Existing Two-Storey Single Dwelling Semi-Detached Single Dwelling House to have Loft Conversion with Rear Facing Full Width Dormer, Single Storey Wrap Around Side & Rear Extension and First Floor Internal Openings to Spine Wall, Corridors and Orginal Flank & Rear Elevation. Chimney Breast to be removed at Ground & First Floor Level with a straight Beam to support stack above in new Second Floor. Steelwork required for Ridge Beam, Steel Post within En-Suite Studwork, Floor Beam supporting Post & Chimney Breast, Floor & Pitched Roof via Stud, Rear Dormer Face, Ground Floor Spine Wall opening, Orginal Rear Elevation opening and New Rear Façade openings. Steel & Concrete Lintels for all other openings. Padstones to be specified. Timber Joist Design to Second Floor and Flat Roofs. Double Joist/Flitch Trimmers to be provided for assumed 1.5m x 1.5m Skylights. Foundations to be standard Mass Concrete Strip Footing. Ground Bearing Concrete Slabs throughout Extensions.
Design to BS6399, BS5268, BS5628, BS5950 & BS8110.

LOADINGS	**DEAD LOAD**	**IMPOSED LOAD**	**TOTAL (kN/m^2)**
Pitched Roof – Joists, Battens, Felt, Tiles, Ply, Insulation, Ceiling Ties	1.35	0.75	2.1
Flat Roof – Joists, Battens, Felt, Tiles, Ply, Insulation	1.05	0.75	1.8
Timber Floor – Joists, Battens, Floorboards, Finishes, Insulation, Ceiling	0.7	1.5	2.2
100 Stud Wall with External Finishes	1.0	-	1.0
100 Stud Internal	0.7	-	0.7
225 Brickwork	4.8	-	4.8
Cavity Wall	3.5	-	3.5
112 Brickwork	2.5	-	2.5
100 Blockwork (3.6N)	1.0	-	1.0
New Skylight	1.2	-	1.2

Ridge Beam A

UDL DL = (2.7/2 x 1.05) + (1.8/2 x 1.35) = 2.7 kN/m
F.Roof P.Roof
UDL IL = (2.7/2 x 0.75) + (1.8/2 x 0.75) = 1.7 kN/m

Span = 5.0m

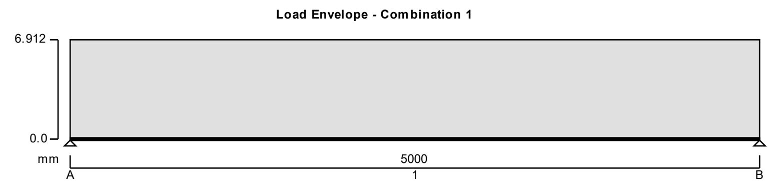
STEEL BEAM A ANALYSIS & DESIGN (BS5950)

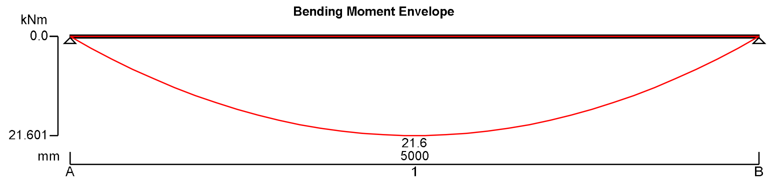
STEEL BEAM ANALYSIS & DESIGN (BS5950)

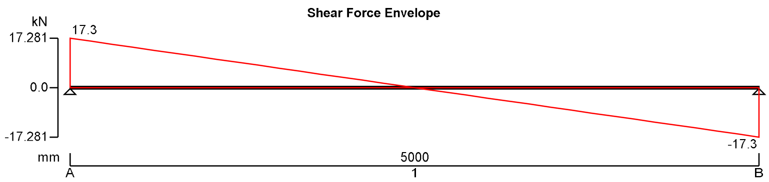
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05

Tedds Make Structures Ltd. 20 Gallery Close Northampton, Northants NN3 5NT	Project 45 Bedford Avenue, Hayes UB4 0DR				Job Ref. 210026	
	Section Loft Conversion & Rear/Side Extension				Sheet no./rev. 2	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.7 kN/m
	Imposed full UDL 1.7 kN/m

Load combinations

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

Analysis results

Maximum moment	M_{max} = **21.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **17.3** kN	V_{min} = **-17.3** kN
Deflection	δ_{max} = **10.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.3** kN	R_{A_min} = **17.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.3** kN	

Maximum reaction at support B	R_{B_max} = **17.3** kN	R_{B_min} = **17.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.3** kN	

Section details

Section type — **UKC 152x152x30 (Tata Steel Advance)**

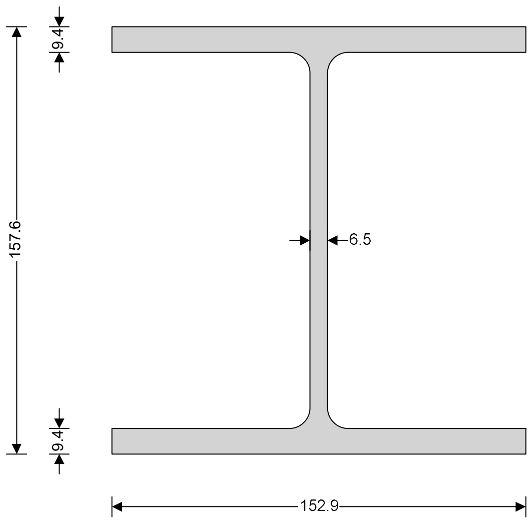
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **9.4** mm

Design strength — p_y = **275** 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 — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **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 = **123.6** mm

$d / t = 19.0 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **76.5** mm

$b / T = 8.1 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **17.3** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D = \mathbf{1024}$ mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v = \mathbf{169}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{21.6}$ kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{68.1}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1} = \mathbf{5000}$ mm

Slenderness ratio $\lambda = L_E / r_{yy} = \mathbf{130.643}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = \mathbf{0.849}$

Torsional index $x = \mathbf{15.999}$

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.693}$

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

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{76.829}$

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.310}$

λ_{LT} > λ_{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 $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.298}$

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{342.8}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{359.9}$ N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{172.1}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = \mathbf{16.2}$ kNm

Moment at centre-line of segment $M_3 = \mathbf{21.6}$ kNm

Moment at three quarter point of segment $M_4 = \mathbf{16.2}$ kNm

Maximum moment in segment $M_{abs} = \mathbf{21.6}$ kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs} = \mathbf{21.6}$ 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 $M_b = p_b \times S_{xx} = \mathbf{42.6}$ kNm

$M_b / m_{LT} = \mathbf{46.1}$ 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 $\delta_{lim} = \min(16 \text{ mm}, L_{s1} / 300) = \mathbf{16}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{10.661}$ mm

PASS - Maximum deflection does not exceed deflection limit

Post @ RA_1

F_x = 17.3 kN

Using 100 x 100 SHS Post…

M_x = 17.3 x 0.025 x 2.4 = 1.1 kN/m
2.5% H
Or
M_x = 17.3 x (0.1 + 0.05) = 2.6 kNm
Ecc.

STEEL POST @ RA1 DESIGN (BS5950)

STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.05

Section details

Section type **SHS 100x100x4.0 (Tata Steel Celsius)**

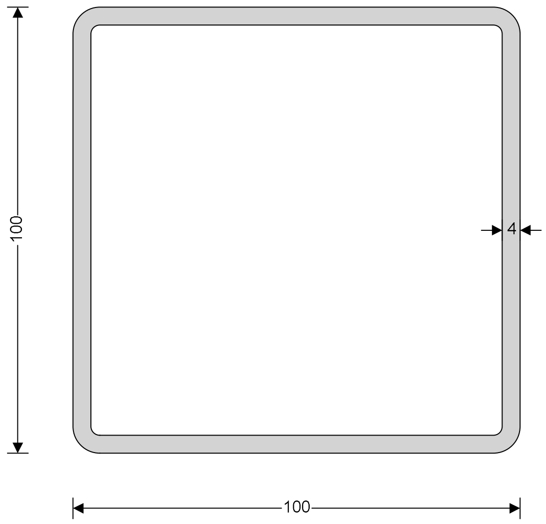
Steel grade **S275**

From table 9: Design strength p_y

Thickness of element t = **4.0** mm

Design strength p_y = **275** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Distance between major axis restraints L_x = **2400** mm

Distance between minor axis restraints L_y = **2400** mm

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.50**

Effective length factor for lateral-torsional buckling K_{LT} = **1.00**

Classification of cross sections - Section 3.5

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

Web - major axis - Table 12

Depth of section d= D - 3 × t = **88** mm

Stress ratios r1 = min(F_c / (2 × d × t × p_{yw}), 1) = **0.089**

r2 = F_c / (A × p_{yw}) = **0.041**

d / t = 22.0 × ε <= max(64 × ε / (1 + r1), 40 × ε) Class 1 plastic

Flange - major axis - Table 12

Width of section b = B - 3 × t = **88** mm

b / t = 22.0 × ε <= 40 × ε Class 3 semi-compact

Section is class 3 semi-compact

Moment capacity - Section 4.2.5

Design bending moment M = **2.6** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange β_{2f} = min(32 × ε, 62 × ε - 0.5 × d / t) = **32**

Limiting value for class 3 semi-compact flange β_{3f} = 40 × ε = **40**

Limiting value for class 2 compact web β_{2w} = max(80 × ε / (1 + r1), 40 × ε) = **73.438**

Limiting value for class 3 semi-compact web β_{3w} = max(120 × ε / (1 + 2 × r2), 40 × ε) = **110.82**

Effective plastic modulus - cl.3.5.6.3

S_{eff} = min(Z + (S - Z) × min([(β_{3w} / (d / t) - 1) / (β_{3w} / β_{2w} - 1)], [(β_{3f} / (b / t) - 1) / (β_{3f} / β_{2f} - 1)]), S) = **54444** mm^3

Moment capacity low shear - cl.4.2.5.2 M_c = min(p_y × S_{eff}, 1.2 × p_y × Z) = **15** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling L_E = 1.0 × L_y = **2400** mm

Slenderness ratio λ = L_E / r_{yy} = **61.432**

Equivalent slenderness - Annex B.2.6.1

Torsion constant J = **3611034** mm^4

γ_b = (1 - I_{yy} / I_{xx}) × (1 - J / (2.6 × I_{xx})) = **0.000**

ϕ_b = [S_{xx}^2 × γ_b / (A × J)]$^{0.5}$ = **0.000**

Ratio - cl.4.3.6.9 β_W = S_{eff} / S_{xx} = **1.000**

Equivalent slenderness λ_{LT} = 2.25 × √[ϕ_b × λ × β_W] = **0.000**

Limiting slenderness - Annex B.2.2 λ_{L0} = 0.4 × (π^2 × E / p_y)$^{0.5}$ = **34.310**

λ_{LT} < λ_{L0} - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength p_b = p_y = **275** N/mm^2

Buckling resistance moment M_b = p_b × S_{eff} = **15** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **17.3** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling L_{Ex} = L_x × K_x = **2400** mm

Slenderness ratio - cl.4.7.2 λ_x = L_{Ex} / r_{xx} = **61.432**

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{17.155}$

Strut curve - Table 23 a

Robertson constant $\alpha_x = \mathbf{2.0}$

Perry factor $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = \mathbf{0.089}$

Euler stress $p_{Ex} = \pi^2 \times E / \lambda_x^2 = \mathbf{536.1}$ N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = \mathbf{429.3}$ N/mm^2

Compressive strength - Annex C.1 $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = \mathbf{237.3}$ N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cx} = A \times p_{cx} = \mathbf{360.4}$ kN

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ey} = L_y \times K_y = \mathbf{3600}$ mm

Slenderness ratio - cl.4.7.2 $\lambda_y = L_{Ey} / r_{yy} = \mathbf{92.148}$

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{17.155}$

Strut curve - Table 23 a

Robertson constant $\alpha_y = \mathbf{2.0}$

Perry factor $\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = \mathbf{0.150}$

Euler stress $p_{Ey} = \pi^2 \times E / \lambda_y^2 = \mathbf{238.3}$ N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = \mathbf{274.5}$ N/mm^2

Compressive strength - Annex C.1 $p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = \mathbf{175.4}$ N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cy} = A \times p_{cy} = \mathbf{266.4}$ kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2 $F_c / (A \times p_y) + M / M_c = \mathbf{0.215}$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b $M_{LT} = M_x = \mathbf{2.60}$ kNm

Equivalent uniform moment factor for major axis flexural buckling

$m_x = \mathbf{1.000}$

$m_y = \mathbf{1.000}$

Buckling resistance checks - cl.4.8.3.3.3 $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx}) = \mathbf{0.226}$

$F_c / P_{cy} + 0.5 \times m_{LT} \times M_{LT} / M_{cx} = \mathbf{0.152}$

PASS - Member buckling resistance checks are satisfied

Beam B - Dormer Head Beams

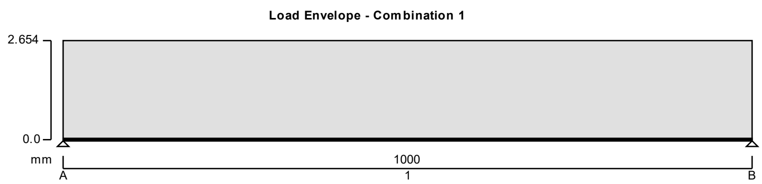
UDL DL = (2.8/2 x 1.05) = 1.5 kN/m
F.Roof
UDL IL = (2.8/2 x 0.75) = 1.1 kN/m

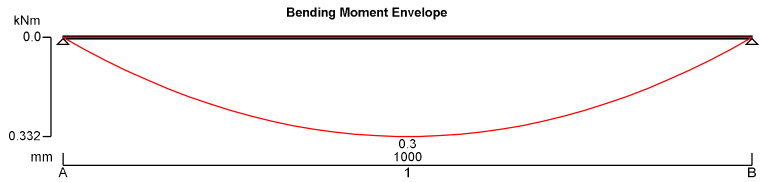
Span = 1.0m

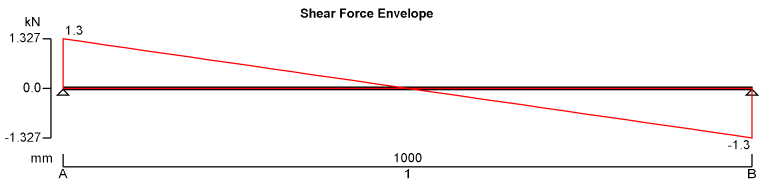
TIMBER BEAM B ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.500 kN/m
Imposed full UDL 1.100 kN/m

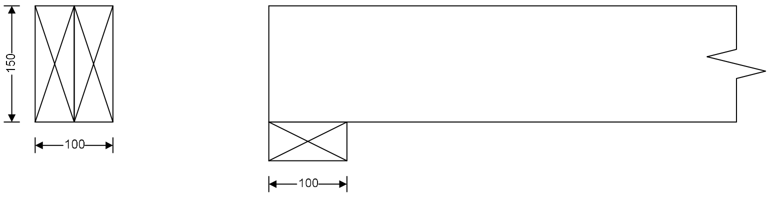
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00

	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **0.332** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.332** kNm	
Maximum shear	F_{max} = **1.327** kN	F_{min} = **-1.327** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.327** kN	
Total load on beam	W_{tot} = **2.654** kN	
Reactions at support A	R_{A_max} = **1.327** kN	R_{A_min} = **1.327** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.777** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.550** kN	
Reactions at support B	R_{B_max} = **1.327** kN	R_{B_min} = **1.327** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.777** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.550** kN	



Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **15000** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **375000** mm^3
	Z_y = h × (N × b)^2 / 6 = **250000** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **28125000** mm^4
	I_y = h × (N × b)^3 / 12 = **12500000** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **28.9** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.08**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19 **2.00**

Actual depth-to-breadth ratio $h / (N \times b) = \mathbf{1.50}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.420}$ N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.133}$ N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.055}$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{6.292}$ N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x = \mathbf{0.885}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.141}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.737}$ N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) = \mathbf{0.133}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.180}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 = \mathbf{6612}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) = \mathbf{3.000}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{0.186}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.064}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{0.250}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.083}$

PASS - Total deflection is less than permissible deflection

<u>Flat Roof Joists</u>

UDL DL = 1.05 – 0.15 = 0.9 kN/m^2

SWJ

UDL IL = 0.75 kN/m^2

Span = 2.7m

<u>DORMER FLAT ROOF TIMBER JOIST DESIGN (BS5268)</u>

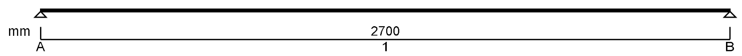
<u>TIMBER JOIST DESIGN (BS5268-2:2002)</u>

TEDDS calculation version 1.1.02

Joist details

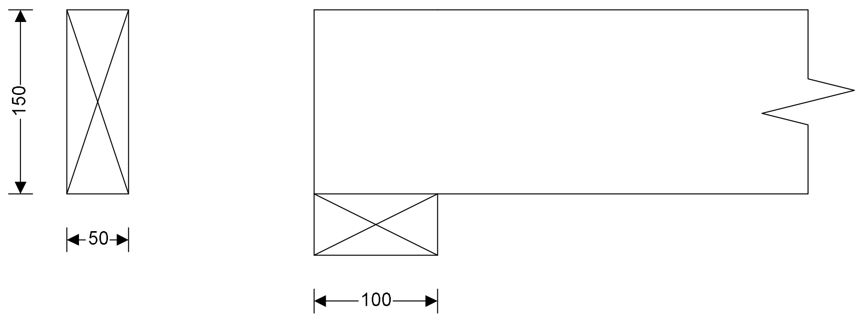
Joist breadth $b = \mathbf{50}$ mm

Joist depth	h = **150** mm
Joist spacing	s = **400** mm
Timber strength class	**C16**
Service class of timber	**1**



Span details

Number of spans	N_{span} = **1**
Length of bearing	L_b = **100** mm
Effective length of span	L_{s1} = **2700** mm



Section properties

Second moment of area	$I = b \times h^3 / 12$ = **14062500** mm^4
Section modulus	$Z = b \times h^2 / 6$ = **187500** mm^3

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.02** kN/m
Dead load	F_{d_udl} = **0.90** kN/m^2
Imposed UDL(Medium term)	F_{i_udl} = **0.75** kN/m^2
Imposed point load (Short term)	F_{i_pt} = **0.90** kN

Modification factors

Service class for bending parallel to grain	K_{2m} = **1.00**
Service class for compression	K_{2c} = **1.00**
Service class for shear parallel to grain	K_{2s} = **1.00**
Service class for modulus of elasticity	K_{2e} = **1.00**
Section depth factor	K_7 = **1.08**
Load sharing factor	K_8 = **1.10**

<u>Consider medium term loads</u>

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **0.622** kNm
Maximum shear force	V = **0.922** kN
Maximum support reaction	R = **0.922** kN

Maximum deflection δ = **3.999** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.865** N/mm^2

Applied bending stress σ_{m_max} = M / Z = **3.318** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.921** N/mm^2

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.184** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.025** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.184** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm})$ = **8.100** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **3.818** mm

Shear deflection δ_{shear} = **0.181** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **3.999** mm

PASS - Actual deflection within permissible limits

<u>Consider short term loads</u>

Load duration factor K_3 = **1.50**

Maximum bending moment M = **0.956** kNm

Maximum shear force V = **1.417** kN

Maximum support reaction R = **1.417** kN

Maximum deflection δ = **5.401** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **9.438** N/mm^2

Applied bending stress σ_{m_max} = M / Z = **5.100** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **1.106** N/mm^2

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.283** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.630** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.283** N/mm^2

PASS - Applied bearing stress within permissible limits

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	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date

Check deflection

Permissible deflection δ_{adm} = min(L_{s1} × 0.003, 14 mm) = **8.100** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **5.123** mm

Shear deflection δ_{shear} = **0.278** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **5.401** mm

PASS - Actual deflection within permissible limits

Above First Floor:

<u>Beam C – Chimney & Post Support</u>

UDL DL = (3.2 x 7.4 x 0.75) = 17.8 kN/m
Chim.Breast

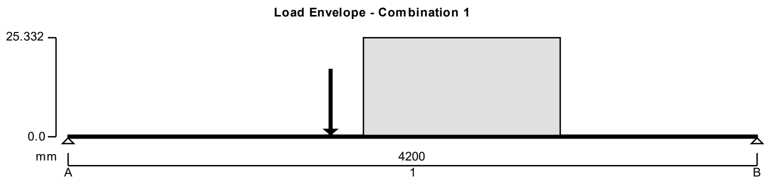
PL DL = 7.5 kN
RA_1
PL IL = 4.3 kN

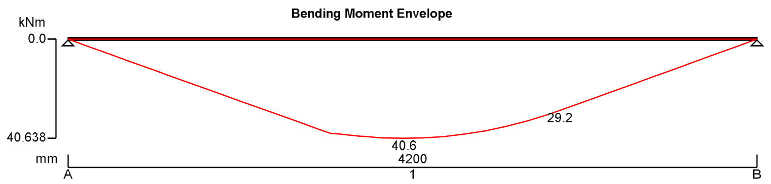
STEEL BEAM C ANALYSIS & DESIGN (BS5950)

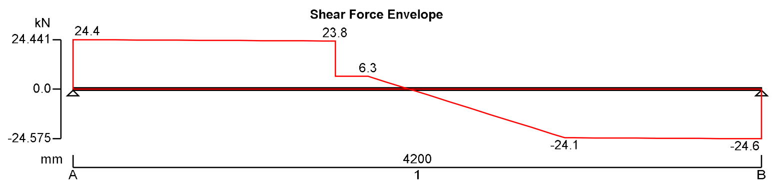
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 — Dead self weight of beam × 1
Dead partial UDL 17.8 kN/m from 1800 mm to 3000 mm
Dead point load 7.5 kN at 1600 mm
Imposed point load 4.3 kN at 1600 mm

Load combinations

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

Analysis results

Maximum moment	M_{max} = **40.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **24.4** kN	V_{min} = **-24.6** kN
Deflection	δ_{max} = **13.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **24.4** kN	R_{A_min} = **24.4** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **14.4** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.7** kN	
Maximum reaction at support B	R_{B_max} = **24.6** kN	R_{B_min} = **24.6** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **15.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.6** kN	

Section details

Section type — **UKC 152x152x30 (Tata Steel Advance)**

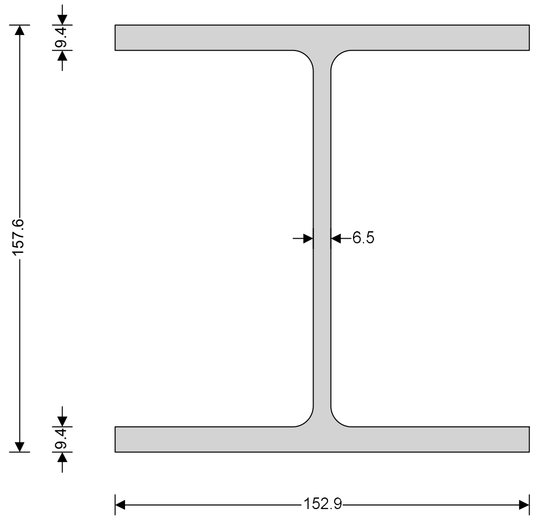
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **9.4** mm

Design strength — p_y = **275** 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 — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **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 = **123.6** mm

$d / t = 19.0 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **76.5** mm

$b / T = 8.1 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **24.6** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1024** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **169** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **40.6** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **68.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1}$ = **4200** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **109.740**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.849**

Torsional index x = **15.999**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.739**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **68.815**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.242**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **427.3** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **402.7** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **191.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **25.4** kNm

Moment at centre-line of segment M_3 = **40.6** kNm

Moment at three quarter point of segment M_4 = **25.6** kNm

Maximum moment in segment M_{abs} = **40.6** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **40.6** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.888**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **47.4** kNm

M_b / m_{LT} = **53.4** 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 $\delta_{lim} = \min(16\text{ mm}, L_{s1} / 300)$ = **14** mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **13.641** mm

PASS - Maximum deflection does not exceed deflection limit

<u>Beam D – Stair Landing Trimmer</u>

UDL DL = 2.8/2 x 0.7 = 1.0 kN/m

2nd Flr

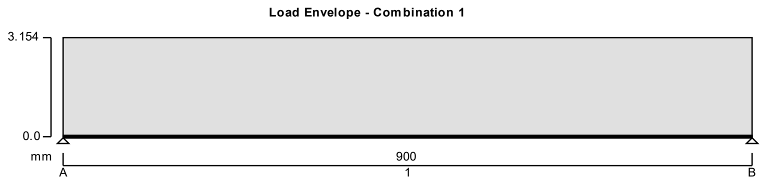
UDL IL = 2.8/2 x 1.5 = 2.1 kN/m

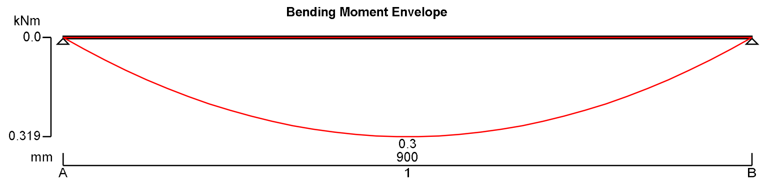
Span = 0.9m

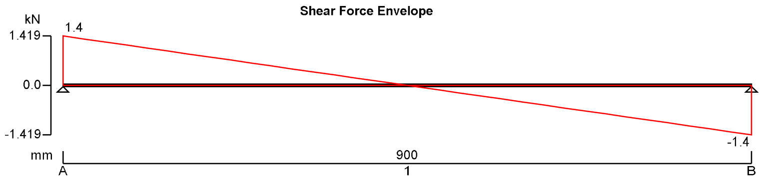
TIMBER BEAM D ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 1.000 kN/m

Imposed full UDL 2.100 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **0.319** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.319** kNm	
Maximum shear	F_{max} = **1.419** kN	F_{min} = **-1.419** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.419** kN	
Total load on beam	W_{tot} = **2.839** kN	
Reactions at support A	R_{A_max} = **1.419** kN	R_{A_min} = **1.419** kN

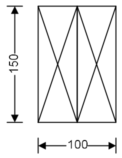
Unfactored dead load reaction at support A R_{A_Dead} = **0.474** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **0.945** kN

Reactions at support B R_{B_max} = **1.419** kN R_{B_min} = **1.419** kN

Unfactored dead load reaction at support B R_{B_Dead} = **0.474** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.945** kN



Timber section details

Breadth of sections b = **50** mm

Depth of sections h = **150** mm

Number of sections in member N = **2**

Overall breadth of member $b_b = N \times b$ = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1**

Load duration **Long term**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **15000** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **375000** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **250000** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **28125000** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **12500000** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **43.3** mm

$i_y = \sqrt{(I_y / A)}$ = **28.9** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.00**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.08**

Load sharing - cl.2.10.11 K_8 = **1.10**

Minimum modulus of elasticity - Table 20 K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19 **5.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.50**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.142** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.059**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **6.292** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **0.852** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.135**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.737** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.142** N/mm^2

$\tau_a / \tau_{adm} =$ **0.193**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 =$ **6612** N/mm^2

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) =$ **2.700** mm

Bending deflection $\delta_{b\_s1} =$ **0.145** mm

Shear deflection $\delta_{v\_s1} =$ **0.062** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.207** mm

$\delta_a / \delta_{adm} =$ **0.077**

PASS - Total deflection is less than permissible deflection

Use 2No.175 x 50 C16 due to Floor Depth Required – See Floor Design Below

<u>Beam E – Stair Side Trimmer</u>

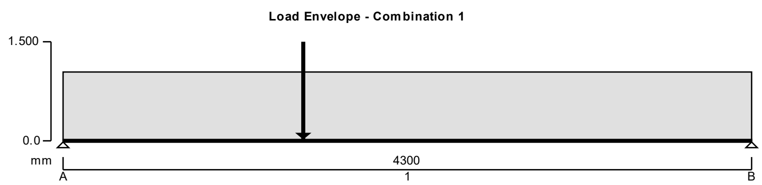
UDL DL = 0.4 x (0.7 – 0.15) = 0.3 kN/m
c/c SWJ
UDL IL = 0.4 x 1.5 = 0.6 kN/m

PL DL = 0.5 kN
RD_1
PL IL = 1.0 kN

TIMBER BEAM E ANALYSIS & DESIGN (BS5268)

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07



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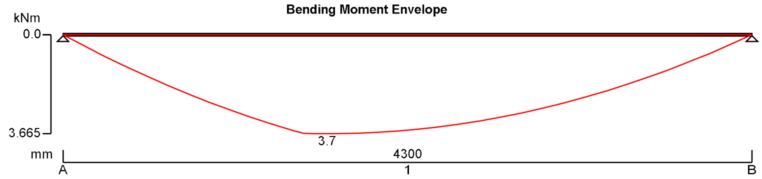
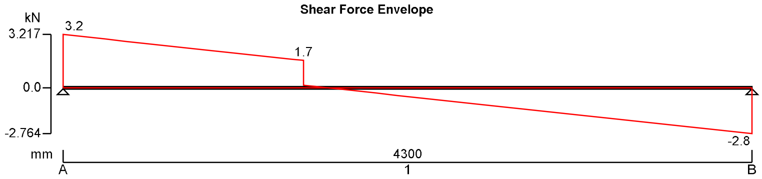
	Section Loft Conversion & Rear/Side Extension				Sheet no./rev. 20	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date





Applied loading

Beam loads

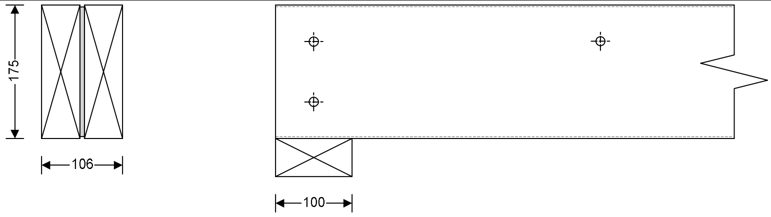
Dead self weight of beam × 1
Dead full UDL 0.300 kN/m
Imposed full UDL 0.600 kN/m
Dead point load 0.500 kN at 1500 mm
Imposed point load 1.000 kN at 1500 mm

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **3.665** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.665** kNm	
Maximum shear	F_{max} = **3.217** kN	F_{min} = **-2.764** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **3.217** kN	
Total load on beam	W_{tot} = **5.981** kN	
Reactions at support A	R_{A_max} = **3.217** kN	R_{A_min} = **3.217** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.276** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.941** kN	
Reactions at support B	R_{B_max} = **2.764** kN	R_{B_min} = **2.764** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.125** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.639** kN	



Timber section details

Breadth of timber sections — $b = \mathbf{50}$ mm

Depth of timber sections — $h = \mathbf{175}$ mm

Number of timber sections in member — $N = \mathbf{2}$

Timber strength class — **C16**

Steel section details

Breadth of steel plate — $b_s = \mathbf{6}$ mm

Depth of steel plate — $h_s = \mathbf{170}$ mm

Number of steel plates in beam — $N_s = \mathbf{1}$

Steel stress — $p_y = \mathbf{165}$ N/mm^2

Bolt diameter — $\phi_b = \mathbf{12}$ mm

Member details

Service class of timber — **1**

Load duration — **Long term**

Length of bearing — $L_b = \mathbf{100}$ mm

Section properties

Cross sectional area of beam — $A = N \times b \times h = \mathbf{17500}$ mm^2

Timber section modulus — $Z_{xt} = N \times b \times h^2 / 6 = \mathbf{510417}$ mm^3

Steel section modulus — $Z_{xs} = N_s \times b_s \times h_s^2 / 6 = \mathbf{28900}$ mm^3

Second moment of area of timber — $I_{xt} = N \times b \times h^3 / 12 = \mathbf{44661458}$ mm^4

Second moment of area of steel — $I_{xs} = N_s \times b_s \times h_s^3 / 12 = \mathbf{2456500}$ mm^4

Load proportions

Instant deflection under permanent actions — $u_{instG} = \mathbf{3.006}$ mm

Instant deflection under principal variable action — $u_{instQ1} = \mathbf{4.604}$ mm

$k_{def} = \mathbf{0.6}$

$\psi_2 = \mathbf{0.3}$

Final minimum modulus of elasticity

$$E_{min,fin} = E_{min} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) = \mathbf{4309}\ \text{N/mm}^2$$

Proportion of applied load in timber — $k_t = E_{mean} \times I_{xt} / (E_{mean} \times I_{xt} + E_{S5950} \times I_{xs}) = \mathbf{0.438}$

Proportion of applied load in steel — $k_s = 1.1 \times E_{S5950} \times I_{xs} / (E_{min,fin} \times I_{xt} + E_{S5950} \times I_{xs}) = \mathbf{0.796}$

Modification factors

Duration of loading - Table 17 — $K_3 = \mathbf{1.00}$

Bearing stress - Table 18 — $K_4 = \mathbf{1.00}$

Total depth of member - cl.2.10.6 — $K_7 = (300\text{ mm} / h)^{0.11} = \mathbf{1.06}$

Load sharing - cl.2.10.11 — $K_8 = \mathbf{1.10}$

Minimum modulus of elasticity - Table 20 — $K_9 = \mathbf{1.14}$

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19 — **2.00**

Actual depth-to-breadth ratio $h / (N \times b + N_s \times b_s) = \mathbf{1.65}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.420}$ N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.322}$ N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.133}$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{6.186}$ N/mm^2

Applied timber bending stress $\sigma_{m\_a} = k_t \times M / Z_{xt} = \mathbf{3.147}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.509}$

PASS - Timber bending stress is less than permissible timber bending stress

Applied steel bending stress $\sigma_{m\_a\_s} = k_s \times M / Z_{xs} = \mathbf{100.919}$ N/mm^2

$\sigma_{m\_a\_s} / p_y = \mathbf{0.612}$

PASS - Steel bending stress is less than permissible steel bending stress

Check beam in shear

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = \mathbf{0.737}$ N/mm^2

Applied shear stress $\tau_a = 3 \times k_t \times F / (2 \times A) = \mathbf{0.121}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.164}$

PASS - Shear stress within permissible limits

Deflection

Modulus of elasticity for deflection $E = E_{mean} = \mathbf{8800}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) = \mathbf{12.900}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{7.610}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.457}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{8.067}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.625}$

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Total load on beam $W_{tot} = \mathbf{5.981}$ kN

Total load taken by steel $W_s = k_s \times W_{tot} = \mathbf{4.760}$ kN

Basic bolt shear load - Table 70 $v_{90} = \mathbf{1.659}$ kN

Number of interfaces $N_{int} = (N + N_s) – 1 = \mathbf{2}$

Number of bolts required at supports $N_{be} = \max(k_s \times R_{A\_max} / (N_{int} \times v_{90}), 2) = \mathbf{2}$

Limiting bolt spacing $S_{limit} = \min(2.5 \times h, 600\text{ mm}) = \mathbf{438}$ mm

Maximum bolt spacing $S_{max} = \mathbf{375}$ mm

Minimum number of bolts along length of beam $N_{bl} = W_s / (N_{int} \times v_{90}) = \mathbf{1.434}$

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at maximum 375 mm centres staggered 40 mm alternately above and below the centre line

Minimum bolt spacings

Minimum end spacing $S_{end} = 4 \times \phi_b = \mathbf{48}$ mm

Minimum edge spacing $S_{edge} = 4 \times \phi_b = \mathbf{48}$ mm

Minimum bolt spacing $S_{bolt} = 4 \times \phi_b = \mathbf{48}$ mm

Minimum washer diameter $\phi_w = 3 \times \phi_b = \mathbf{36}$ mm

Minimum washer thickness $t_w = 0.25 \times \phi_b = \mathbf{3}$ mm

<u>Beam F – Front Party-to-Flank Wall</u>

UDL DL = (4.3/2 x 0.7) + (1.3 x 0.7) + (4.2/2 x 1.35) = 5.3 kN/m
2^{nd} Flr Int.Stud P.Roof
UDL IL = (4.3/2 x 1.5) + (4.2/2 x 0.75) = 4.8 kN/m

PL1 DL = 14.4 kN
RC$_1$
PL1 IL = 2.7 kN

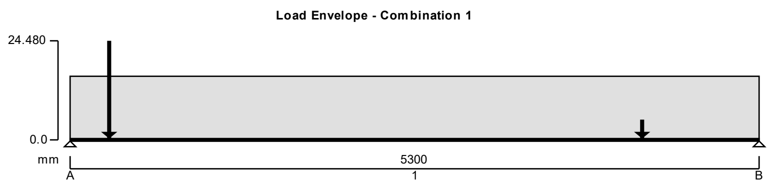
PL2 DL = 1.3 kN
RE$_1$
PL2 IL = 2.0 kN

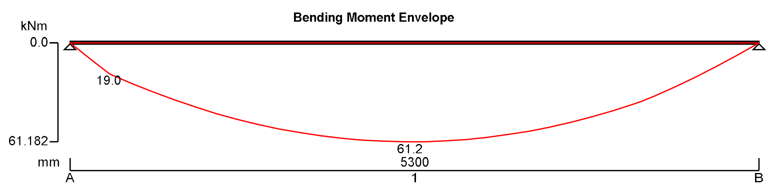
STEEL BEAM F ANALYSIS & DESIGN (BS5950)

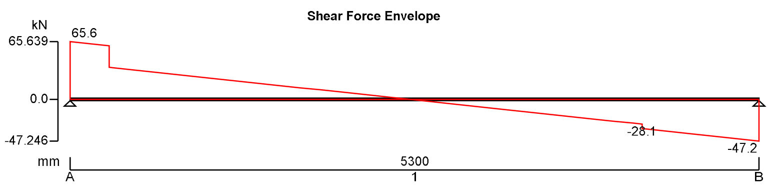
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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	Dead self weight of beam × 1
	Dead full UDL 5.3 kN/m
	Imposed full UDL 4.8 kN/m
	Dead point load 14.4 kN at 300 mm
	Imposed point load 2.7 kN at 300 mm
	Dead point load 1.3 kN at 4400 mm
	Imposed point load 2 kN at 4400 mm

Load combinations

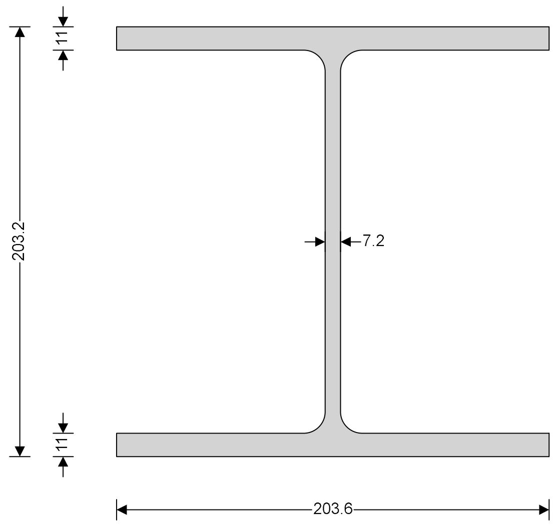
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **61.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **65.6** kN	V_{min} = **-47.2** kN
Deflection	δ_{max} = **13.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **65.6** kN	R_{A_min} = **65.6** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **29** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **15.6** kN	
Maximum reaction at support B	R_{B_max} = **47.2** kN	R_{B_min} = **47.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **17.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **14.5** kN	

Section details

Section type **UKC 203x203x46 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **65.6** kN Design shear resistance P_v = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **61.2** kNm Moment capacity low shear M_c = **136.8** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **95.9** kNm M_b / m_{LT} = **102.9** 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 δ_{lim} = **16** mm Maximum deflection δ = **13.072** mm

PASS - Maximum deflection does not exceed deflection limit

<u>Beam G – Rear Dormer Support</u>

UDL DL = (4.3/2 x 0.7) + (2.5 x 1.0 x 0.85) + (2.8/2 x 1.05) = 5.1 kN/m

2^{nd} Flr 100 Stud Ext. F.Roof

UDL IL = (4.3/2 x 1.5) + (2.8/2 x 0.75) = 4.3 kN/m

PL1 DL = 15.7 kN

RC_2

PL1 IL = 1.6 kN

PL2 DL = 1.2 kN

RE_2

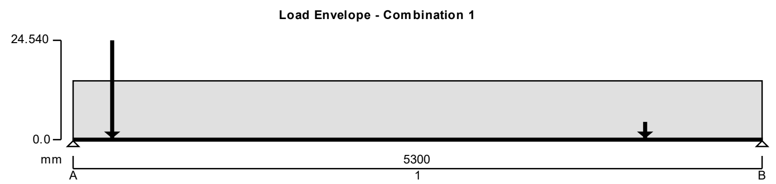
PL2 IL = 1.7 kN

STEEL BEAM G ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05



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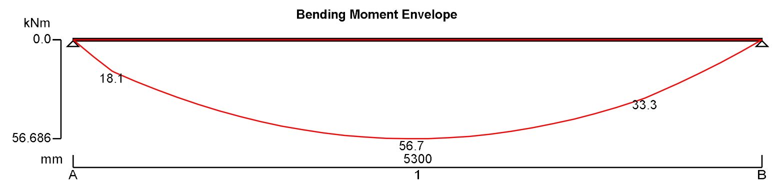
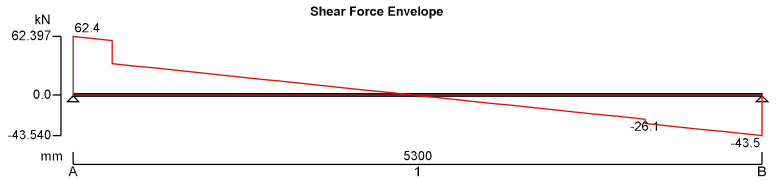
	Section Loft Conversion & Rear/Side Extension				Sheet no./rev. 26	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date





Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 5.1 kN/m
	Imposed full UDL 4.3 kN/m
	Dead point load 15.7 kN at 300 mm
	Imposed point load 1.6 kN at 300 mm
	Dead point load 1.2 kN at 4400 mm
	Imposed point load 1.7 kN at 4400 mm

Load combinations

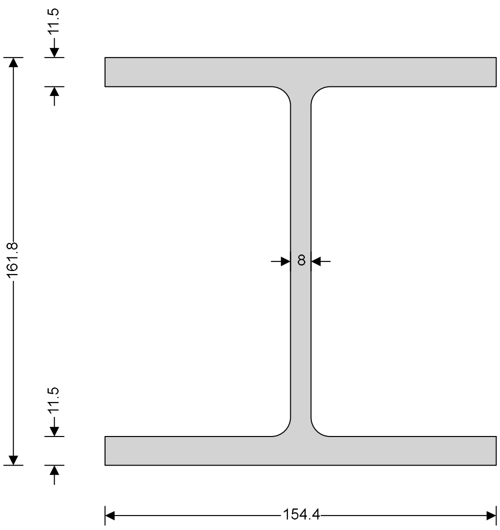
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **56.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **62.4** kN	V_{min} = **-43.5** kN
Deflection	δ_{max} = **10.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **62.4** kN	R_{A_min} = **62.4** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **29.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **13.2** kN	
Maximum reaction at support B	R_{B_max} = **43.5** kN	R_{B_min} = **43.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **16.4** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **12.9** kN	

Section details

Section type **UKC 152x152x37 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **62.4** kN Design shear resistance P_v = **213.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **56.7** kNm Moment capacity low shear M_c = **84.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **55.7** kNm M_b / m_{LT} = **59.7** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **14.722** mm Maximum deflection δ = **10.505** mm

PASS - Maximum deflection does not exceed deflection limit

<u>First Floor Joists</u>

UDL DL = 0.7 – 0.15 = 0.55 kN/m^2

SWJ

UDL IL = 1.5 kN/m^2

Span = 4.3m

SECOND FLOOR TIMBER JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

Joist details

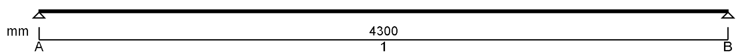
Joist breadth $b = \mathbf{75}$ mm

Joist depth $h = \mathbf{175}$ mm

Joist spacing $s = \mathbf{300}$ mm

Timber strength class **C16**

Service class of timber **1**

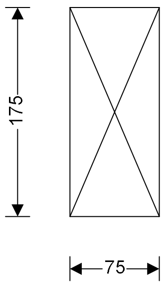


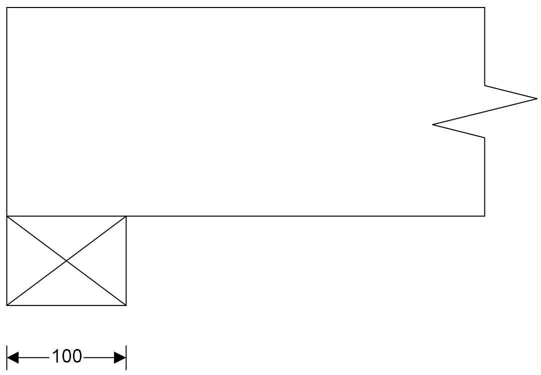
Span details

Number of spans $N_{span} = \mathbf{1}$

Length of bearing $L_b = \mathbf{100}$ mm

Effective length of span $L_{s1} = \mathbf{4300}$ mm





Section properties

Second moment of area $I = b \times h^3 / 12 = \mathbf{33496094}$ mm^4

Section modulus $Z = b \times h^2 / 6 = \mathbf{382813}$ mm^3

Loading details

Joist self weight $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.04}$ kN/m

Dead load $F_{d\_udl} = \mathbf{0.55}$ kN/m^2

Imposed UDL(Long term) $F_{i\_udl} = \mathbf{1.50}$ kN/m^2

Imposed point load (Medium term) $F_{i\_pt} = \mathbf{1.40}$ kN

Modification factors

Service class for bending parallel to grain $K_{2m} = \mathbf{1.00}$

Service class for compression $K_{2c} = \mathbf{1.00}$

Service class for shear parallel to grain $K_{2s} = \mathbf{1.00}$

Service class for modulus of elasticity $K_{2e} = \mathbf{1.00}$

Section depth factor K_7 = **1.06**

Load sharing factor K_8 = **1.10**

<u>Consider long term loads</u>

Load duration factor K_3 = **1.00**

Maximum bending moment M = **1.514** kNm

Maximum shear force V = **1.408** kN

Maximum support reaction R = **1.408** kN

Maximum deflection δ = **10.142** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **6.186** N/mm^2

Applied bending stress σ_{m_max} = M / Z = **3.954** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.737** N/mm^2

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.161** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.420** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.188** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = min($L_{s1} \times 0.003$, 14 mm) = **12.900** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **9.890** mm

Shear deflection δ_{shear} = **0.252** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **10.142** mm

PASS - Actual deflection within permissible limits

<u>Consider medium term loads</u>

Load duration factor K_3 = **1.25**

Maximum bending moment M = **1.979** kNm

Maximum shear force V = **1.841** kN

Maximum support reaction R = **1.841** kN

Maximum deflection δ = **11.290** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.733** N/mm^2

Applied bending stress σ_{m_max} = M / Z = **5.169** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.921** N/mm^2

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.210** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.025** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.245** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **12.900** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **10.962** mm

Shear deflection δ_{shear} = **0.329** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **11.290** mm

PASS - Actual deflection within permissible limits

Above Ground Floor:

<u>Beam H – Spine Wall</u>

UDL DL = (7.0/2 x 0.7) + (2.7 x 2.5) = 9.2 kN/m
1st Flr 112 Brk
UDL IL = (7.0/2 x 1.5) = 5.3 kN/m

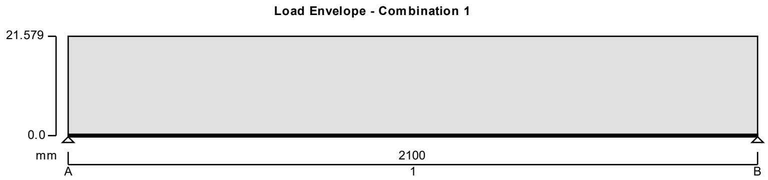
Span = 2.1m

STEEL BEAM H ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05



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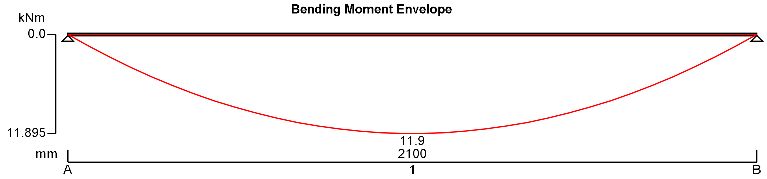
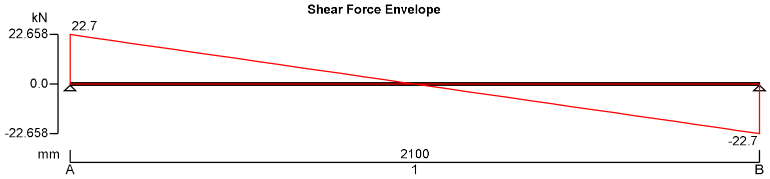
	Section Loft Conversion & Rear/Side Extension			Sheet no./rev. 31	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date





Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 9.2 kN/m
	Imposed full UDL 5.3 kN/m

Load combinations

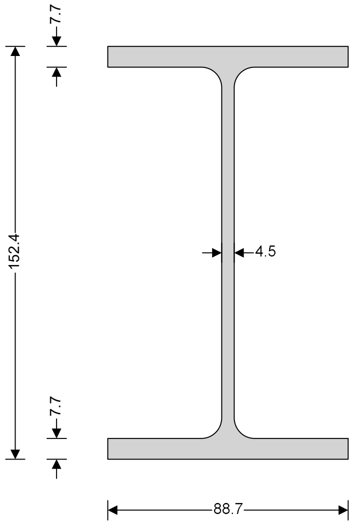
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **11.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **22.7** kN	V_{min} = **-22.7** kN
Deflection	δ_{max} = **2.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **22.7** kN	R_{A_min} = **22.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **9.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5.6** kN	
Maximum reaction at support B	R_{B_max} = **22.7** kN	R_{B_min} = **22.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **9.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.6** kN	

Section details

Section type **UKB 152x89x16 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00**

Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **22.7** kN

Design shear resistance P_v = **113.2** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **11.9** kNm

Moment capacity low shear M_c = **33.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **22.6** kNm

M_b / m_{LT} = **24.4** 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 δ_{lim} = **7** mm

Maximum deflection δ = **2.17** mm

PASS - Maximum deflection does not exceed deflection limit

<u>Lintel I – Coriddor Lintel</u>

UDL SLS = (1.8/2 x 2.2) + (2.8 x 2.5 x 0.85) = 8.0 kN/m
Ass.1st Flr 112 Brk w/Void

Span = 1.0m

Use 100 x 100 S4 Naylor Concrete Lintel (SWL = 12.8 kN/m)

Lintel J – Entrance Hall to Side Extension

UDL SLS = (1.8/2 x 2.2) + (4.6 x 4.8) + (2.2/2 x 1.8) + ((17.1 + 14.5) / (2.8 + 2.0)) + ((0.5 + 1.0) / (2.8 + 2.8)) = 32.9 kN/m
1st Flr 225 Brk F.Roof RF_2 Spread RD_2 Spread

Span = 0.9m

Use 2No. 100 x 140 R6 Naylor Concrete Lintels (SWL = 2 x 29.06 = 58.12 kN/m)

Lintel K – Front Elevation of Side Extension

UDL SLS = (0.8 x 3.5) = 2.8 kN/m
Cav.Wall

Span = 1.2m

Total Load = 2.8 x 1.2 = 3.4 kN → Use Use Catnic CG90/100 Lintel (SWL = 15 kN)

Beam L – Orginal Rear Elevation Opening

UDL DL = (2.9 x 4.8 x 0.85) + (3.7/2 x 1.05) + (3.4/2 x 0.7) = 15.0 kN/m
225 Brk w/Win F.Roof 1st Flr
UDL IL = (3.7/2 x 0.75) + (3.4/2 x 1.5) = 4.0 kN/m

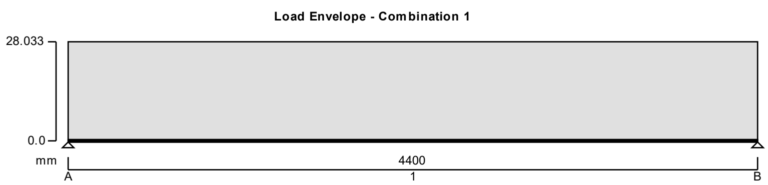
Span = 4.4m

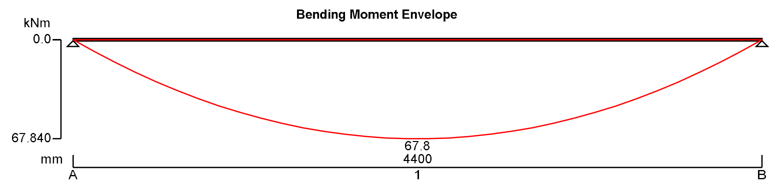
STEEL BEAM L ANALYSIS & DESIGN (BS5950)

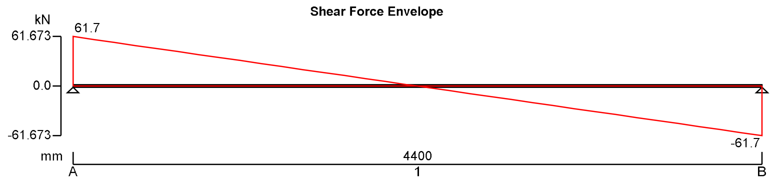
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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	Dead self weight of beam × 1
	Dead full UDL 15 kN/m
	Imposed full UDL 4 kN/m

Load combinations

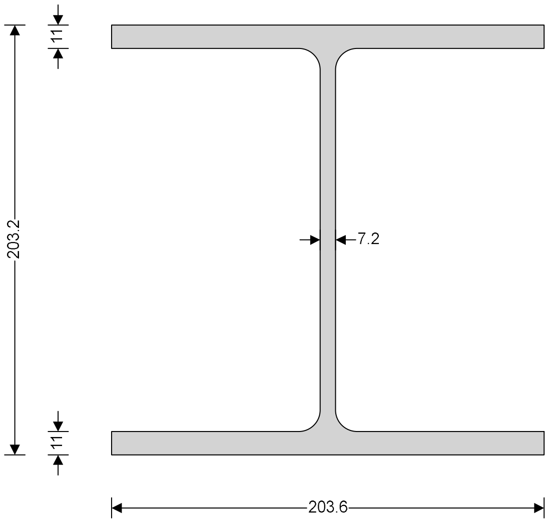
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **67.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **61.7** kN	V_{min} = **-61.7** kN
Deflection	δ_{max} = **10.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **61.7** kN	R_{A_min} = **61.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **34** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **8.8** kN	
Maximum reaction at support B	R_{B_max} = **61.7** kN	R_{B_min} = **61.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **34** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **8.8** kN	

Section details

Section type **UKC 203x203x46 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00**   Section classification **Compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **61.7** kN   Design shear resistance P_v = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **67.8** kNm   Moment capacity low shear M_c = **136.8** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **106.4** kNm M_b / m_{LT} = **115** 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 δ_{lim} = **14.667** mm   Maximum deflection δ = **10.138** mm

PASS - Maximum deflection does not exceed deflection limit

Beam M – New Rear Elevation Folding Doors

UDL DL = (0.7 x 3.5) + (3.7/2 x 1.05) = 4.4 kN/m
Cav.Wall F.Roof
UDL IL = (3.7/2 x 0.75) = 1.4 kN/m

Span = 4.0m

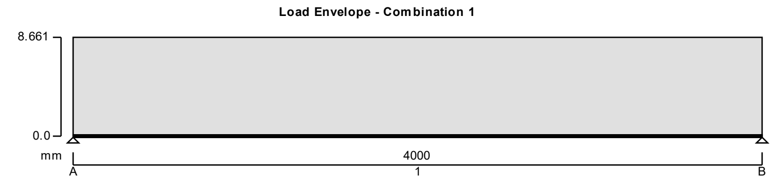
STEEL BEAM M ANALYSIS & DESIGN (BS5950)

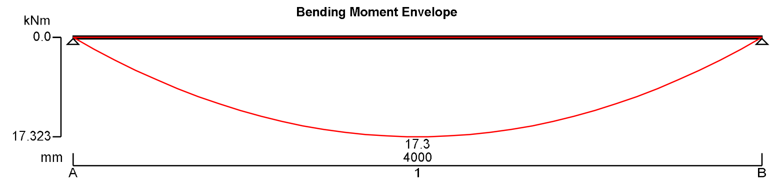
STEEL BEAM ANALYSIS & DESIGN (BS5950)

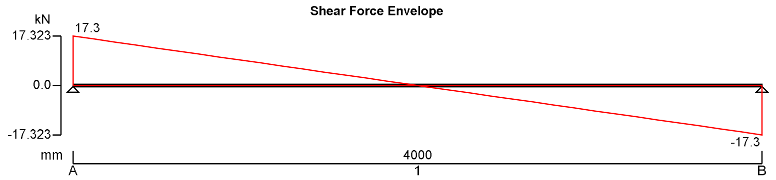
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05

Tedds Make Structures Ltd. 20 Gallery Close Northampton, Northants NN3 5NT	Project 45 Bedford Avenue, Hayes UB4 0DR				Job Ref. 210026	
	Section Loft Conversion & Rear/Side Extension				Sheet no./rev. 36	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 4.4 kN/m
	Imposed full UDL 1.4 kN/m

Load combinations

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

Analysis results

Maximum moment	M_{max} = **17.3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **17.3** kN	V_{min} = **-17.3** kN
Deflection	δ_{max} = **7.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.3** kN	R_{A_min} = **17.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **9.2** kN	

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **2.8** kN

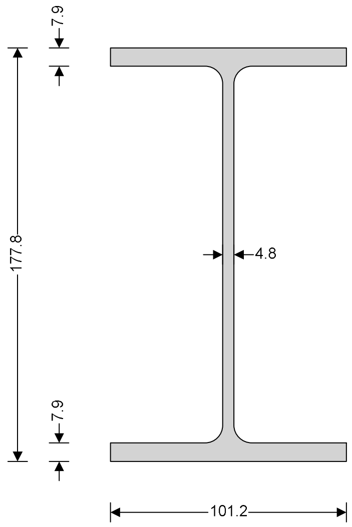
Maximum reaction at support B R_{B_max} = **17.3** kN R_{B_min} = **17.3** kN

Unfactored dead load reaction at support B R_{B_Dead} = **9.2** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.8** kN

Section details

Section type **UB 178x102x19 (BS4-1)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **17.3** kN Design shear resistance P_v = **140.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **17.3** kNm Moment capacity low shear M_c = **47.1** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **19.4** kNm M_b / m_{LT} = **21** 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 δ_{lim} = **13.333** mm Maximum deflection δ = **7.179** mm

PASS - Maximum deflection does not exceed deflection limit

8mm Steel Plate Intermittent Welded to Bottom Flange. Section to be Fully Galvanised.

Beam N – New Rear Elevation Window

UDL DL = (0.9 x 3.5) + (3.7/2 x 1.05) = 5.1 kN/m
Cav.Wall F.Roof
UDL IL = (3.7/2 x 0.75) = 1.4 kN/m

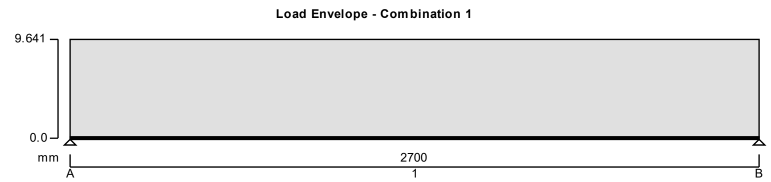
Span = 2.0m

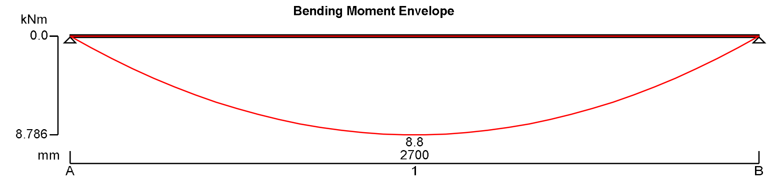
STEEL BEAM N ANALYSIS & DESIGN (BS5950)

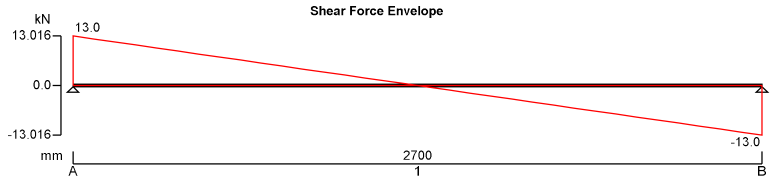
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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	Dead self weight of beam × 1
	Dead full UDL 5.1 kN/m

Imposed full UDL 1.4 kN/m

Load combinations

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

Analysis results

Maximum moment	M_{max} = **8.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **13** kN	V_{min} = **-13** kN
Deflection	δ_{max} = **1.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **13** kN	R_{A_min} = **13** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.9** kN	
Maximum reaction at support B	R_{B_max} = **13** kN	R_{B_min} = **13** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.9** kN	

Section details

Section type **UKB 178x102x19 (Tata Steel Advance)**

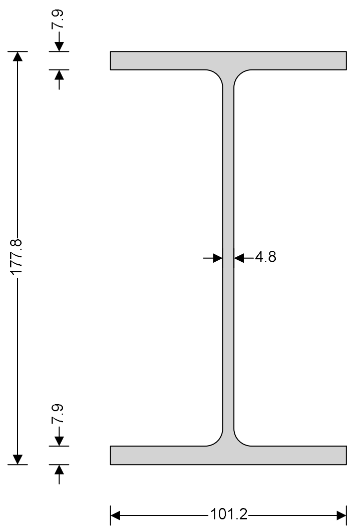
Steel grade **S275**

From table 9: Design strength p_y

Thickness of element max(T, t) = **7.9** mm

Design strength p_y = **275** 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 $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **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 = **146.8** mm

$d / t = 30.6 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **50.6** mm

$b / T = 6.4 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **13** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **853** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **140.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **8.8** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **47.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1}$ = **2700** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **113.744**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.888**

Torsional index x = **22.560**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.815**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **82.246**

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 α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.336**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **299.1** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **337.2** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **159.8** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **6.6** kNm

Moment at centre-line of segment M_3 = **8.8** kNm

Moment at three quarter point of segment M_4 = **6.6** kNm

Maximum moment in segment M_{abs} = **8.8** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **8.8** kNm

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **27.4** kNm

M_b / m_{LT} = **29.6** 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 δ_{lim} = min(16 mm, L_{s1} / 300) = **9** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **1.665** mm

PASS - Maximum deflection does not exceed deflection limit

Steel Posts to Rear Corner Piers

F_x = 17.3 kN
RM2

M_x = 17.3 x 0.025 x 2.3 = 1.0 kNm
2.5% H
Or
M_x = 17.3 x (0.1 + 0.05) = 2.6 kNm
Ecc.
Or
M_x = 4.0/2 x 0.7 x 2.3^2 / 2 = 3.7 kNm
Ass.Wind Panel L-arm

V_x = 4.0/2 x 0.7 x 2.3/2 = 1.7 kNm
Ass.Wind Panel

STEEL POST @ RM2 DESIGN (BS5950)

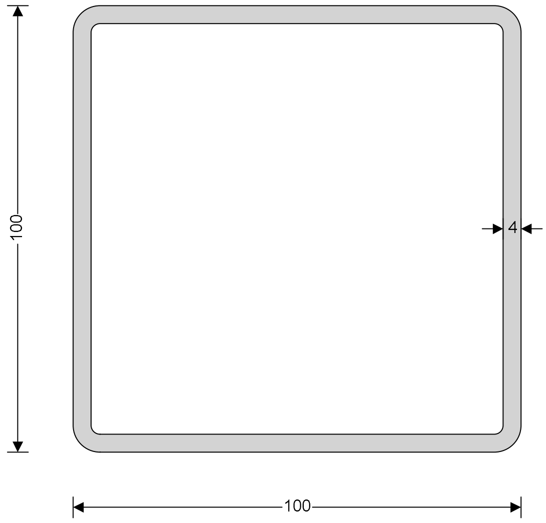
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.05

Section details

Section type **SHS 100x100x4.0 (Tata Steel Celsius)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **1.7** kN Design shear resistance $P_{y,v}$ = **125.3** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Moment capacity - Section 4.2.5

Design bending moment M = **3.7** kNm Moment capacity low shear M_c = **15** kNm

Buckling resistance moment - Section 4.3.6.4

Bending strength p_b = **275** N/mm^2 Buckling resistance moment M_b = **15** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **17.3** kN Compression resistance P_{cx} = **185.9** kN

PASS - Compression resistance exceeds design compression force

Design compression force F_c = **17.3** kN Compression resistance P_{cy} = **365.7** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check $F_c / (A \times p_y) + M / M_c$ = **0.289**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - cl.4.8.3.3.3

Buckling resistance checks $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx})$ = **0.352**

$F_c / P_{cy} + 0.5 \times m_{LT} \times M_{LT} / M_{cx}$ = **0.171**

PASS - Member buckling resistance checks are satisfied

Steel Posts to Middle Pier

F_x = 17.3 + 13.0 = 30.3 kN
RM1/RN2

M_x = 30.3 x 0.025 x 2.3 = 1.0 kNm
2.5% H
Or
M_x = (30.3 – 7.1) x (0.1 + 0.05) = 3.5 kNm
F_x RN2DL Ecc.

M_y = 7.0/2 x 0.7 x 2.3^2 / 2 = 6.5 kNm
Ass.Wind Panel L-arm

V_y = 7.0/2 x 0.7 x 2.3/2 = 2.9 kNm
Ass.Wind Panel

STEEL POST @ RM1/RN2 DESIGN (BS5950)

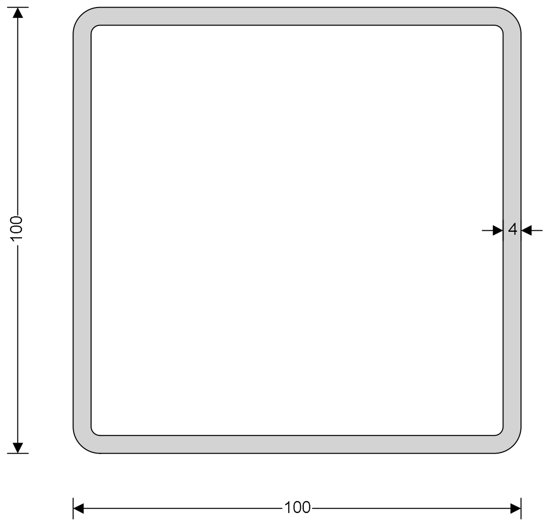
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.05

Section details

Section type **SHS 100x100x4.0 (Tata Steel Celsius)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Shear capacity - Section 4.2.3

Design shear force F_v = **2.9** kN Design shear resistance $P_{x,v}$ = **125.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M_x = **3.5** kNm — Moment capacity low shear M_{cx} = **15** kNm

Buckling resistance moment - Section 4.3.6.4

Bending strength p_b = **275** N/mm^2 — Buckling resistance moment M_b = **15** kNm

PASS - Moment capacity exceeds design bending moment

Moment capacity - Section 4.2.5

Design bending moment M_y = **6.5** kNm — Moment capacity low shear M_{cy} = **15** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **30.3** kN — Compression resistance P_{cx} = **185.9** kN

PASS - Compression resistance exceeds design compression force

Design compression force F_c = **30.3** kN — Compression resistance P_{cy} = **185.9** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check $F_c / (A \times p_y) + M_x / M_{cx} + M_y / M_{cy} = \mathbf{0.740}$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - cl.4.8.3.3.3

Buckling resistance checks $F_c / P_{cx} + m_x \times M_x / M_{cx} \times (1 + 0.5 \times F_c / P_{cx}) + 0.5 \times m_{yx} \times M_y / M_{cy} = \mathbf{0.633}$

$F_c / P_{cy} + 0.5 \times m_{LT} \times M_{LT} / M_{cx} + m_y \times M_y / M_{cy} \times (1 + 0.5 \times F_c / P_{cy}) = \mathbf{0.749}$

Interactive buckling

$m_x \times M_x \times (1 + 0.5 \times (F_c / P_{cx})) / (M_{cx} \times (1 - F_c / P_{cx})) + m_y \times M_y \times (1 + 0.5 \times (F_c / P_{cy})) / (M_{cy} \times (1 - F_c / P_{cy})) = \mathbf{0.863}$

PASS - Member buckling resistance checks are satisfied

Base of Post Connection

Worst Case @ RM_1/RN_2:

M_x = 3.5 kNm, M_y = 6.5 kNm

Designing for M_y in both directions, using 250 x 250 x 10mm Steel Base Plate and 4No.M12 Bolts/Resin Anchors with 30mm Edge & End Distances…

Tension per Pair = M / s = 6.5 / 0.18 = 36.2 kN → Tension Per Bolt = 36.2 / 2 = 18.1 kN < 37.8 kN P_{Nom} Therefore OK

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	Section Loft Conversion & Rear/Side Extension			Sheet no./rev. 45	
	Calc. by KL	Date 31/10/2021	Chk'd by	Date	App'd by	Date

Lower Flat Roof Joists to Extensions

UDL DL = 1.05 – 0.15 = 0.9 kN/m^2
SWJ
UDL IL = 0.75 kN/m^2

Span = 3.7m

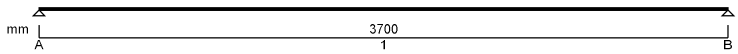
FLAT ROOF TIMBER JOIST EXTENSION DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

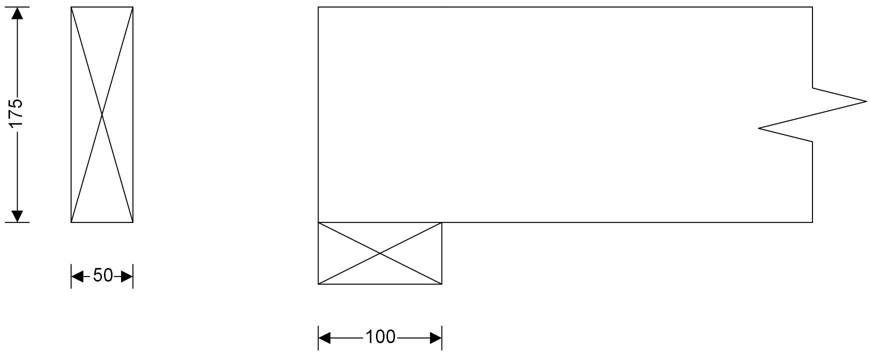
Joist details

Joist breadth	b = **50** mm	Joist depth	h = **175** mm
Joist spacing	s = **400** mm	Service class of timber	**1**
Timber strength class	**C16**		



Span details

Number of spans	N_{span} = **1**	Length of bearing	L_b = **100** mm
Clear length of span	L_{s1} = **3700** mm		



Section properties

Second moment of area	I = **22330729** mm^4	Section modulus	Z = **255208** mm^3

Loading details

Joist self weight	F_{swt} = **0.03** kN/m	Dead load	F_{d_udl} = **0.90** kN/m^2
Imposed UDL(Medium term)	F_{i_udl} = **0.75** kN/m^2		
Imposed point load (Short)	F_{i_pt} = **0.90** kN		

Consider medium term loads

Design bending moment M = **1.175** kNm Design shear force V = **1.270** kN
Design support reaction R = **1.270** kN Design deflection δ = **8.819** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **7.733** N/mm^2 Applied bending stress σ_{m_max} = **4.604** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.921** N/mm^2 Applied shear stress τ_{max} = **0.218** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.025** N/mm^2 Applied bearing stress σ_{c_max} = **0.254** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **11.100** mm Actual deflection δ = **8.819** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Design bending moment M = **1.494** kNm Design shear force V = **1.615** kN
Design support reaction R = **1.615** kN Design deflection δ = **10.006** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **9.279** N/mm^2 Applied bending stress σ_{m_max} = **5.854** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.106** N/mm^2 Applied shear stress τ_{max} = **0.277** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.630** N/mm^2 Applied bearing stress σ_{c_max} = **0.323** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **11.100** mm Actual deflection δ = **10.006** mm

PASS - Actual deflection within permissible limits

Assuming 1.5m x 1.5m Skylights…

Skylight Trimmer T1

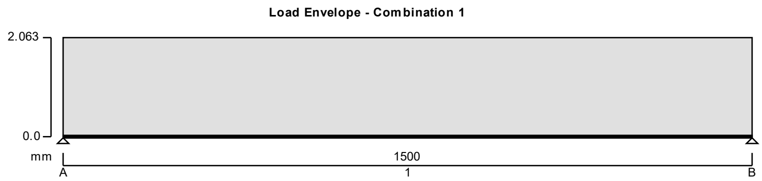
UDL DL = (1.1/2 x 1.05) + (1.5/2 x 1.2) = 1.5 kN/m
F.Roof Skylight
UDL IL = (1.1/2 x 0.75) = 0.5 kN/m

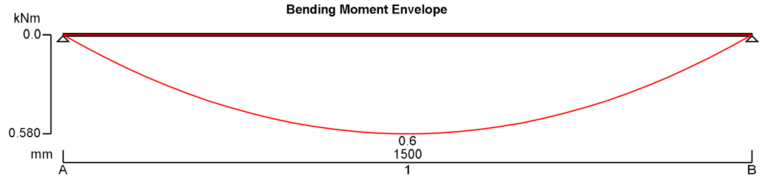
Span = 1.5m

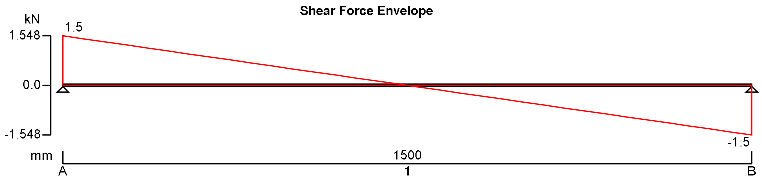
TIMBER BEAM T1 ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 1.500 kN/m

Imposed full UDL 0.500 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **0.580** kNm	M_{min} = **0.000** kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **0.580** kNm	
Maximum shear	F_{max} = **1.548** kN	F_{min} = **-1.548** kN
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **1.548** kN	
Total load on beam	W_{tot} = **3.095** kN	
Reactions at support A	R_{A_max} = **1.548** kN	R_{A_min} = **1.548** kN

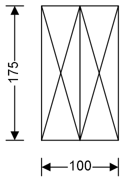
Unfactored dead load reaction at support A R_{A_Dead} = **1.173** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **0.375** kN

Reactions at support B R_{B_max} = **1.548** kN R_{B_min} = **1.548** kN

Unfactored dead load reaction at support B R_{B_Dead} = **1.173** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.375** kN



Timber section details

Breadth of sections b = **50** mm

Depth of sections h = **175** mm

Number of sections in member N = **2**

Overall breadth of member $b_b = N \times b$ = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1**

Load duration **Long term**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **17500** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **510417** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **291667** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **44661458** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **14583333** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **50.5** mm

$i_y = \sqrt{(I_y / A)}$ = **28.9** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.00**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.06**

Load sharing - cl.2.10.11 K_8 = **1.10**

Minimum modulus of elasticity - Table 20 K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19 **5.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.75**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.155** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.064**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{6.186}$ N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x = \mathbf{1.137}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.184}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.737}$ N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) = \mathbf{0.133}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.180}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 = \mathbf{6612}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) = \mathbf{4.500}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{0.461}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.096}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{0.557}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.124}$

PASS - Total deflection is less than permissible deflection

Trimmer T2 – Skylight Trimmer Side

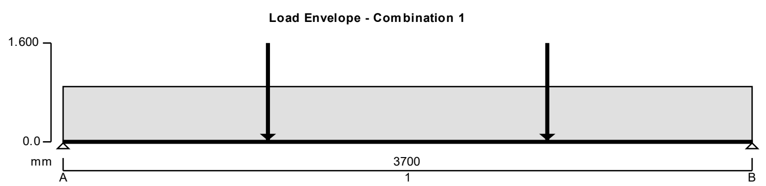
UDL DL = 0.4 x 1.05 = 0.5 kN/m
Nom.F.Roof
UDL IL = 0.4 x 0.75 = 0.3 kN/m

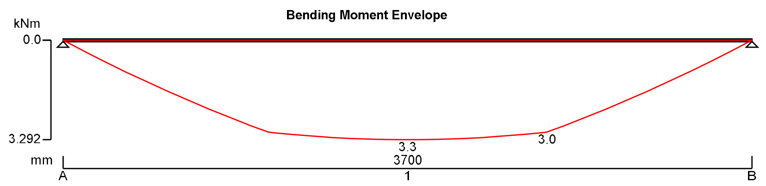
PL DL = 1.2 kN
$RT1_{1/2}$
PL IL = 0.4 kN

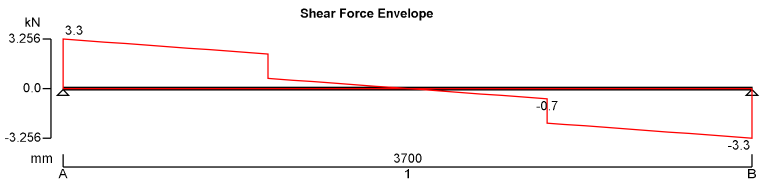
TIMBER BEAM T2 ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

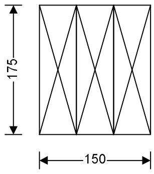
Dead self weight of beam × 1
Dead full UDL 0.500 kN/m
Imposed full UDL 0.300 kN/m
Dead point load 1.200 kN at 1100 mm
Imposed point load 0.400 kN at 1100 mm
Dead point load 1.200 kN at 2600 mm
Imposed point load 0.400 kN at 2600 mm

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment M = **3.292** kNm   Design shear F = **3.256** kN
Total load on beam W_{tot} = **6.512** kN
Reactions at support A R_{A_max} = **3.256** kN R_{A_min} = **3.256** kN
Unfactored dead load reaction at support A R_{A_Dead} = **2.301** kN
Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **0.955** kN
Reactions at support B R_{B_max} = **3.256** kN R_{B_min} = **3.256** kN
Unfactored dead load reaction at support B R_{B_Dead} = **2.301** kN
Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.955** kN



Timber section details

Breadth of section	b = **50** mm	Depth of section	h = **175** mm
Number of sections	N = **3**	Breadth of beam	b_b = **150** mm
Timber strength class	**C16**		

Member details

Service class of timber	**1**	Load duration	**Long term**
Length of bearing	L_b = **100** mm		

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio	**2.00**	Actual depth-to-breadth ratio	**1.17**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress	σ_{c_adm} = **2.420** N/mm^2	Applied bearing stress	σ_{c_a} = **0.217** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	σ_{m_adm} = **6.186** N/mm^2	Applied bending stress	σ_{m_a} = **4.300** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	τ_{adm} = **0.737** N/mm^2	Applied shear stress	τ_a = **0.186** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection	δ_{adm} = **11.100** mm	Total deflection	δ_a = **10.641** mm

PASS - Total deflection is less than permissible deflection

Padstones

Perpendicular Bearing on 3.6N Blockwork, Factor = 1.5 F_k / y_m = 1.5 x 3.5 / 3.5 = 1.5 N/mm^2
Parallel Bearing on 3.6N Blockwork, Factor = 1.25 F_k / y_m = 1.25 x 3.5 / 3.5 = 1.25 N/mm^2
Basic Brickwork Bearing Factor = 0.42 N/mm^2
Local Design Good Brickwork Factor = 1.5 x 0.42 = 0.63 N/mm^2 – *To be Checked On Site*

@ RA_2 – (7.5 + 4.3) x 10^3 / (0.42 x 100) = 281.0 → Use 300 x 100 x 215 Deep Padstone

@ RF_1 – (29 + 15.6) x 10^3 / (0.63 x 100) = 707.9 → Use 750 Long Concrete Encased Channel

@ RF_2 – (17.1 + 14.5) x 10^3 / (0.63 x 100) = 501.6 → Use 660 x 100 x 215 Deep Padstone

@ RG_1 – (29.5 + 13.2) x 10^3 / (0.63 x 100) = 677.8 → Use 700 Long Concrete Encased Channel

@ RG_2 – (16.4 + 12.9) x 10^3 / (0.63 x 100) = 465.1 → Use 660 x 100 x 215 Deep Padstone

@ $RH_{1/2}$ – (9.8 + 5.6) x 10^3 / (0.42 x 100) = 366.7 → Use 440 x 100 x 215 Deep Padstone

@ $RL_{1/2}$ – (34 + 8.8) x 10^3 / (0.42 x 215) = 474.0 → Use 660 x 215 x 215 Deep Padstone

Foundations

From British Geological Survey, Expected Soil Conditions to be Langley Silt Member – Clay & Silt
Use conservative Ground Bearing Pressure of 75 kN/m^2

Worst Case – Rear Elevation under middle Pier:

UDL SLS = ((17.3 + 9.2 + 7.1 + 1.9) / (1.0 + 1.0) + (3.5 x 3.2) + (3.7/2 x 1.8) = 32.3 kN/m
RM_1 RN_2 Spread Cav.Wall F.Roof

UDL / GBP = 32.3 / 75 = 0.43m → Use 500mm Wide Mass Concrete Strip Foundation

Assuming No Siginifcant Trees within close proximity of New Construction, Use 1000mm Depth – To Be Confirmed by Building Control Officer On Site.