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REFURBISHMENT TO

Camden Lock Market
London NW1

CALCULATIONS

Job No: 1350 **Date: Sep 2020**

General

These calculations cover the design of the structural elements for the proposed roof replacement.

The design is based on:

BS 6399-Loading
BS 8110-Concrete
BS 5950-Steel
BS 5628-Masonry
BS 5268-Timber

General loading

Roof:

- Dead load: $1.0\,\frac{\mathrm{kN}}{\mathrm{m}^2}$
- Imposed load: $0.75\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

Ground, floors:

- Dead load: $0.85\,\frac{\mathrm{kN}}{\mathrm{m}^2}$
- Imposed load: $1.5\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

Walls:

220 mm Brick wall

- $4.4\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

120 mm Brick wall

- 2.4

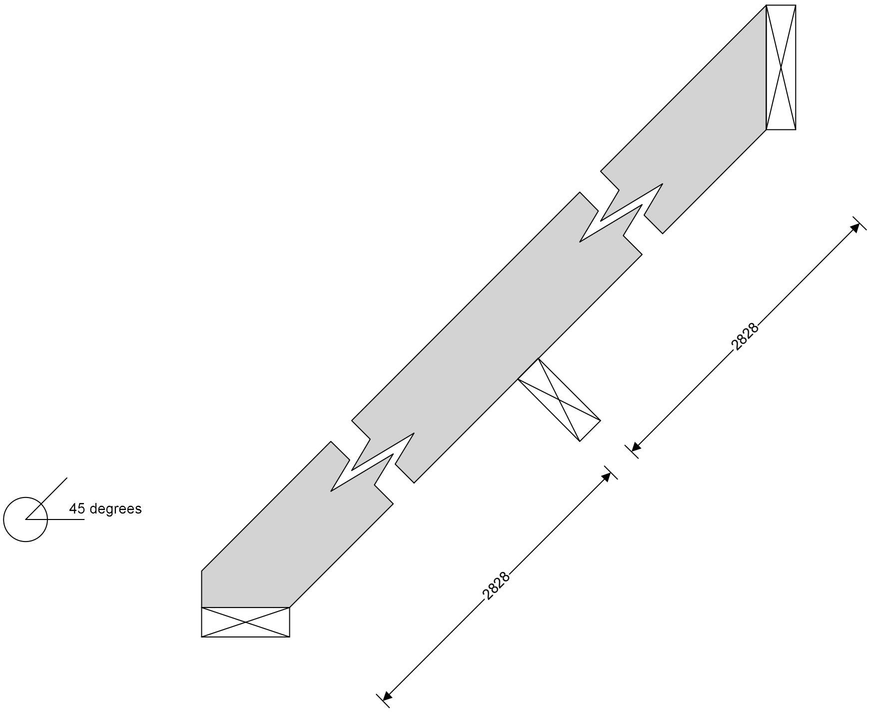
120 mm Stud wall

- 0.6

ROOF:

R - TIMBER RAFTER DESIGN (BS5268-2:2002) - 150x50 @400 c/c

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	b = **50** mm
Depth of timber sections;	h = **150** mm
Rafter spacing;	s = **400** mm
Rafter slope;	α = **45.0** deg
Clear span of rafter on horizontal;	L_{clh} = **2000** mm
Clear span of rafter on slope;	$L_{cl} = L_{clh} / \cos(\alpha)$ = **2828** mm
Rafter span;	**Continuous**
Timber strength class;	**C16**

Section properties

Cross sectional area of rafter;	$A = b \times h$ = **7500** mm^2
Section modulus;	$Z = b \times h^2 / 6$ = **187500** mm^3
Second moment of area;	$I = b \times h^3 / 12$ = **14062500** 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 = **1.00** 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; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.299** kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **4** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **2832** mm

Check bending stress at purlin

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **1.598** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.62**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.623** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.155** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.383** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.989**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.290**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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; $\sigma_{m\_max} = 9 \times F \times L_{eff}^2 / (128 \times Z)$ = **0.899** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.62**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.623** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 13 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.226** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.383** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.984**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.194**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

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

Applied shear stress; $\tau_{max} = 15 \times F \times L_{eff} / (16 \times A)$ = **0.106** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **8.496** mm

Bending deflection; $\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I)$ = **0.840** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.087** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **0.927** 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; $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha)$ = **0.449** kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **5** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **2834** mm

Check bending stress at purlin

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **2.403** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.58**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.406** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 8 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.233** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.367** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.985**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.353**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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

Applied bending stress; $\sigma_{m\_max} = 9 \times F \times L_{eff}^2 / (128 \times Z)$ = **1.352** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.58**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.406** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3 \times F \times L_{eff} \times (\cot(\alpha) + 13 \times \tan(\alpha) / 3) / (8 \times A)$ = **0.339** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.367** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.978**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.239**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

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

Applied shear stress; $\tau_{max} = 15 \times F \times L_{eff} / (16 \times A)$ = **0.159** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **8.501** mm

Bending deflection; $\delta_b = F \times L_{eff}^4 / (185 \times E_{mean} \times I)$ = **1.264** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.131** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **1.396** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor; K_3 = **1.50**

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.299** kN/m

Notional bearing length; $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **6** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **2835** mm

Check bending stress at purlin

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z)+3 \times F_p \times \cos(\alpha) \times L_{eff}/(32 \times Z)$ = **2.503** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain at purlin

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.54**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **6.047** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3 \times F \times L_{eff} \times (\cot(\alpha)+8 \times \tan(\alpha)/3)/(8 \times A)+F_p \times \sin(\alpha)/A$ = **0.240** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain at purlin

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.358** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.985**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.309**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check bending stress in lower portion of rafter

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

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

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(16 \times Z)+13 \times F_p \times \cos(\alpha) \times L_{eff}/(64 \times Z)$ = **2.755** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain in lower portion of rafter

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.54**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **6.047** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = 3\times F\times L_{eff}\times(\cot(\alpha)+4\times\tan(\alpha))/(8\times A)+F_p\times\sin(\alpha)/A$ = **0.297** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain in lower portion of rafter

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **13.358** N/mm^2

Euler coefficient; $K_{eu} = 1 – (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.982**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.346**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

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

Applied shear stress; $\tau_{max} = 15 \times F \times L_{eff} / (16 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.233** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **8.504** mm

Bending deflection; $\delta_b = L_{eff}^3 \times (F\times L_{eff} /185 + 0.015\times F_p \times\cos(\alpha)) / (E_{mean} \times I)$ = **2.600** mm

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F\times L_{eff} +2\times F_p \times\cos(\alpha))/(5 \times E_{mean} \times A)$ = **0.219** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **2.818** mm

PASS - Total deflection within permissible limits

PURLINS PR1 150x150 Timber Joists by inspection

D1 - TIMBER MEMBER DESIGN TO BS5268-2:2002 - 150x75 Timber Joists

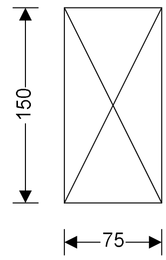
dTEDDS calculation version 1.5.07

Load derivation:

P = 4.05 x 0.5 x (1.0 + 0.75) x 1.25 x 1.6 x cos45 = 5.01 kN

Analysis results

Design axial compression; $P = \mathbf{5.010}$ kN



Timber section details

Breadth of sections;	$b = \mathbf{75}$ mm
Depth of sections;	$h = \mathbf{150}$ mm
Number of sections in member;	$N = \mathbf{1}$
Overall breadth of member;	$b_b = N \times b = \mathbf{75}$ mm
Timber strength class;	**C16**

Member details

Service class of timber;	**1**
Load duration;	**Long term**

Effective length - cl.2.11.3

Unbraced length in x-axis;	$L_x = \mathbf{2700}$ mm
Effective length factor in x-axis - Table 21;	$K_x = \mathbf{1}$
Effective length in x-axis;	$L_{ex} = L_x \times K_x = \mathbf{2700}$ mm
Unbraced length in y-axis;	$L_y = \mathbf{2700}$ mm
Effective length factor in y-axis - Table 21;	$K_y = \mathbf{1}$
Effective length in y-axis;	$L_{ey} = L_y \times K_y = \mathbf{2700}$ mm

Section properties

Cross sectional area of member;	$A = N \times b \times h = \mathbf{11250}$ mm^2
Section modulus;	$Z_x = N \times b \times h^2 / 6 = \mathbf{281250}$ mm^3
	$Z_y = h \times (N \times b)^2 / 6 = \mathbf{140625}$ mm^3
Second moment of area;	$I_x = N \times b \times h^3 / 12 = \mathbf{21093750}$ mm^4
	$I_y = h \times (N \times b)^3 / 12 = \mathbf{5273437}$ mm^4
Radius of gyration;	$i_x = \sqrt{(I_x / A)} = \mathbf{43.3}$ mm
	$i_y = \sqrt{(I_y / A)} = \mathbf{21.7}$ mm

Modification factors

Duration of loading - Table 17;	$K_3 = \mathbf{1.00}$
Total depth of member - cl.2.10.6;	$K_7 = (300\text{ mm} / h)^{0.11} = \mathbf{1.08}$
Load sharing - cl.2.9;	$K_8 = \mathbf{1.00}$

Members subject to axial compression - Table 22; K_{12} = **0.28**

Slenderness ratio - cl.2.11.4

Permissible slenderness ratio; λ_{max} = **180**

Slenderness ratio; $\lambda = \max(L_{ex} / i_x, L_{ey} / i_y)$ = **124.708**

PASS - Slenderness ratio is less than permissible slenderness ratio

Compression parallel to grain

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **1.872** N/mm^2

Applied compressive stress; $\sigma_{c\_a} = P / A$ = **0.445** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.238**

PASS - Applied compressive stress is less than permissible compressive stress

<u>B1 - 152 UC 23 Tie beam</u>

<u>Padstone P1 350x100x10mm Th. Steel Plate</u>

B2 - STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

Loading:

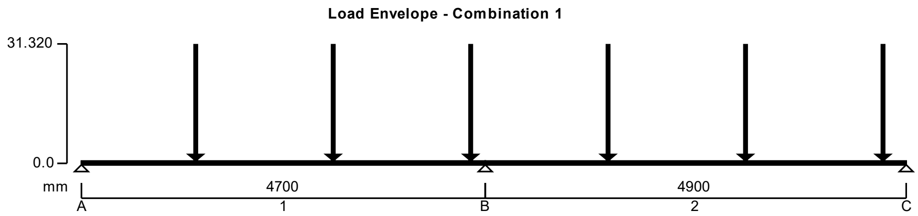
Dead load: 4.0 x 0.5 x 1.0 x 1.25 x 1.6 x 1.4 = 5.6 kN - point load

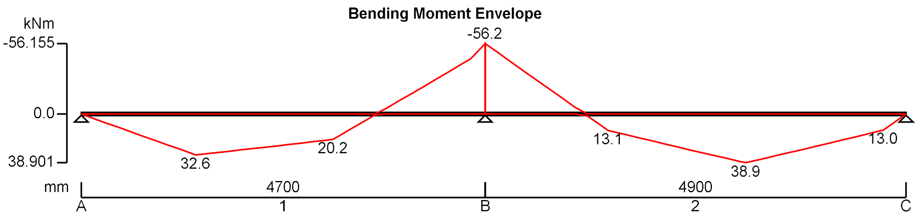
Nd = 2 x 5.7 = 11.4 kN

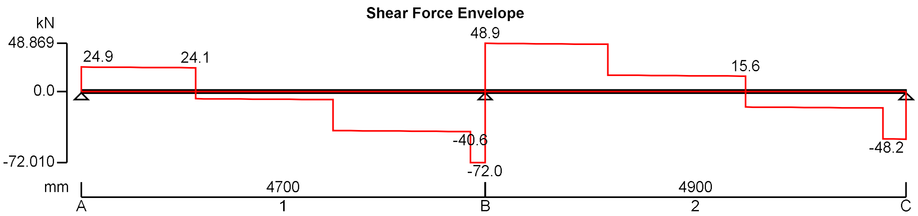
Imposed load: 4.0 x 0.5 x 0.75 x 1.25 x 1.6 x 16 = 4.8 kN

Nl = 2 x 4.8 = 9.6 kN

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead point load 11.4 kN at 1330 mm

Imposed point load 9.6 kN at 1330 mm
Dead point load 11.4 kN at 2930 mm
Imposed point load 9.6 kN at 2930 mm
Dead point load 11.4 kN at 4530 mm
Imposed point load 9.6 kN at 4530 mm
Dead point load 11.4 kN at 6130 mm
Imposed point load 9.6 kN at 6130 mm
Dead point load 11.4 kN at 7730 mm
Imposed point load 9.6 kN at 7730 mm
Dead point load 11.4 kN at 9330 mm
Imposed point load 9.6 kN at 9330 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
	Span 2	Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **38.9** kNm;	M_{min} = **-56.2** kNm
Maximum moment span 1;	M_{s1_max} = **32.6** kNm;	M_{s1_min} = **-56.2** kNm
Maximum moment span 2;	M_{s2_max} = **38.9** kNm;	M_{s2_min} = **-56.2** kNm
Maximum shear;	V_{max} = **48.9** kN;	V_{min} = **-72** kN
Maximum shear span 1;	V_{s1_max} = **24.9** kN;	V_{s1_min} = **-72** kN
Maximum shear span 2;	V_{s2_max} = **48.9** kN;	V_{s2_min} = **-48.2** kN
Deflection;	δ_{max} = **2.1** mm;	δ_{min} = **0** mm
Deflection span 1;	δ_{s1_max} = **1.6** mm;	δ_{s1_min} = **0** mm
Deflection span 2;	δ_{s2_max} = **2.1** mm;	δ_{s2_min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **24.9** kN;	R_{A_min} = **24.9** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **9.5** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **7.3** kN	
Maximum reaction at support B;	R_{B_max} = **120.9** kN;	R_{B_min} = **120.9** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **45.3** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **35.9** kN	
Maximum reaction at support C;	R_{C_max} = **48.2** kN;	R_{C_min} = **48.2** kN
Unfactored dead load reaction at support C;	R_{C_Dead} = **18** kN	
Unfactored imposed load reaction at support C;	$R_{C\_Imposed}$ = **14.4** kN	

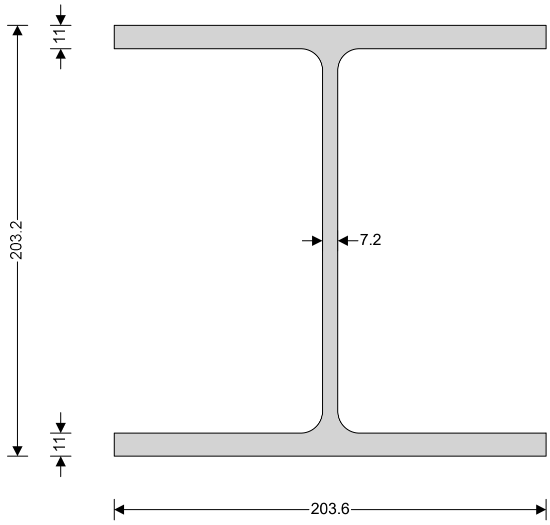
Section details

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

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** mm
Design strength;	p_y = **275** N/mm^2

Modulus of elasticity; $E = \mathbf{205000}$ N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Span 2 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; $K_x = \mathbf{1.00}$

Effective length factor in minor axis; $K_y = \mathbf{1.00}$

Effective length factor for lateral-torsional buckling; $K_{LT.A} = \mathbf{1.00}$;

$K_{LT.B} = \mathbf{1.00}$;

$K_{LT.C} = \mathbf{1.00}$;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = \mathbf{160.8}$ mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{101.8}$ mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{72}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{1463}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{241.4}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 2 - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s2\_max}), \text{abs}(M_{s2\_min})) = \mathbf{56.2}$ kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.5 \times p_y \times Z_{xx}) = \mathbf{136.8}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s2} = \mathbf{4900}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{95.437}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.847}$

Torsional index; $x = \mathbf{17.713}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.799}$

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

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{64.564}$

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

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

Bending strength - Section 4.3.6.5

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

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.212}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{485.4}$ N/mm²

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{431.6}$ N/mm²

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{201.8}$ N/mm²

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{3.2}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{29.7}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{28.7}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{56.2}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{56.2}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.550}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{100.4}$ kNm

$M_b / m_{LT} = \mathbf{182.6}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s2} / 360 = \mathbf{13.611}$ mm

Maximum deflection span 2; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{2.087}$ mm

PASS - Maximum deflection does not exceed deflection limit

Padstone P2

$$\frac{(9.5+7.3)\cdot 10^3}{0.42\cdot 100} = 400\text{mm}$$

Provide 450x100x12mm Th. Steel Plate

B3 - STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

Loading:

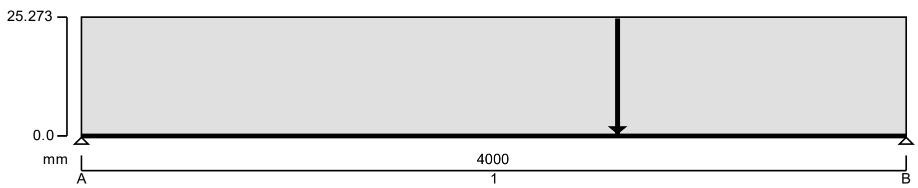
Dead load: 4.0 x 4.4 = 17.6 kN/m - brick wall

R = 9.5 kN - reaction from B2

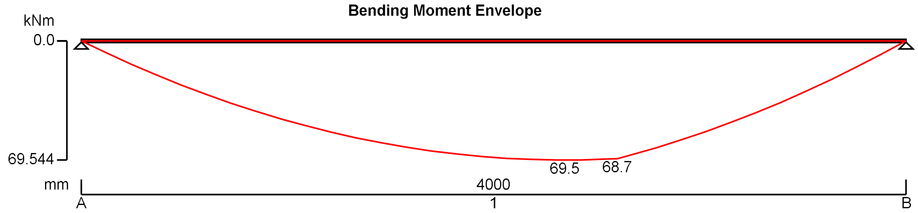
Imposed load: R = 7.3 kN - reaction from B2

TEDDS calculation version 3.0.05

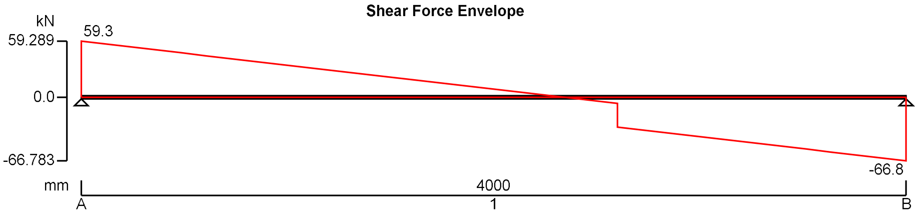
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



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 17.6 kN/m
	Dead point load 9.5 kN at 2600 mm
	Imposed point load 7.3 kN at 2600 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

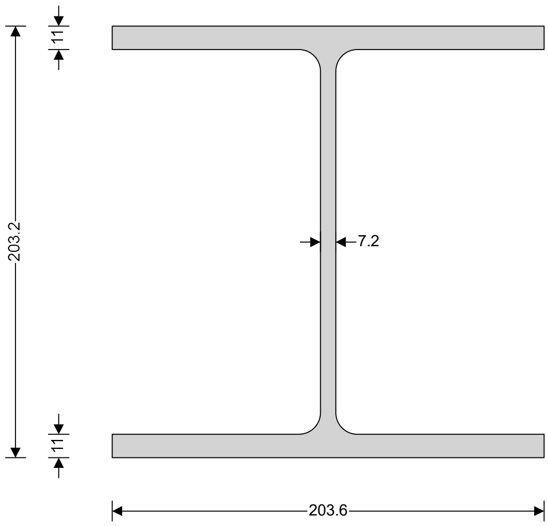
Maximum moment;	M_{max} = **69.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **59.3** kN;	V_{min} = **-66.8** kN
Deflection;	δ_{max} = **0.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **59.3** kN;	R_{A_min} = **59.3** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **39.4** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **2.6** kN	
Maximum reaction at support B;	R_{B_max} = **66.8** kN;	R_{B_min} = **66.8** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **42.3** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **4.7** kN	

Section details

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

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** 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

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

Internal compression parts - Table 11

Depth of section; $d = \mathbf{160.8}$ mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{101.8}$ mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{66.8}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{1463}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{241.4}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{69.5}$ kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{136.8}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \mathbf{4000}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{77.908}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.847}$

Torsional index; $x = \mathbf{17.713}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.844}$

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

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{55.686}$

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

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

Bending strength - Section 4.3.6.5

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

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.150}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{652.5}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{512.6}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{224}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{46.7}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{68}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{54.1}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{69.5}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{69.5}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.907}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx}$ = **111.4** kNm

M_b / m_{LT} = **122.9** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **11.111** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.921** mm

PASS - Maximum deflection does not exceed deflection limit

Padstone P3

$$\frac{(42.3 + 4.7)\cdot 10^3}{1.0 \cdot 100} = 470\text{mm}$$

Provide 500x100x20mm Th. Steel Plate

FIRST FLOOR:

C1 - STEEL MEMBER DESIGN (BS5950)

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

N = 120.9 + 0.6 = 121.5 kN

M = 0.1 x 121.5 = 12.2 kNm

TEDDS calculation version 3.0.05

Section details

Section type; **CHS 193.7x8.0 (Tata Steel Celsius)**

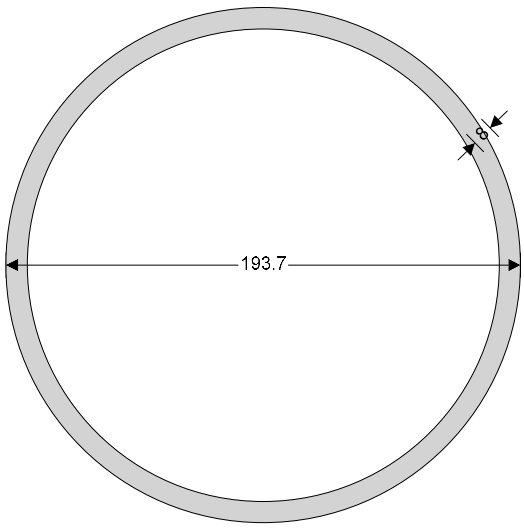
Steel grade; **S275**

From table 9: Design strength p_y

Thickness of element; t = **8.0** 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 = **2700** mm

Distance between minor axis restraints; L_y = **0** mm

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} = **1.00**;

Classification of cross sections - Section 3.5

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

Tubular sections - Table 12

$D / t = 24.2 \times \varepsilon <= 80 \times \varepsilon^2$; Class 3 semi-compact

Section is class 3 semi-compact

Moment capacity - Section 4.2.5

Design bending moment; M = **12.2** kNm

Effective plastic modulus - Section 3.5.6

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

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

Limiting value for class 2 compact web; $\beta_{2w} = \max(100 \times \varepsilon / (1 + 1.5 \times r1), 40 \times \varepsilon) =$ **100**

Limiting value for class 3 semi-compact web; $\beta_{3w} = \max(120 \times \varepsilon / (1 + 2 \times r2), 40 \times \varepsilon) =$ **120**

Effective plastic modulus - cl.3.5.6.4

$S_{eff} = \min(Z + 1.485 \times (S - Z) \times [\sqrt{[(140 / (D / t)) \times (275 \text{ N/mm}^2 / p_y)]} - 1], S) =$ **276047** mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z) =$ **68.7** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force; $F_c =$ **121.5** kN

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

Effective length for buckling; $L_{Ex} = L_x \times K_x =$ **2700** mm

Slenderness ratio - cl.4.7.2; $\lambda_x = L_{Ex} / r_{xx} =$ **41.086**

Compressive strength - Section 4.7.5

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

Strut curve - Table 23; a

Robertson constant; $\alpha_x =$ **2.0**

Perry factor; $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 =$ **0.048**

Euler stress; $p_{Ex} = \pi^2 \times E / \lambda_x^2 =$ **1198.6** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 =$ **765.5** N/mm^2

Compressive strength - Annex C.1; $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) =$ **259.2** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4; $P_{cx} = A \times p_{cx} =$ **1209.6** 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; $F_c / (A \times p_y) + M / M_c =$ **0.272**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b; $M_{LT} = M_x =$ **12.15** kNm

Equiv uniform mnt factor - major axis flex buckling; $m_x =$ **1.000**

Buckling resistance check - cl.4.8.3.3.3; $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx}) =$ **0.286**

PASS - Member buckling resistance checks are satisfied

GENERAL NOTES:

1: All dimension to be verified on site.
2: All drawings to be read in conjunction with Architect Drawings.
3: All steelwork design and fabrication in accordance with BS 5950.
4: Apply 2 coats of red oxide primer to all steel prior to erection.
5. All structural steelwork to be Mild Steel Grade S275, designed, fabricated & erected in accordance with B5950 Part 1. Details of main connections are shown on drawings. All other connections to have a minimum of 2No. M20 8.8 grade bolts, sherardized or zinc plated (generally use 12mm end plate and 4No M20 8.8 grade bolts)
6: All welding to be min. 6 mm fillet welds.
6: All bolts to be grade 8.8. For Splice connection use HSFG Bolts.
7. Timber joists to be min. grade C16.
8: Double joist to be bolted together with M10 bolts + 63 dia. TP connectors and washers plate @ 400 mm c/c.
9: Connections:
Timber/Masonry: BAT SPH HANGERS
Timber/Timber: BAT JIFFY HANGERS or Framing Anchors.
10: Allow for Bat M305 Straps @ 1200 mm c/c for restraints to all structural levels.
11: Concrete Padstones to be grade C25 (1:2:4).
12: Temporary propping by Contractor.
13: All works to be approved by Building Control Officer.
14: Mass concrete foundation to be grade C20(SR). General RC to be grade C35.
15: All Waterproofing and Drainage to Architect specification.
16: New brickwork to be 21 N/mmsq. New blockwork to be min. 7 N/mmsq setin 1:1:6 mortar.
17. Underside of the foundation to be found on undisturbed ground and to be approved by Building Control Surveyor on site.