Structural Calculations

Address: 19 Belmont Park, London, SE13 5BJ

Job no : 1666

Structural Calculations for the proposed additional floor to 19 Belmont Park, London, SE13 5BJ.
The property is a two storey midterraced house. It is of traditional construction, with cavity masonry elevation walls, suspended timber floors and timber roof with tile finish. The rafters are supported by timber purlin and diagonal struts that are in turn supported on internal loadbearing wall.

UNFACTURED LOADING	DEAD LOAD	LIVE LOAD	WIND LOAD
Pitched Roof			
Wind			0.70kN/m2
Imposed		0.75 kN/m2	
Tiles	0.70 kN/m2		
Insulation	0.05 KN/m2		
Rafters, Battens & Lining	0.20 kN/m2		
Total on Slope	0.95 kN/m2		
Load on plan (11 degrees)	**0.97 kN/m2**		
Existing Flat Ceiling (over First Floor)			
Imposed		0.25 kN/m2	
Plasterboard and skim	0.20kN/m2		
Joists	0.05 kN/m2		
Total on plan	**1.22 kN/m2**	**1.00 kN/m2**	**0.70 kN/m2**
Ceiling			
Imposed		0.25 kN/m2	
Joists + insulation	0.10 kN/m2		
Plasterboard	0.15 kN/m2		
Total	**0.25 kN/m2**	**0.25 kN/m2**	
Timber Floor			
Imposed		1.50 kN/m2	
Joists	0.13 kN/m2		
Insulation	0.04 kN/m2		
Floor Boards	0.18 kN/m2		
Ceiling	0.15 kN/m2		
Total	**0.50 kN/m2**	**1.50 kN/m2**	
Cavity Wall			
Brickwork	2.04 kN/m2		
Plaster	0.20 kN/m2		
Blockwork	1.80 kN/m2		
Total	**4.00 kN/m2**		

1. Load on Ceiling Beam R/1

Steel Beam span		5.80	m		o/p								
										DEAD		**LIVE**	
** Load from Ceiling													
Roof span on plan =					7.80	m	o/p						
	LOAD				SPAN								
Dead =	0.25	kN/m2		x	5.1	m	x	0.5	=	0.98	kN/m		
Live =	0.25	kN/m2		x	5.1	m	x	0.5	=			0.98	kN/m

Total Load on Steel Beam (SLS)

	DEAD		LIVE	
Dead =	**1.0**	**kN/m**		
Live =			**1.0**	**kN/m**

Beam self weight
sw= 0.2 kn/m

<u>REACTION RA</u>			**<u>REACTION RB</u>**		
RA_{DEAD}=	**3.41**	**KN**	**RB_{DEAD}=**	**3.41**	**KN**
RA_{LIVE}=	**2.83**	**KN**	**RB_{LIVE}=**	**2.83**	**KN**

2. Load on Steel Beam 2/1

Steel Beam spans		5.80	m		o/p								
										DEAD		**LIVE**	
* *Load from Second Floor													
				span on plan =	5.00	m	o/p						
	LOAD				SPAN								
Dead =	0.50	kN/m2		x	5.0	m	x	0.5	=	1.25	kN/m		
Live =	1.50	kN/m2		x	5.0	m	x	0.5	=			3.75	kN/m

Total Load on Steel Beam (SLS)

	DEAD		LIVE	
Dead =	**1.3**	**kN/m**		
Live =			**3.8**	**kN/m**

Beam self weight
sw= 0.4 kn/m

<u>REACTION RA</u>			**<u>REACTION RB</u>**		
RA_{DEAD}=	**4.79**	**KN**	**RB_{DEAD}=**	**4.79**	**KN**
RA_{LIVE}=	**10.88**	**KN**	**RB_{LIVE}=**	**10.88**	**KN**

3. Load on Steel Beam 2/2

Steel Beam spans		5.80	m		o/p								
										DEAD		**LIVE**	
* *Load from Second Floor													
				span on plan =	5.50	m	o/p						
	LOAD				SPAN								
Dead =	0.50	kN/m2		x	5.5	m	x	0.5	=	1.38	kN/m		
Live =	1.50	kN/m2		x	5.5	m	x	0.5	=			4.13	kN/m

Total Load on Steel Beam (SLS)

	DEAD		LIVE	
Dead =	**1.4**	**kN/m**		
Live =			**4.1**	**kN/m**

Beam self weight
sw= 0.4 kn/m

<u>REACTION RA</u>			**<u>REACTION RB</u>**		
RA_{DEAD}=	**5.15**	**KN**	**RB_{DEAD}=**	**5.15**	**KN**
RA_{LIVE}=	**11.96**	**KN**	**RB_{LIVE}=**	**11.96**	**KN**

4. LOAD ON Existing Fundation

DEAD **LIVE**

* *Load from external wall

	Wall Height 9.00 m											
	LOAD			SPAN								
Dead =	4.00	kN/m2	x	9.0	=				36.00	kN/m		

* *Load from Main roof

	Roof Rafter span on plan =			3.90	m	o/p						
	LOAD			SPAN								
Dead =	1.22	kN/m2	x	3.9	m	x	0.5	=	2.38	kN/m		
Live =	1.00	kN/m2	x	3.9	m	x	0.5	=			1.95	kN/m

* *Load from Second Floor

			span on plan =	2.80	m	o/p						
	LOAD			SPAN								
Dead =	0.50	kN/m2	x	2.8	m	x	0.5	=	0.70	kN/m		
Live =	1.50	kN/m2	x	2.8	m	x	0.5	=			2.10	kN/m

Total Load on Foundation (SLS) UDL				
Dead =	**39.1**	**kN/m**		
Live =			**4.1**	**kN/m**

5. Load on Lintel

	Lintel span 1.9 m			o/p								
									DEAD		**LIVE**	

* *Load from Main roof

	Roof Rafter span on plan =			3.90	m	o/p						
	LOAD			SPAN								
Dead =	1.22	kN/m2	x	3.9	m	x	0.5	=	2.38	kN/m		
Live =	1.00	kN/m2	x	3.9	m	x	0.5	=			1.95	kN/m

* *Load from external wall

	Wall Height 0.50 m											
	LOAD			SPAN								
Dead =	4.00	kN/m2	x	0.5	=				2.00	kN/m		

Total Load on Lintell (SLS)				
Dead =	**2.4**	**kN/m**		
Live =			**2.0**	**kN/m**

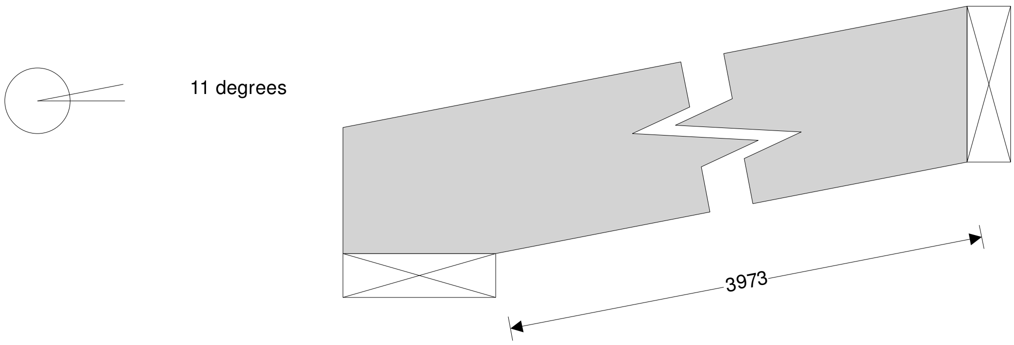
Working Load on Lintel L1 (ULS)

V= 8.2 kN

SWM STRUCTURES Suite 101 C1 Peel House, Morden, SM4 5BT stan@swmstructures.co.uk	Project 17 Belmont Park, London, SE13 5BJ				Job no. 1665	
	Calcs for TIMBER RAFTER				Start page no./Revision 6	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

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

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

Rafter spacing — $s = \mathbf{400}$ mm

Rafter slope — $\alpha = \mathbf{11.0}$ deg

Clear span of rafter on horizontal — $L_{clh} = \mathbf{3900}$ mm

Clear span of rafter on slope — $L_{cl} = L_{clh} / \cos(\alpha) = \mathbf{3973}$ mm

Rafter span — **Single span**

Timber strength class — **C16**

Section properties

Cross sectional area of rafter — $A = b \times h = \mathbf{8750}$ mm^2

Section modulus — $Z = b \times h^2 / 6 = \mathbf{255208}$ mm^3

Second moment of area — $I = b \times h^3 / 12 = \mathbf{22330729}$ mm^4

Radius of gyration — $r = \sqrt{(I / A)} = \mathbf{50.5}$ mm

Loading details

Rafter self weight — $F_j = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.03}$ kN/m

Dead load on slope — $F_d = \mathbf{0.75}$ kN/m^2

Imposed load on plan — $F_u = \mathbf{0.75}$ kN/m^2

Imposed point load — $F_p = \mathbf{0.90}$ kN

Modification factors

Section depth factor — $K_7 = (300 \text{ mm} / h)^{0.11} = \mathbf{1.06}$

Load sharing factor — $K_8 = \mathbf{1.10}$

Consider long term load condition

Load duration factor — $K_3 = \mathbf{1.00}$

Total UDL perpendicular to rafter — $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.321}$ kN/m

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

Effective span — $L_{eff} = L_{cl} + L_b = \mathbf{3978}$ mm

Check bending stress

Bending stress parallel to grain — $\sigma_m = \mathbf{5.300}$ N/mm^2

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 — $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{2.485}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

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} = **0.52**

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **3.887** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.417** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **9.231** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.965**

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.524 < 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 $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.109** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff}$ = **11.935** mm

Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **5.321** mm

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.158** mm

Total deflection $\delta_{max} = \delta_b + \delta_s$ = **5.479** mm

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor K_3 = **1.25**

Total UDL perpendicular to rafter $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha)$ = **0.610** kN/m

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

Effective span $L_{eff} = L_{cl} + L_b$ = **3983** mm

Check bending stress

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

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

Applied bending stress $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **4.737** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

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} = **0.47**

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.393** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.795** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **9.209** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.939**

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.833 < 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	$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.208** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff}$ = **11.949** mm
Bending deflection	$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **10.168** mm
Shear deflection	$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.301** mm
Total deflection	$\delta_{max} = \delta_b + \delta_s$ = **10.469** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor	K_3 = **1.50**
Total UDL perpendicular to rafter	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.321** kN/m
Notional bearing length	$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **9** mm
Effective span	$L_{eff} = L_{cl} + L_b$ = **3982** mm

Check bending stress

Bending stress parallel to grain	σ_m = **5.300** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **9.279** N/mm^2
Applied bending stress	$\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z)$ = **5.936** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

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} = **0.43**
Permissible compressive stress	$\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.780** N/mm^2
Applied compressive stress	$\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha))/(2 \times A) + F_p \times \sin(\alpha)/A$ = **0.437** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress	$\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **9.214** N/mm^2
Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.970**
Combined axial compression and bending check	$\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.751 < 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	$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.261** N/mm^2

PASS - Applied shear stress within permissible limits

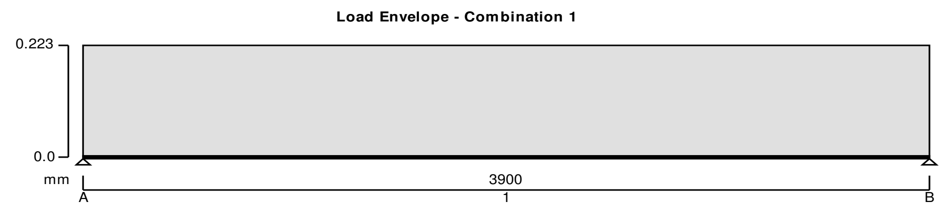
Check deflection

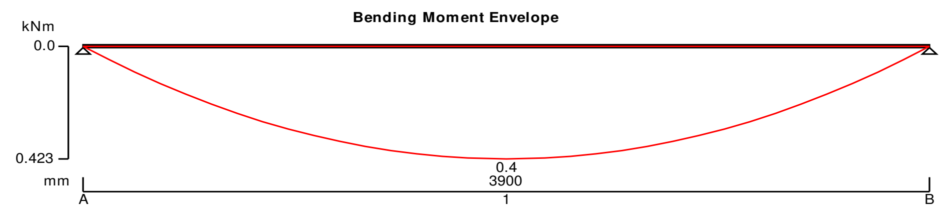
Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff}$ = **11.946** mm
Bending deflection	$\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha)/ 48) / (E_{mean} \times I)$ = **11.254** mm
Shear deflection	$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha))/(5 \times E_{mean} \times A)$ = **0.378** mm
Total deflection	$\delta_{max} = \delta_b + \delta_s$ = **11.632** mm

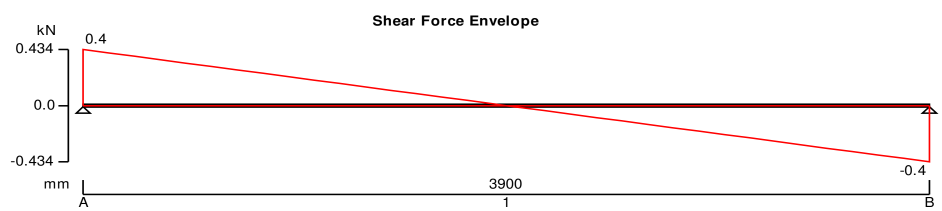
PASS - Total deflection within permissible limits

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Dead full UDL 0.100 kN/m

Imposed full UDL 0.100 kN/m

Dead self weight of beam × 1

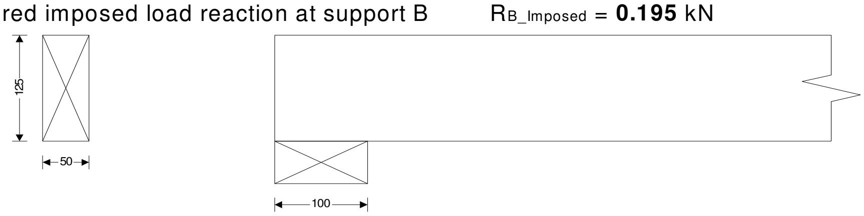
Load combinations

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} = **0.423** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.423** kNm	
Maximum shear	F_{max} = **0.434** kN	F_{min} = **-0.434** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **0.434** kN	
Total load on beam	W_{tot} = **0.868** kN	

Reactions at support A	R_{A_max} = **0.434** kN	R_{A_min} = **0.434** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.239** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.195** kN	
Reactions at support B	R_{B_max} = **0.434** kN	R_{B_min} = **0.434** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.239** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.195** kN	



Timber section details

Breadth of sections — b = **50** mm

Depth of sections — h = **125** mm

Number of sections in member — N = **1**

Overall breadth of member — $b_b = N \times b$ = **50** mm

Timber strength class — **C16**

Member details

Service class of timber — **1**

Load duration — **Long term**

Length of bearing — L_b = **100** mm

The beam is part of a load-sharing system consisting of four or more members

Section properties

Cross sectional area of member — $A = N \times b \times h$ = **6250** mm^2

Section modulus — $Z_x = N \times b \times h^2 / 6$ = **130208** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **52083** mm^3

Second moment of area — $I_x = N \times b \times h^3 / 12$ = **8138021** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **1302083** mm^4

Radius of gyration — $i_x = \sqrt{(I_x / A)}$ = **36.1** mm

$i_y = \sqrt{(I_y / A)}$ = **14.4** 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 — $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.10**

Load sharing - cl.2.9 — K_8 = **1.10**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19 — **4.00**

Actual depth-to-breadth ratio — $h / (N \times b)$ = **2.50**

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 — $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.087** N/mm^2

Suite 101 C1
Peel House,
Morden,
SM4 5BT
stan@swmstructures.co.uk

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.036**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.419** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **3.251** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.507**

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 $\tau_a = 3 \times F / (2 \times A)$ = **0.104** N/mm^2

τ_a / τ_{adm} = **0.141**

PASS - Applied shear stress is less than permissible shear stress

Deflection

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

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1})$ = **11.700** mm

Bending deflection δ_{b_s1} = **9.366** mm

Shear deflection δ_{v_s1} = **0.148** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **9.514** mm

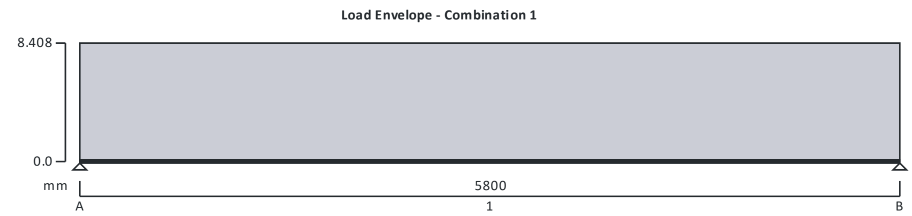
δ_a / δ_{adm} = **0.813**

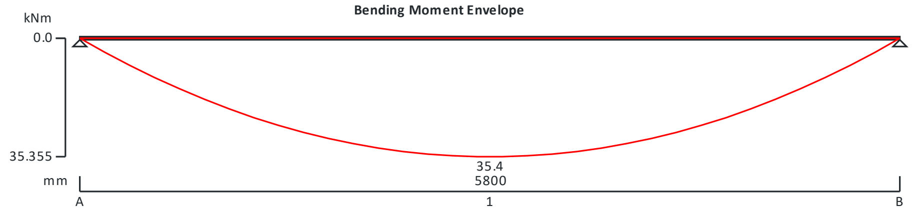
PASS - Total deflection is less than permissible deflection

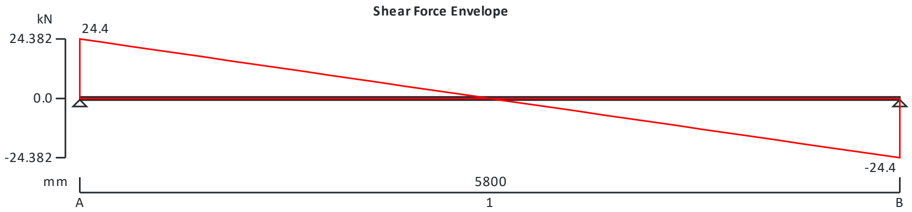
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead full UDL 1.3 kN/m
	Imposed full UDL 3.8 kN/m
	Dead self weight of beam × 1

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} = **35.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **24.4** kN	V_{min} = **-24.4** kN
Deflection	δ_{max} = **17.8** 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} = **4.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **11** kN	
Maximum reaction at support B	R_{B_max} = **24.4** kN	R_{B_min} = **24.4** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **4.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **11** kN	

Section details

Section type **UC 152x152x37 (BS4-1)**

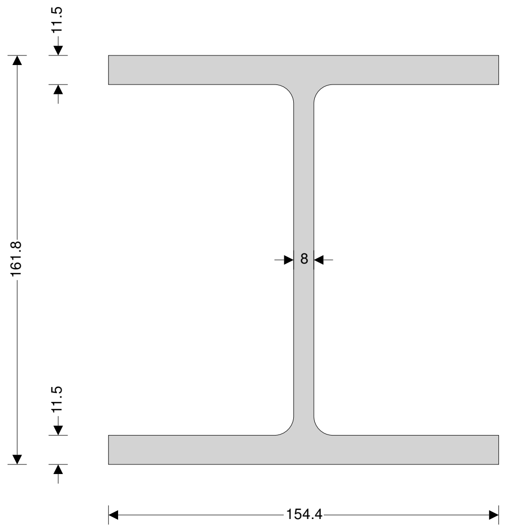
Steel grade **S275**

From table 9: Design strength p_y

Thickness of element max(T, t) = **11.5** 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 $K_{LT.A}$ = **1.20** + 2 × D

$K_{LT.B}$ = **1.20** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section d = **123.6** mm

$d / t = 15.5 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section — $b = B / 2 =$ **77.2** mm

$b / T = 6.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **24.4** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D =$ **1294** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v =$ **213.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **35.4** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **84.9** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.2 \times L_{s1} + 2 \times D =$ **7284** mm

Slenderness ratio — $\lambda = L_E / r_{yy} =$ **188.119**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — $u =$ **0.848**

Torsional index — $x =$ **13.334**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.550**

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

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **87.710**

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 — $\alpha_{LT} =$ **7.0**

Perry factor — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.374**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **263** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **318.2** N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **148.2** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment — $M_2 =$ **26.5** kNm

Moment at centre-line of segment — $M_3 =$ **35.4** kNm

Moment at three quarter point of segment — $M_4 =$ **26.5** kNm

Maximum moment in segment — $M_{abs} =$ **35.4** kNm

Maximum moment governing buckling resistance — $M_{LT} = M_{abs} =$ **35.4** kNm

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx} =$ **45.7** kNm

$M_b / m_{LT} =$ **49.5** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 = \mathbf{23.2}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{17.763}$ mm

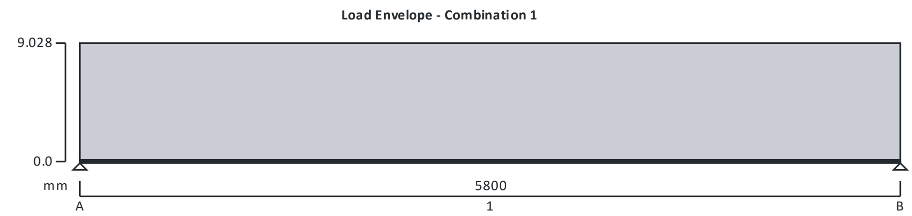
PASS - Maximum deflection does not exceed deflection limit

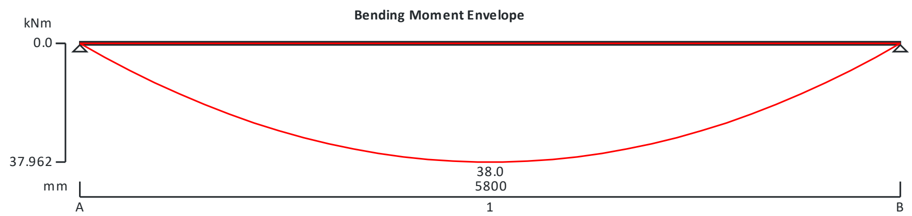
SWM STRUCTURES Suite 101 C1 Peel House, Morden, SM4 5BT stan@swmstructures.co.uk	Project 17 Belmont Park, London, SE13 5BJ				Job no. 1665	
	Calcs for STEEL BEAM 2/2				Start page no./Revision 20	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

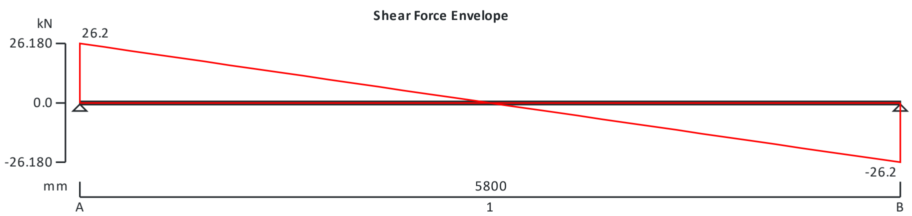
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead full UDL 1.4 kN/m Imposed full UDL 4.1 kN/m Dead self weight of beam × 1

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

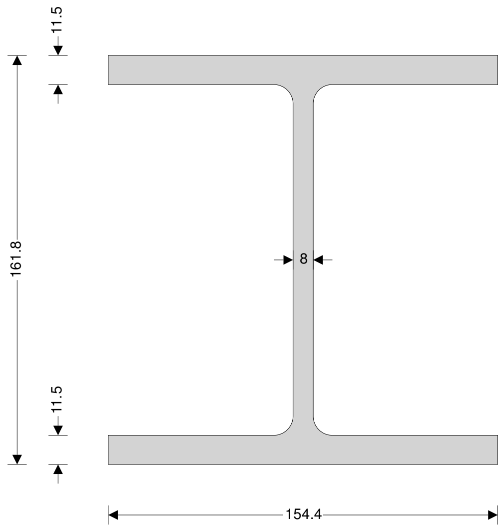
Maximum moment	M_{max} = **38** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **26.2** kN	V_{min} = **-26.2** kN
Deflection	δ_{max} = **19.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **26.2** kN	R_{A_min} = **26.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **5.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **11.9** kN	
Maximum reaction at support B	R_{B_max} = **26.2** kN	R_{B_min} = **26.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **5.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **11.9** kN	

Section details

Section type	**UC 152x152x37 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.5** 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	$K_{LT.A}$ = **1.20** $+ 2 \times$ D
	$K_{LT.B}$ = **1.20** $+ 2 \times$ D

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 = 15.5 $\times \varepsilon$ <= 80 $\times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section — $b = B / 2 =$ **77.2** mm

$b / T = 6.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **26.2** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D =$ **1294** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v =$ **213.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **38** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **84.9** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.2 \times L_{s1} + 2 \times D =$ **7284** mm

Slenderness ratio — $\lambda = L_E / r_{yy} =$ **188.119**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — $u =$ **0.848**

Torsional index — $x =$ **13.334**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.550**

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

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **87.710**

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 — $\alpha_{LT} =$ **7.0**

Perry factor — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.374**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **263** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **318.2** N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **148.2** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment — $M_2 =$ **28.5** kNm

Moment at centre-line of segment — $M_3 =$ **38** kNm

Moment at three quarter point of segment — $M_4 =$ **28.5** kNm

Maximum moment in segment — $M_{abs} =$ **38** kNm

Maximum moment governing buckling resistance — $M_{LT} = M_{abs} =$ **38** kNm

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx} =$ **45.7** kNm

$M_b / m_{LT} =$ **49.5** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **23.2** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **19.064** mm

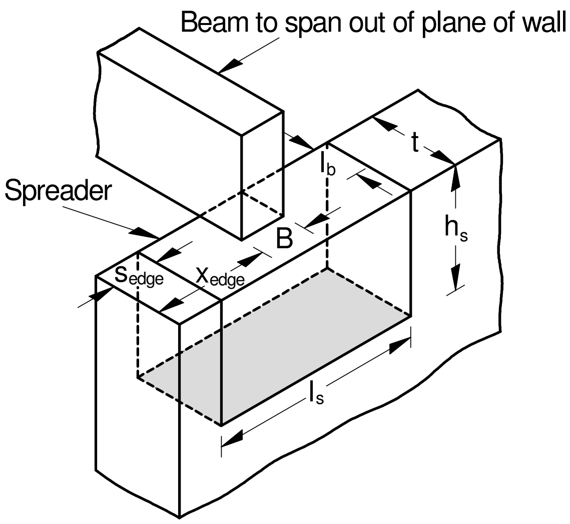
PASS - Maximum deflection does not exceed deflection limit

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.08

Masonry details

Masonry type	**Clay or calcium silicate bricks**		
Compressive strength	p_{unit} = **5.0** N/mm^2	Mortar designation	**iv**
Masonry units	**Category II**	Construction control	**Normal**
Partial safety factor	γ_m = **3.5**	Characteristic strength	f_k = **2.2** N/mm^2
Leaf thickness	t = **112** mm	Effective wall thickness	t_{ef} = **225** mm
Wall height	h = **2600** mm	Effective height of wall	h_{ef} = **2600** mm



Bearing details

Beam spanning out of plane of wall

Width of bearing	B = **152** mm	Length of bearing	l_b = **100** mm
Edge distance	x_{edge} = **2000** mm		

Loading details

Concentrated dead load	G_k = **5** kN	Concentrated imposed load	Q_k = **12** kN
Design concentrated load	F = **26.5** kN		
Distributed dead load	g_k = **0.0** kN/m	Distributed imposed load	q_k = **0.0** kN/m
Design distributed load	f = **0.0** kN/m		

Masonry bearing type

Bearing type	**Type 2**	Bearing safety factor	γ_{bear} = **1.50**

Check design bearing without a spreader

Design bearing stress	f_{ca} = **1.742** N/mm^2	Allowable bearing stress	f_{cp} = **0.943** N/mm^2

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader	l_s = **450** mm	Depth of spreader	h_s = **150** mm
Edge distance	s_{edge} = **1851** mm		

Suite 101 C1
Peel House,
Morden,
SM4 5BT
stan@swmstructures.co.uk

Spreader bearing type

Bearing type **Type 1** Bearing safety factor γ_{bear} = **1.25**

Check design bearing with a spreader

Loading acts at midpoint of spreader

Design bearing stress f_{ca} = **0.525** N/mm^2 Allowable bearing stress f_{cp} = **0.786** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Design bearing stress f_{ca} = **0.106** N/mm^2 Allowable bearing stress f_{cp} = **0.412** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

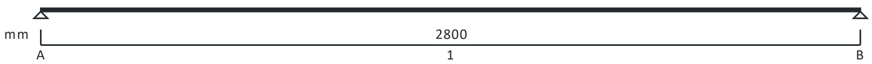
SWM STRUCTURES Suite 101 C1 Peel House, Morden, SM4 5BT stan@swmstructures.co.uk	Project 17 Belmont Park, London, SE13 5BJ				Job no. 1665	
	Calcs for TIMBER FLOOR JOISTS				Start page no./Revision 26	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

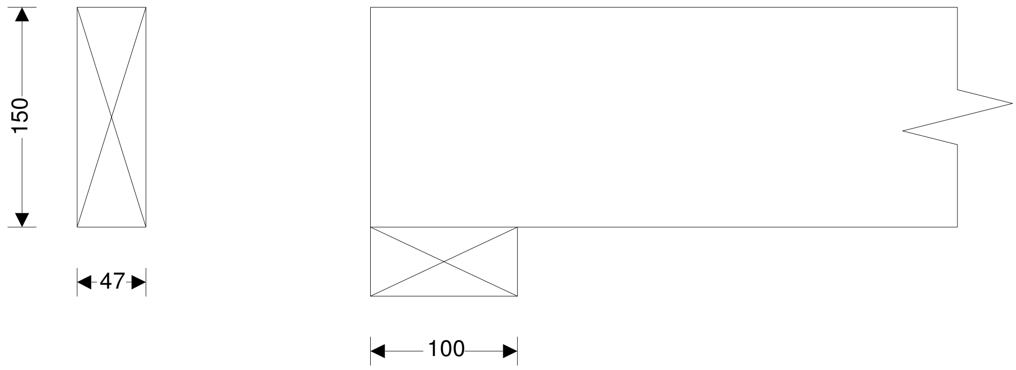
Joist details

Joist breadth	$b = \mathbf{47}$ mm
Joist depth	$h = \mathbf{150}$ mm
Joist spacing	$s = \mathbf{400}$ 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	$L_{s1} = \mathbf{2800}$ mm



Section properties

Second moment of area	$I = b \times h^3 / 12 = \mathbf{13218750}$ mm^4
Section modulus	$Z = b \times h^2 / 6 = \mathbf{176250}$ mm^3

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.02}$ kN/m
Dead load	$F_{d\_udl} = \mathbf{0.50}$ 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	$K_7 = \mathbf{1.08}$

Load sharing factor K_8 = **1.10**

Consider long term loads

Load duration factor K_3 = **1.00**
Maximum bending moment M = **0.805** kNm
Maximum shear force V = **1.150** kN
Maximum support reaction R = **1.150** kN
Maximum deflection δ = **5.901** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm^2
Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **6.292** N/mm^2
Applied bending stress σ_{m_max} = M / Z = **4.567** 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 $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.245** 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 $\sigma_{c\_max} = R / (b \times L_b)$ = **0.245** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **8.400** mm
Bending deflection (based on E_{mean}) $\delta_{bending}$ = **5.652** mm
Shear deflection δ_{shear} = **0.249** mm
Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **5.901** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor K_3 = **1.25**
Maximum bending moment M = **1.197** kNm
Maximum shear force V = **1.710** kN
Maximum support reaction R = **1.710** kN
Maximum deflection δ = **7.398** 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 = **6.792** 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 $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.364** 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 — $\sigma_{c\_max} = R / (b \times L_b)$ = **0.364** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection — $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **8.400** mm

Bending deflection (based on E_{mean}) — $\delta_{bending}$ = **7.028** mm

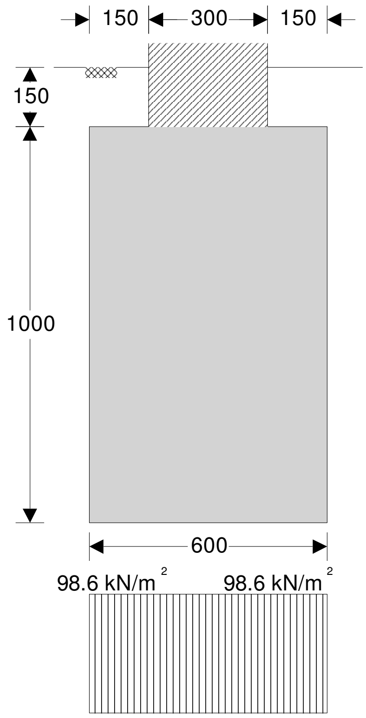
Shear deflection — δ_{shear} = **0.370** mm

Total deflection — $\delta = \delta_{bending} + \delta_{shear}$ = **7.398** mm

PASS - Actual deflection within permissible limits

STRIP FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Strip footing details

Width of strip footing	B = **600** mm	Depth of strip footing	h = **1000** mm
Depth of soil over strip footing	h_{soil} = **150** mm	Density of concrete	ρ_{conc} = **23.6** kN/m^3

Load details

Load width	b = **300** mm	Load eccentricity	e_P = **0** mm

Soil details

Depth of soil over pad footing	h_{soil} = **150** mm	Density of soil	ρ_{soil} = **20.0** kN/m^3
Allowable bearing pressure	$P_{bearing}$ = **100** kN/m^2		

Axial loading on strip footing

Dead axial load	P_G = **39.1** kN/m	Imposed axial load	P_Q = **4.1** kN/m
Wind axial load	P_W = **0.0** kN/m	Total axial load	P = **43.2** kN/m

Foundation loads

Dead surcharge load	F_{Gsur} = **0.000** kN/m^2	Imposed surcharge load	F_{Qsur} = **0.000** kN/m^2
Strip footing self weight	F_{swt} = **23.600** kN/m^2	Soil self weight	F_{soil} = **3.000** kN/m^2
Total foundation load	F = **16.0** kN/m		

Calculate base reaction

Total base reaction	T = **59.2** kN/m	Eccentricity of base reaction	e_T = **0** mm

Base reaction acts within middle third of base

Calculate pad base pressures

Base pressures	q_1 = **98.600** kN/m^2	q_2 = **98.600** kN/m^2	
Minimum base pressure	q_{min} = **98.600** kN/m^2	Maximum base pressure	q_{max} = **98.600** kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

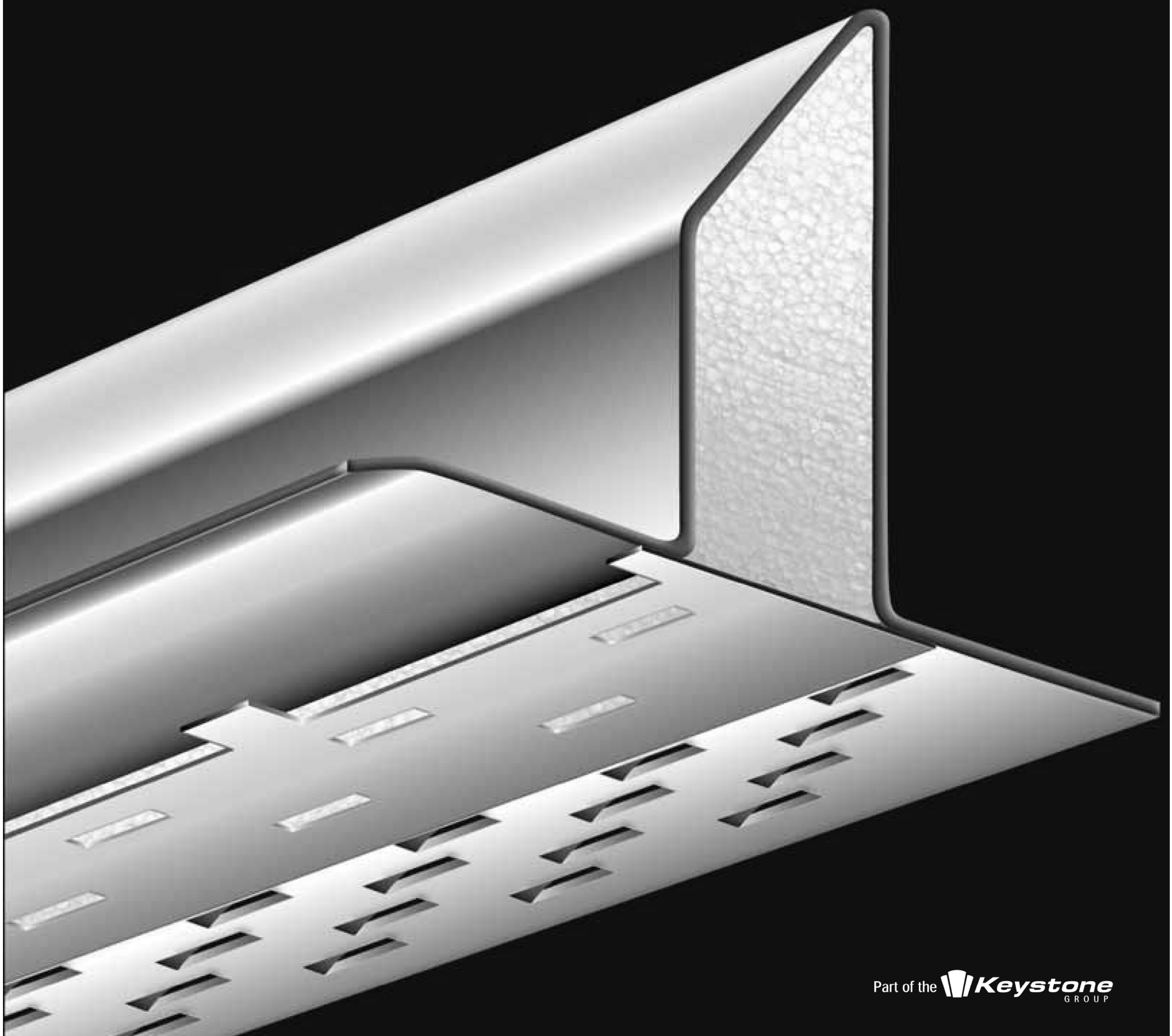
NEW IMPROVED PROFILE

CI/SfB (31.9) Xh2

January 2007

Better by design

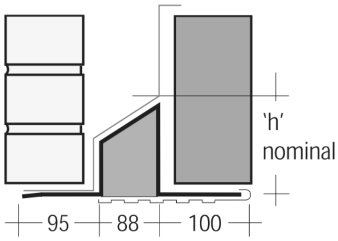
STEEL LINTELS



Part of the Keystone Group

L1/S 100

For 95-110mm cavity wall construction.
Standard duty loading condition.

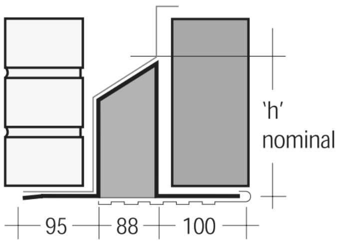


* A continuous bottom plate added   Note: Maximum block dimensions 125mm (95mm cavity)

Manufactured Length 150mm increments	0600 1200	1350 1500	1650 1800	1950 2100	2250 2400	2550 2700	2850 3000	3150 3600	3750 4050	*4200 4800
Height 'h'	88	88	107	125	150	162	171	200	200	200
Thickness 't'	1.6	2	2	2	2	2.6	2.6	3.2	3.2	3.2
Total UDL(kN) Load ratio (1)	12	16	19	21	23	27	27	27	26	27
Total UDL(kN) Load ratio (2)	10	13	16	17	18	22	20	20	19	22

L1/HD 100

For 95-110mm cavity wall construction.
Heavy duty loading condition.

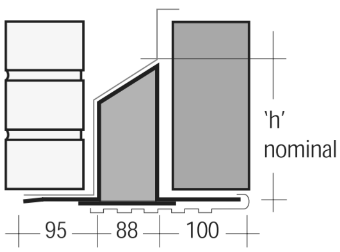


* A continuous bottom plate added   Note: Maximum block dimensions 125mm (95mm cavity)

Manufactured Length 150mm increments	0600 1200	*1350 1500	*1650 2100	*2250 2550	*2700 3000	*3150 3600	*3750 4200			
Height 'h'	110	135	163	203	203	203	203			
Thickness 't'	3.2	3.2	3.2	3.2	3.2	3.2	3.2			
Total UDL(kN) Load ratio (1)	30	30	40	40	40	35	33			
Total UDL(kN) Load ratio (2)	22	22	35	35	35	32	28			

L1/XHD 100

For 95-110mm cavity wall construction.
Extra heavy duty loading condition.



* A continuous bottom plate added   Note: Maximum block dimensions 125mm (95mm cavity)

Manufactured Length 150mm increments	*0600 1500	*1650 1800	*1950 2100							
Height 'h'	163	163	203							
Thickness 't'	3.2	3.2	3.2							
Total UDL(kN) Load ratio (1)	50	50	55							
Total UDL(kN) Load ratio (2)	45	45	45							

Load Ratio - see Structural Performance on page 33.