DESIGN BASIS

Superimposed loading to BS6399-1:1996 & BS6399-3:1998

Structural steelwork to BS5950-1:2000

Structural timber to BS5268-2:2002

Masonry stresses to BS5628-1:2005

These calculations to be read in conjunction with relevant architectural details

UNIT LOADING

TIMBER FLOOR LOADING (1ST FLOOR)

Dead load

Boards — $Floor_{1\_D1}$ = **0.15** kN/m^2

Joists — $Floor_{1\_D2}$ = **0.20** kN/m^2

Ceiling — $Floor_{1\_D3}$ = **0.25** kN/m^2

Total dead load — $Floor_{1\_DL}$ = sum($Floor_{1\_D1}$,$Floor_{1\_D2}$,$Floor_{1\_D3}$) = **0.60** kN/m^2

Imposed load

Imposed load — $Floor_{1\_I1}$ = **2.00** kN/m^2

Partitions — $Floor_{1\_I2}$ = **0.00** kN/m^2

Total imposed load — $Floor_{1\_IL}$ = sum($Floor_{1\_I1}$,$Floor_{1\_I2}$) = **2.00** kN/m^2

Total 1st floor loads

Unfactored foundation design loads — w_{floor1_u} = $Floor_{1\_DL}$ + $Floor_{1\_IL}$ = **2.60** kN/m^2

Factored design loads — w_{floor1_f} = 1.4 × $Floor_{1\_DL}$ + 1.6 × $Floor_{1\_IL}$ = **4.04** kN/m^2

ROOF LOADING (PITCHED TILED ROOF)

Roof slope θ = **42.0** °

Dead load

Tiles — $Roof_{D1}$ = **0.70** kN/m^2

Battens — $Roof_{D2}$ = **0.05** kN/m^2

Felt — $Roof_{D3}$ = **0.05** kN/m^2

Rafters — $Roof_{D4}$ = **0.35** kN/m^2

Dead load on slope — $Roof_{DL\_sroof}$ = sum($Roof_{D1}$,$Roof_{D2}$,$Roof_{D3}$,$Roof_{D4}$) = **1.15** kN/m^2

Total dead load on plan — $Roof_{DL}$ = $Roof_{DL\_sroof}$ / cos(θ) $_f$= **1.55** kN/m^2

Imposed load

Roof imposed load — $Roof_{IL}$ = **0.40** kN/m^2 on plan

Total roof loads

Unfactored foundation design loads — w_{roof_u} = $Roof_{DL}$ + $Roof_{IL}$ = **1.95** kN/m^2

Factored design loads — w_{roof_f} = 1.4 × $Roof_{DL}$ + 1.6 × $Roof_{IL}$ = **2.81** kN/m^2

WIND LOADING

WIND LOADING (EN1991-1-4)

TEDDS calculation version 3.0.13

Building data

Type of roof	Duopitch		
Length of building	L = **8500** mm	Width of building	W = **4800** mm
Pitch of roof	α_0 = **42.0** deg		
Total height	h = **6261** mm		

Basic values

Location	Battle		
Wind speed velocity (FigNA.1)	$v_{b,map}$ = **22.0** m/s	Distance to shore	L_{shore} = **8.00** km
Altitude above sea level	A_{alt} = **75.0**m	Altitude factor	c_{alt} = **1.075**
Fund wind speed velocity	$v_{b,0}$ = **23.7** m/s	Direction factor	c_{dir} = **1.00**
Season factor	c_{season} = **1.00**	Probability factor	c_{prob} = **1.00**
Basic wind speed (Exp. 4.1)	v_b = **23.7** m/s	Ref mean velocity pressure	q_b = **0.343** kN/m^2

Orography

Orography factor not signif	c_o = 1.0
Terrain category	Country
Displacement height	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 2 parts as the height h is greater than b but less than 2b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height	z = **4100**mm
Exposure factor (Figure NA.7)	c_e = **1.93**
Peak velocity pressure	q_p = **0.66** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**	Size factor (Table NA.3)	c_s = **0.93**
Dynamic factor (Figure NA.9)	c_d = **1.02**	Structural factor	c_sc_d = **0.949**

Peak velocity pressure - windward wall (lower part) - Wind 90 deg

Reference height	z = **4800**mm
Exposure factor (Figure NA.7)	c_e = **2.03**
Peak velocity pressure	q_p = **0.70** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**	Size factor (Table NA.3)	c_s = **0.94**
Dynamic factor (Figure NA.9)	c_d = **1.04**	Structural factor	c_sc_d = **0.949**

Peak velocity pressure - windward wall (upper part) - Wind 90 deg and roof

Reference height	z = **6261**mm
Exposure factor (Figure NA.7)	c_e = **2.20**
Peak velocity pressure	q_p = **0.75** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**	Size factor (Table NA.3)	c_s = **0.96**
Dynamic factor (Figure NA.9)	c_d = **1.04**	Structural factor	c_sc_d = **0.949**

Structural factor

Structural damping δ_s = **0.100**
Size factor (Table NA.3) c_s = **0.94**
Dynamic factor (Figure NA.9) c_d = **1.04**
Structural factor c_sc_d = **0.949**

Structural factor - roof 0 deg

Structural damping δ_s = **0.100**
Size factor (Table NA.3) c_s = **0.93**
Dynamic factor (Figure NA.9) c_d = **1.02**
Structural factor c_sc_d = **0.949**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.75** kN/m^2

Pressures and forces

Net pressure $p = c_sc_d \times q_p \times c_{pe} - q_{p,i} \times c_{pi}$

Net force $F_w = p_w \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} -0.30, + c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.80	0.75	0.80	4.86	3.88
G (+ve)	0.58	0.75	0.64	4.86	3.11
H (+ve)	0.64	0.75	0.68	17.73	12.12
I (+ve)	-0.50	0.75	-0.13	17.73	-2.33
J (+ve)	-0.82	0.75	-0.36	9.72	-3.50

Total vertical net force $F_{w,v}$ = **9.87** kN
Total horizontal net force $F_{w,h}$ = **16.69** kN

Walls load case 1 - Wind 0, c_{pi} -0.30, + c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.75	-0.63	8.27	-5.23
B	-0.80	0.75	-0.35	16.60	-5.74
D	0.80	0.66	0.73	34.85	25.40
E	-0.52	0.66	-0.10	34.85	-3.41

Overall loading

Leeward force overall F_l = **-3.4** kN
Windward force overall F_w = **25.4** kN
Lack of correlation (cl.7.2.2(3)) f_{corr} = **0.86**
Overall loading overall section $F_{w,D}$ = **39.2** kN

Roof load case 2 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.58	0.75	0.64	1.55	0.99
G (+ve)	0.48	0.75	0.57	1.55	0.88
H (+ve)	0.38	0.75	0.50	12.40	6.17
I (+ve)	0.28	0.75	0.43	39.40	16.80

Total vertical net force $F_{w,v}$ = **18.46** kN
Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 2 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.66	-0.53	3.94	-2.08
B	-0.80	0.66	-0.28	15.74	-4.36
C	-0.50	0.66	-0.09	15.17	-1.34
D_b	0.76	0.70	0.73	22.50	16.44
D_u	0.76	0.75	0.77	2.37	1.83
E	-0.43	0.75	-0.08	24.87	-2.02

Overall loading

Leeward force upper F_l = **-0.2** kN

Lack of correlation (cl.7.2.2(3)) f_{corr} = **0.85**

Leeward force bottom F_l = **-1.8** kN

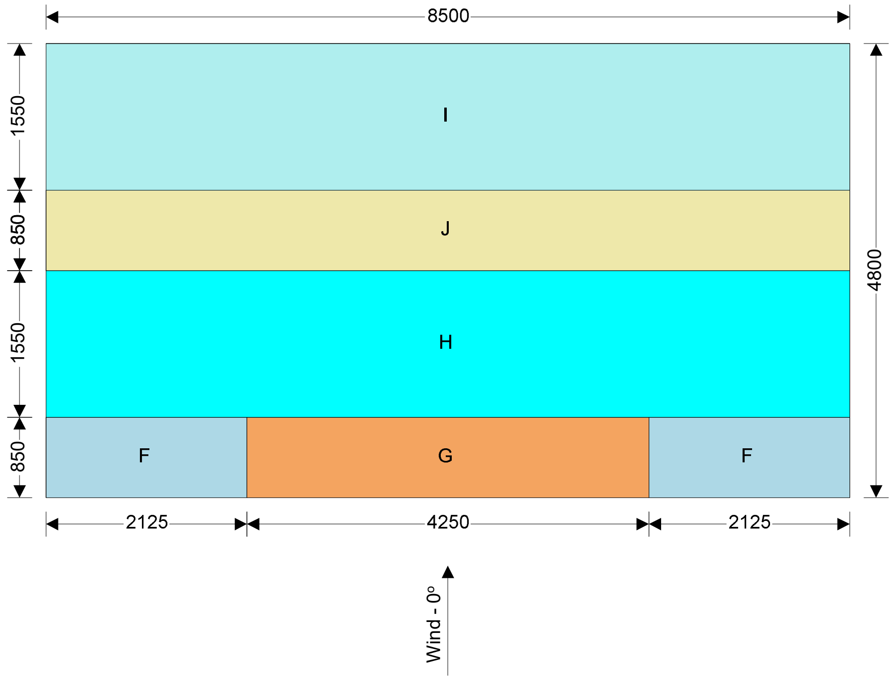
Lack of correlation (cl.7.2.2(3)) f_{corr} = **0.85**

Windward force upper F_w = **1.8** kN

Overall loading upper section $F_{w,u}$ = **1.7** kN

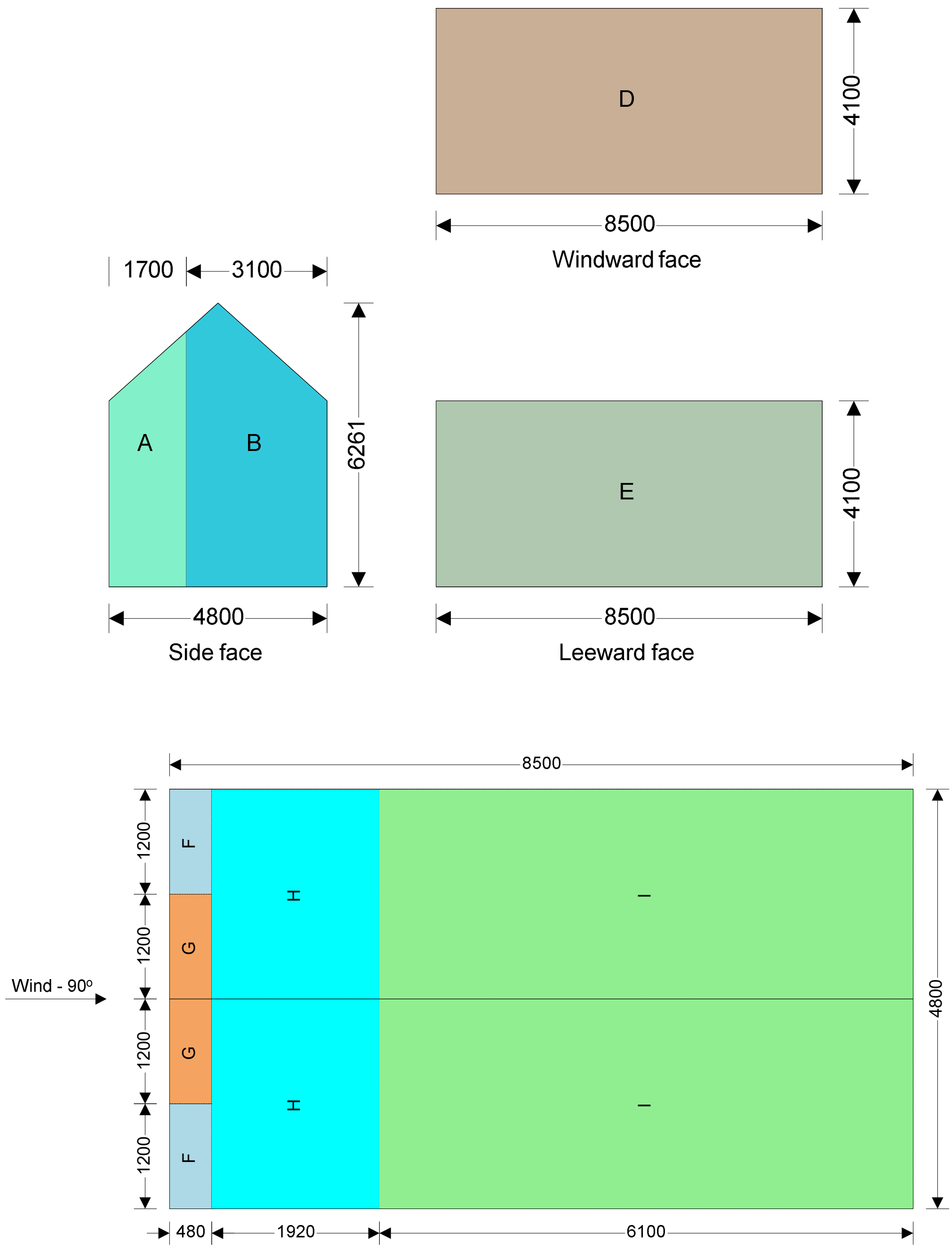
Windward force bottom F_w = **16.4** kN

Overall loading bottom section $F_{w,b}$ = **15.5** kN

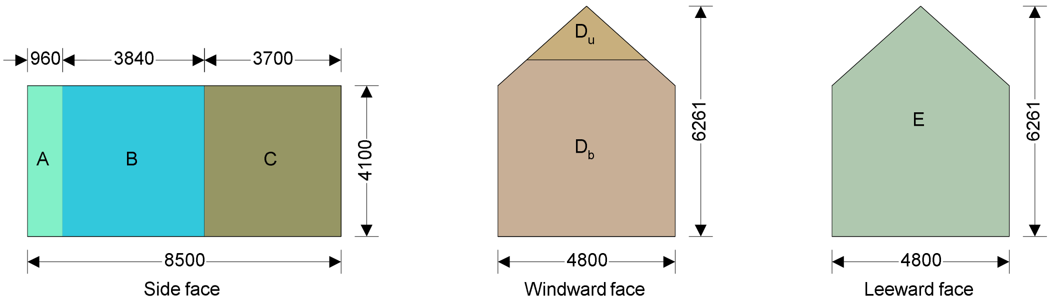


Plan view - Duopitch roof

E.A.R. Sheppard CONSULTING CIVIL AND STRUCTURAL ENGINEERS 5 Chiswick Place Eastbourne BN21 4NH	Project The Almonry, Battle				Job Ref. H4154	
	Section Proposed Alterations and Extension				Sheet no./rev. 5	
	Calcs by RJB	Date 20/12/2019	Chk'd by	Date	App'd by	Date



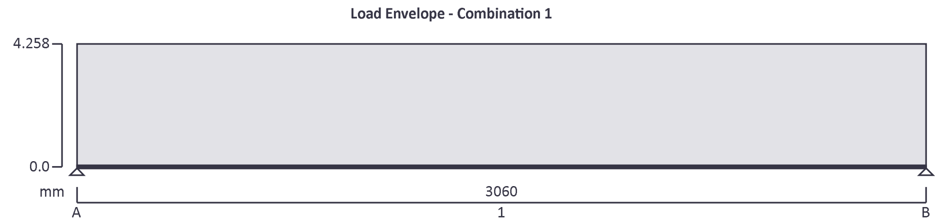
Plan view - Duopitch roof

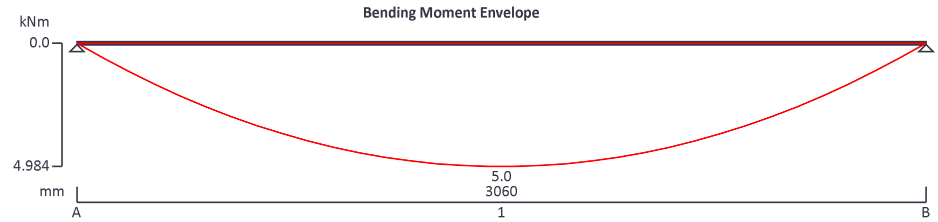


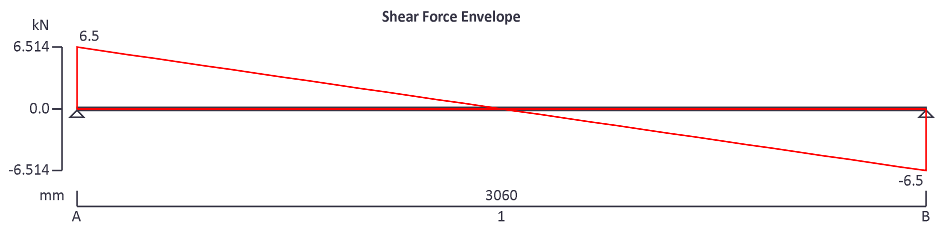
RIDGE BEAM

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

	Dead self weight of beam × 1
Roof = 1.55 x 4.22/2	Dead full UDL 3.270 kN/m
Roof = 0.4 x 4.22/2	Imposed full UDL 0.840 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} = **4.984** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4.984** kNm	
Maximum shear	F_{max} = **6.514** kN	F_{min} = **-6.514** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **6.514** kN	
Total load on beam	W_{tot} = **13.029** kN	
Reactions at support A	R_{A_max} = **6.514** kN	R_{A_min} = **6.514** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **5.229** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.285** kN	
Reactions at support B	R_{B_max} = **6.514** kN	R_{B_min} = **6.514** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **5.229** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.285** kN	

Timber section details

Breadth of sections	b = **100** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**D35**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **3060** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **22500** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **843750** mm^3
	Z_y = h × (N × b)^2 / 6 = **375000** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **94921875** mm^4
	I_y = h × (N × b)^3 / 12 = **18750000** mm^4
Radius of gyration	i_x = √(I_x / A) = **65.0** 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)$^{0.11}$ = **1.03**
Load sharing - cl.2.9	K_8 = **1.00**

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 × b) = **2.25**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.400** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.651** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.192**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **11.354** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **5.906** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.520**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **1.700** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.434** N/mm^2

τ_a / τ_{adm} = **0.255**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **6500** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551\text{ in}, 0.003 \times L_{s1})$ = **9.180** mm

Bending deflection δ_{b_s1} = **7.878** mm

Shear deflection δ_{v_s1} = **0.654** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **8.533** mm

δ_a / δ_{adm} = **0.929**

PASS - Total deflection is less than permissible deflection

1ST FLOOR JOISTS TO OFFICE/STORE

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth b = **50** mm

Joist depth h = **150** mm

Joist spacing s = **400** mm

Timber strength class **C24**

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} = **2900** 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 $F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m

Dead load F_{d_udl} = **0.44** kN/m^2

Imposed UDL(Long term) F_{i_udl} = **2.00** kN/m²

Imposed point load (Medium term) F_{i_pt} = **1.40** 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**

Consider long term loads

Load duration factor K_3 = **1.00**

Maximum bending moment M = **1.053** kNm

Maximum shear force V = **1.453** kN

Maximum support reaction R = **1.453** kN

Maximum deflection δ = **6.324** mm

Check bending stress

Bending stress σ_m = **7.500** N/mm²

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.904** N/mm²

Applied bending stress σ_{m_max} = M / Z = **5.616** N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.710** N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.291** N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.400** N/mm²

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm²

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.291** N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **8.700** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **6.074** mm

Shear deflection δ_{shear} = **0.250** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **6.324** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor K_3 = **1.25**

Maximum bending moment M = **1.227** kNm

Maximum shear force V = **1.693** kN

Maximum support reaction R = **1.693** kN

Maximum deflection δ = **6.198** mm

Check bending stress

Bending stress σ_m = **7.500** N/mm²

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **11.130** N/mm²

Applied bending stress $\sigma_{m\_max} = M / Z =$ **6.544** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau =$ **0.710** N/mm^2

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

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) =$ **0.339** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) $\sigma_{cp1} =$ **2.400** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 =$ **3.300** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b) =$ **0.339** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm) =$ **8.700** mm

Bending deflection (based on E_{mean}) $\delta_{bending} =$ **5.907** mm

Shear deflection $\delta_{shear} =$ **0.291** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear} =$ **6.198** mm

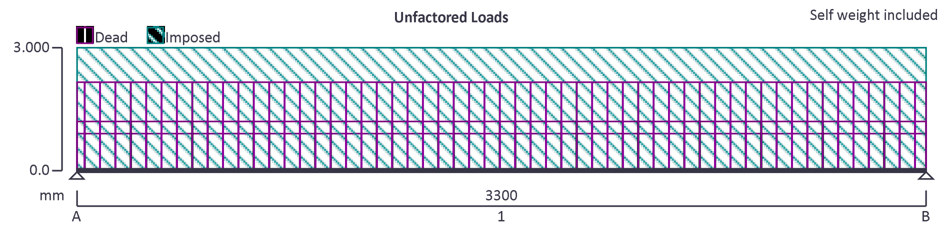
PASS - Actual deflection within permissible limits

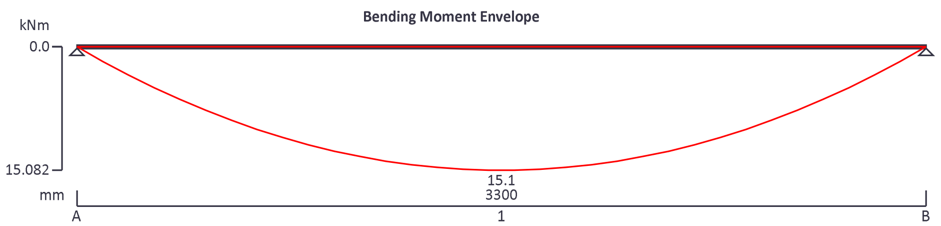
<u>STEEL BEAM OVER KITCHENETTE/ARCHIVE</u>

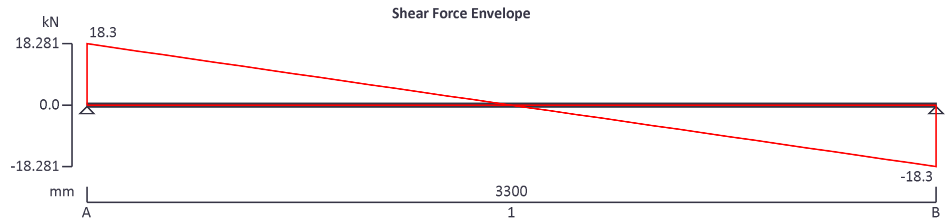
<u>STEEL BEAM ANALYSIS & DESIGN (BS5950)</u>

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Floor = 0.6 x 3.0/2 - Dead full UDL 0.9 kN/m
	Floor = 2.0 x 3.0/2 - Imposed full UDL 3 kN/m
	Partition = 0.5 x 2.4 - Dead full UDL 1.2 kN/m
	Wall = 4.8 x 0.45 - Dead full UDL 2.16 kN/m

Load combinations

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

Analysis results

Maximum moment	M_{max} = **15.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **18.3** kN	V_{min} = **-18.3** kN
Deflection	δ_{max} = **1.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **18.3** kN	R_{A_min} = **18.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.4** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5** kN	
Maximum reaction at support B	R_{B_max} = **18.3** kN	R_{B_min} = **18.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.4** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5** kN	

Section details

Section type	**UC 152x152x23 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **6.8** mm
Design strength	p_y = **275** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2

Lateral restraint

Span 1 has full lateral restraint

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

$d / t = 21.3 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **76.1** mm

$b / T = 11.2 \times \varepsilon <= 15 \times \varepsilon$ — Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **18.3** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **884** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **15.1** kNm

Effective plastic modulus - Section 3.5.6

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

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

Limiting value for class 2 compact web — $\beta_{2w} = 100 \times \varepsilon$ = **100**

Limiting value for class 3 semi-compact web — $\beta_{3w} = 120 \times \varepsilon$ = **120**

Effective plastic modulus - cl.3.5.6.2

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

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx})$ = **48.5** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **9.167** mm

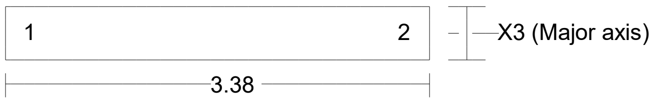
Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.808** mm

PASS - Maximum deflection does not exceed deflection limit

Detailed Results Table for Beam 1

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

Beam:
1



Supports Fixed-3 Fixed-3 (Table 13,14)

CONSTRAINTS

- Sections : Check
- Steel Grade: S275

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 250 (compr.) 300 (tens.)
- Allowable Deflection : 1/250
- Tension Area Reduction Factor : 1.00

INTERMEDIATE SUPPORTS

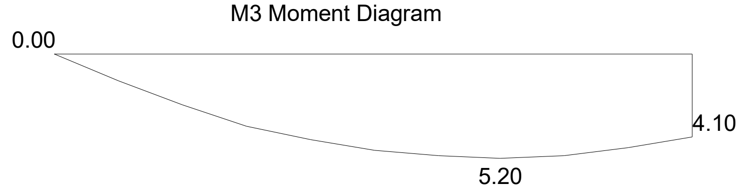
Lat.-Tors.	Cont. at	+
Compress.	Cont. at	Y

Section: UC 152x152x30

Ix = 1748.00 Iy = 560.00cm4 Sx = 248.0 Sy = 112.0cm3 Area = 38.30
hw = 157.60 bf = 152.90mm tw = 6.50 tf = 9.40mm
J = 10.50 x = 16.00 u = 0.85

DESIGN COMBINATION = 2

M3 Moment Diagram



Max. AXIAL Force = -12.43 (compr.) Max. SHEAR Force = 4.50

SECTION CLASSIFICATION: *** PLASTIC ***

Limiting Ratios:		Plastic	Compact	Semi-Compact	
d/t= 19.02	<	75.7	92.2	117.2	(e = 1.000 R = 0.012)
b/t= 8.13	<	9.0	10.0	15.0	

** Design Strength (py) = 275.0 **

DESIGN	EQUATION	FACTORS	VALUES	RESULT
V2 Shear (4.2.3)	Fv/Pv < 1.00	Av = 10.24	Fv = 4.50 Pv = 169.03	0.03
M3 Moment (4.2.5.2) Notes:	M / Mc < 1.00 LOW Shear Load Used for Moment Design	S = 248.00 Z = 221.83	M = 5.20 Mc = 68.20	0.08
Deflection	defl. / (L / 250) < 1.00		defl = 0.00115	0.09
Combined Stresses (Local) (4.8.2.3) (4.8.3)	((Mx)z1 / (Mrx)) + ((My)z2 / (Mry)) < 1.00	n = 0.01180	Mx = 5.20 My = 0.00 Mrx = 68.18 Mry = 30.80 z1 = 2.00 z2 = 1.00	0.01

Detailed Results Table for Beam 1

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

DESIGN	EQUATION	FACTORS	VALUES	RESULT
Axial Force (4.7.4) Note:	F / Agpc < 1.00 Slender. reduct. Strut Selection from table 23(b)	(kL/r)x =49 (kL/r)y =0 x = 0.97	F = 12.43 Ag = 38.30 pc = 238.03 y = 0.00	0.01
Overall Buckling - Simplified (4.8.3.3.1)	F/Pc + mxMx/pyZx + myMy/pyZy < 1.00	Mx = 5.20 My = 0.00 mx = 0.91	pyZx= 61.00 pyZy= 20.14 my = 1.00 Pc = 911.64	0.09
Overall Buckling Check - Exact (4.8.3.3.2) major axis buckling Note:	Fc/Pc + Mx/Max + My/May < 1 Max= Mcx / mx(1+F/2Pcx) EXACT approach governs	Pcx = 911.64 Pcy = 1053.25 mx = 0.91 my = 1.00 May= Mcy / 0.5*my	Mx = 5.20 Max = 74.07 My = 0.00 May = Pc = Pcx	0.08

Detailed Results Table for Beam 2

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

Beam: 2

2 — 3 — X3 (Major axis)

3.97

Supports Fixed-3 Fixed-3 (Table 13,14)

CONSTRAINTS

- Sections : Check
- Steel Grade: S275

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 250 (compr.) 300 (tens.)
- Allowable Deflection : 1/250
- Tension Area Reduction Factor : 1.00

INTERMEDIATE SUPPORTS

Lat.-Tors.	Cont. at	+
Compress.	Cont. at	Y

Section: UC 152x152x30

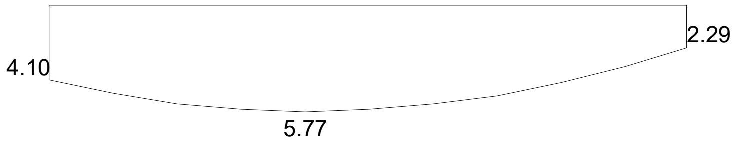
Ix = 1748.00 Iy = 560.00cm4 Sx = 248.0 Sy = 112.0cm3 Area = 38.30
hw = 157.60 bf = 152.90mm tw = 6.50 tf = 9.40mm
J = 10.50 x = 16.00 u = 0.85

Detailed Results Table for Beam 2

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

DESIGN COMBINATION = 2

M3 Moment Diagram



Max. AXIAL Force = -4.67 (compr.) Max. SHEAR Force = 2.97

SECTION CLASSIFICATION: *** PLASTIC ***

Limiting Ratios:		Plastic	Compact	Semi-Compact	
d/t= 19.02	<	78.3	96.9	118.9	(e = 1.000 R = 0.004)
b/t= 8.13	<	9.0	10.0	15.0	

** Design Strength (py) = 275.0 **

DESIGN	EQUATION	FACTORS	VALUES	RESULT
V2 Shear (4.2.3)	Fv/Pv < 1.00	Av = 10.24	Fv = 2.97 Pv = 169.03	0.02
M3 Moment (4.2.5.2) Notes:	M / Mc < 1.00 LOW Shear Load Used for Moment Design	S = 248.00 Z = 221.83	M = 5.77 Mc = 68.20	0.08
Deflection	defl. / L / 250 < 1.00		defl = 0.00212	0.13
Combined Stresses (Local) (4.8.2.3) (4.8.3)	(Mx)z1 / (Mrx) + (My)z2 / (Mry) < 1.00	n = 0.00443	Mx = 5.77 My = 0.00 Mrx = 68.20 Mry = 30.80 z1 = 2.00 z2 = 1.00	0.01
Axial Force (4.7.4) Note:	F / Agpc < 1.00 Slender. reduct. Strut Selection from table 23(b)	(kL/r)x =42 (kL/r)y =0 x = 0.71	F = 4.67 Ag = 38.30 pc = 247.55 y = 0.00	0.00
Overall Buckling - Simplified (4.8.3.3.1)	F / Pc + mxMx / pyZx + myMy / pyZy < 1.00	Mx = 5.77 My = 0.00 mx = 0.97	pyZx= 61.00 pyZy= 20.14 my = 1.00 Pc = 948.11	0.10
Overall Buckling Check - Exact (4.8.3.3.2) major axis buckling Note:	Fc / Pc + Mx / Max + My / May < 1 Max= Mcx / mx(1+F/2Pcx) EXACT approach governs	Pcx = 948.11 Pcy = 1053.25 mx = 0.97 my = 1.00 May= Mcy / 0.5*my	Mx = 5.77 Max = 70.40 My = 0.00 May = Pc = Pcx	0.09

Detailed Results Table for Beam 3

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

Beam: 3

4 — 3 — X3 (Major axis)

1.73

Supports: Fixed-3 — Fixed-3 (Table 13,14)

CONSTRAINTS

- Sections : Check
- Steel Grade: S275

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 250 (compr.) 300 (tens.)
- Allowable Deflection : 1/250
- Tension Area Reduction Factor : 1.00

INTERMEDIATE SUPPORTS

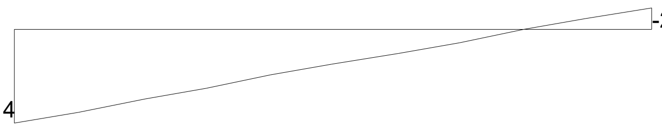
Lat.-Tors.	Cont. at	+
Compress.	Cont. at	Y

Section: UC 152x152x30

Ix = 1748.00 Iy = 560.00cm4 Sx = 248.0 Sy = 112.0cm3 Area = 38.30
hw = 157.60 bf = 152.90mm tw = 6.50 tf = 9.40mm
J = 10.50 x = 16.00 u = 0.85

DESIGN COMBINATION = 2

M3 Moment Diagram



Max. AXIAL Force = -9.41 (compr.) Max. SHEAR Force = 7.27

SECTION CLASSIFICATION: *** PLASTIC ***

Limiting Ratios:		Plastic	Compact	Semi-Compact	
d/t= 19.02	<	76.7	94.0	117.9	(e = 1.000 R = 0.009)
b/t= 8.13	<	9.0	10.0	15.0	

** Design Strength (py) = 275.0 **

DESIGN	EQUATION	FACTORS	VALUES	RESULT
V2 Shear (4.2.3)	Fv/Pv < 1.00	Av = 10.24	Fv = 7.27 Pv = 169.03	0.04
M3 Moment (4.2.5.2)	$\frac{M}{Mc}$ < 1.00	S = 248.00 Z = 221.83	M = 10.14 Mc = 68.20	0.15
Notes:	LOW Shear Load Used for Moment Design			
Deflection	$\frac{defl.}{L / 250}$ < 1.00		defl = 0.00030	0.04
Combined Stresses (Local) (4.8.2.3) (4.8.3)	$\left(\frac{Mx}{Mrx}\right)^{z1} + \left(\frac{My}{Mry}\right)^{z2}$ < 1.00	n = 0.00893	Mx = 10.14 My = 0.00 Mrx = 68.19 Mry = 30.80 z1 = 2.00 z2 = 1.00	0.02

Detailed Results Table for Beam 3

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

DESIGN	EQUATION	FACTORS	VALUES	RESULT
Axial Force (4.7.4) Note:	F / Agpc < 1.00 Slender. reduct. Strut Selection from table 23(b)	(kL/r)x =25 (kL/r)y =0 x = 1.00	F = 9.41 Ag = 38.30 pc = 267.01 y = 0.00	0.01
Lateral Torsional Buckling (4.3.6) (B.2)	M*mLT / Mb < 1.00 beam NOT LOADED (u= 0.85 v = 0.92 Le/ry = 45.2) Critical Segment from 0.00 to 1.73 on -z flange Segment End Moments: 10.14 and -2.29	Sx = 248.00 pb = 273.42 Le = 1.73 mL = 0.51 β = -0.23 λ = 35.00	M = 2.29 Ma = M*mLT Ma = 1.16 Mb = 67.81	0.02
Overall Buckling - Simplified (4.8.3.3.1)	F/Pc + mxMx/pyZx + myMy/pyZy < 1.00	Mx = 10.14 My = 0.00 mx = 0.51	pyZx= 61.00 pyZy= 20.14 my = 1.00 Pc = 1022.65	0.09
Overall Buckling Check - Exact (4.8.3.3.2) major axis buckling Note:	Fc/Pc + Mx/Max + My/May < 1 Max= Mcx / mx(1+F/2Pcx) EXACT approach governs	Pcx = 1022.65 Pcy = 1053.25 mx = 0.51 my = 1.00 May= Mcy / 0.5*my	Mx = 10.14 Max = 133.80 My = 0.00 May = Pc = Pcx	0.08

Detailed Results Table for Beam 4

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

Beam: 4

5 —— 4 —— X3 (Major axis)

4.89

Supports: Fixed-3 Fixed-3 (Table 13,14)

CONSTRAINTS

- Sections : Check
- Steel Grade: S275

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 250 (compr.) 300 (tens.)
- Allowable Deflection : 1/250
- Tension Area Reduction Factor : 1.00

INTERMEDIATE SUPPORTS

Lat.-Tors.	Cont. at	+
Compress.	Cont. at	Y

Section: UC 152x152x30

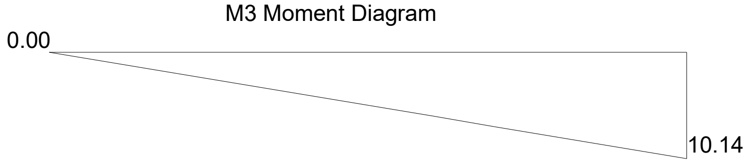
Ix = 1748.00 Iy = 560.00cm4 Sx = 248.0 Sy = 112.0cm3 Area = 38.30
hw = 157.60 bf = 152.90mm tw = 6.50 tf = 9.40mm
J = 10.50 x = 16.00 u = 0.85

Detailed Results Table for Beam 4

*Moments: kN*meter , Forces: kN , Stresses: mPa , Section prop.: cm.*

DESIGN COMBINATION = 2

M3 Moment Diagram



Max. AXIAL Force = -16.31 (compr.) Max. SHEAR Force = 2.07

SECTION CLASSIFICATION: *** PLASTIC ***

Limiting Ratios:	Plastic	Compact	Semi-Compact	
d/t= 19.02 <	74.5	90.0	116.4	(e = 1.000 R = 0.015)
b/t= 8.13 <	9.0	10.0	15.0	

** Design Strength (py) = 275.0 **

DESIGN	EQUATION	FACTORS	VALUES	RESULT
V2 Shear (4.2.3)	Fv/Pv < 1.00	Av = 10.24	Fv = 2.07 Pv = 169.03	0.01
M3 Moment (4.2.5.2) Notes:	M / Mc < 1.00 LOW Shear Load Used for Moment Design	S = 248.00 Z = 221.83	M = 10.14 Mc = 68.20	0.15
Deflection	defl. / (L / 250) < 1.00		defl = 0.00320	0.16
Combined Stresses (Local) (4.8.2.3) (4.8.3)	$\left(\frac{Mx}{Mrx}\right)^{z1} + \left(\frac{My}{Mry}\right)^{z2} < 1.00$	n = 0.01549	Mx = 10.14 My = 0.00 Mrx = 68.16 Mry = 30.80 z1 = 2.00 z2 = 1.00	0.02
Axial Force (4.7.4) Note:	F / Agpc < 1.00 Slender. reduct. Strut Selection from table 23(b)	(kL/r)x =70 (kL/r)y =0 x = 0.97	F = 16.31 Ag = 38.30 pc = 201.92 y = 0.00	0.02
Overall Buckling - Simplified (4.8.3.3.1)	$\frac{F}{Pc} + \frac{mxMx}{pyZx} + \frac{myMy}{pyZy} < 1.00$	Mx = 10.14 My = 0.00 mx = 0.60	pyZx= 61.00 pyZy= 20.14 my = 1.00 Pc = 773.34	0.12
Overall Buckling Check - Exact (4.8.3.3.2) major axis buckling Note:	$\frac{Fc}{Pc} + \frac{Mx}{Max} + \frac{My}{May} < 1$ $Max = \frac{Mcx}{mx(1+F/2Pcx)}$ EXACT approach governs	Pcx = 773.34 Pcy = 1053.25 mx = 0.60 my = 1.00 $May = \frac{Mcy}{0.5*my}$	Mx = 10.14 Max = 112.48 My = 0.00 May = Pc = Pcx	0.11