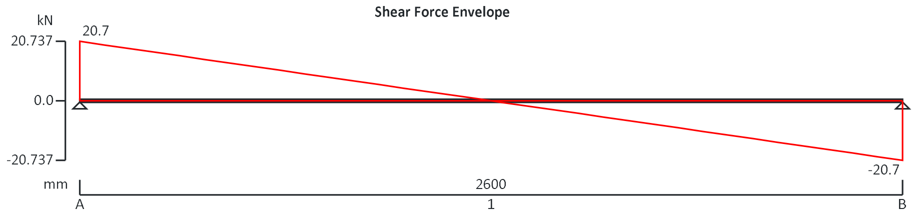
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 × 1
	1.4m Stone - Dead full UDL 5.04 kN/m
	Ridge Reaction - Dead full UDL 3.5 kN/m
	Ridge Reaction - Imposed full UDL 2.3 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60

Support B — Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **13.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **20.7** kN;	V_{min} = **-20.7** kN
Deflection;	δ_{max} = **2.6** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **20.7** kN;	R_{A_min} = **20.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **11.4** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3** kN	
Maximum reaction at support B;	R_{B_max} = **20.7** kN;	R_{B_min} = **20.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **11.4** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **3** kN	

Section details

Section type; **UKC 152x152x23 (Tata Steel Advance)**

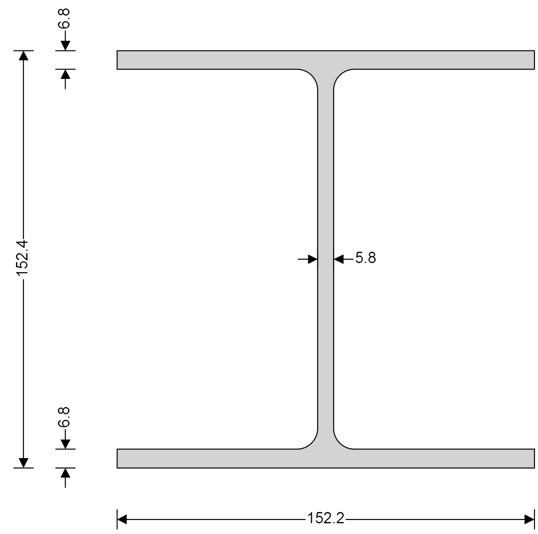
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **6.8** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity; E = **205000** 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 × D

$K_{LT.B}$ = **1.20**; + 2 × D

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** mm

$d / t = 24.2 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{76.1}$ mm

$b / T = 12.7 \times \varepsilon <= 15 \times \varepsilon$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{20.7}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{884}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{188.3}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{13.5}$ kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = 10 \times \varepsilon = \mathbf{8.801}$

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 15 \times \varepsilon = \mathbf{13.202}$

Limiting value for class 2 compact web; $\beta_{2w} = 100 \times \varepsilon = \mathbf{88.014}$

Limiting value for class 3 semi-compact web; $\beta_{3w} = 120 \times \varepsilon = \mathbf{105.617}$

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx}) = \mathbf{170473}$ mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx}) = \mathbf{60.5}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.2 \times L_{s1} + 2 \times D = \mathbf{3425}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{92.614}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.840}$

Torsional index; $x = \mathbf{20.701}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.841}$

Ratio - cl.4.3.6.9; $\beta_W = S_{eff} / S_{xx} = \mathbf{0.937}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{63.276}$

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; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.232}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{505.3}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{488.7}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{244.9}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{10.1}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{13.5}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{10.1}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{13.5}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{13.5}$ 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; $M_b = p_b \times S_{eff} =$ **41.8** kNm

$M_b / m_{LT} =$ **45.1** 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; $\delta_{lim} = L_{s1} / 360 =$ **7.222** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **2.57** mm

PASS - Maximum deflection does not exceed deflection limit

Padstone Check

Maximum beam reaction = 21 kN (ULS)

Permissible bearing stress = 1.90 N/mm^2

Provide 150 mm bearing onto a 215 x 140 x 215 mm concrete padstone to masonry bearings.