Design Calculations

for

Internal Alterations

to

The Quest, West Street,
Harrietsham,
Maidstone.

for

Jessica Arnold & James Whitfield

Calculations Contents

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Job No. **18629**

Rev:

Date: Sep-23

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SECTION A - INFORMATION

1.0 Basic Information

1.1 Jessica Arnold & James Whitfield — Client

1.2 James Clague Architects — Architect

2.0 Codes of Practice — Relevant Design Codes and Building Regs.

2.1 BS6399 : Part 1 : 1984 - Code of practice for dead and imposed loads

2.2 BS6399 : Part 2 : 1995 - Code of practice for wind loads

2.3 BS5268 : Part 2 : 1995 - Structural use of timber

2.4 BS5950 : Part 1 : 1990 - Structural use of steelwork in building

2.5 BS5628 : Part 1 : 1992 - Structural use of unreinforced masonry

2.6 BS8110 : Part 1 : 1997 - Structural use of concrete

3.0 Project Description

3.1 The project comprises alterations to an existing building. The calculation and design package is concerned only with the proposed internal alterrations to the the property.

Intended use of element(s), including future design reqs.

4.0 Fire Protection

By Architect

Fire resistance reqs.

5.0 Vertical Loads

Floor imposed loads are 1.50 kN/m^2 for residential loads. Roof imposed loads are 0.75 kN/m^2. All dead loads to Architects specification.

General loading

6.0 Horizontal Loads

Wind loading as defined in BS6399.

Wind loading

7.0 Geology

From geological survey maps the underlying sub-strate is believed to consist of Folkestone Formation.

Bearing strata

8.0 Foundations

N/A

Foundation type

9.0 Structural Form & Stability

9.1 The existing building is of traditional masonry construction, which has an inherent robustness and stability.

Structural form & stability statement

10.0 Materials — Material Data

10.1	Concrete	RC40
10.2	Masonry	7 N block work in grade (iii) mortar.
10.3	Steel	Grade S355.
10.4	Timber	Timber to be grade C16 or C24 SW marked either 'DRY' or 'KD'.
10.5	Other	Not applicable

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SECTION B : VERTICAL LOADING

1.0	**Pitched Roof**		kg/m^2			pitch, θ = 40.0 °
1.1	Dead Loads	Tiles	65.00			
		Battens	2.00			
		Felt	3.00			
		Boards	10.00			
		Rafters	10.00			
		Insulation	5.00			
		Plasterboard	12.00			
		Skim	6.00			
			113.00	/ cos θ°		
		Plan load	147.51			
			147.51	x 1.4 (γ_f) =	**206.52**	
1.2	Imposed Loads	Roof	**50.00**	x 1.6 (γ_f) =	**80.00**	
1.3	Imposed Loads	Ceiling	**0.00**	x 1.6 (γ_f) =	**0.00**	
		TOTAL	**197.51**		**286.52**	

1.4	Dead Load =	**1.48 kN/m^2** (service) AND =	**2.07 kN/m^2** (ultimate)	γ_f = 1.45
1.5	Imposed Load =	**0.50 kN/m^2** (service) AND =	**0.80 kN/m^2** (ultimate)	Ser = 1.98 kN/m^2
1.6	Total Load =	**1.98 kN/m^2** (service) AND =	**2.87 kN/m^2** (ultimate)	Ult = 2.87 kN/m^2

2.0	**Timber Floor**		kg/m^2		
2.1	Dead Loads	Boards	15.00		
		Joists	13.00		
		Plasterboard	12.00		
		Skim	6.00		
			46.00	x 1.4 (γ_f) =	**64.40**
2.2	Imposed Loads	IL	**150.00**	x 1.6 (γ_f) =	**240.00**
		TOTAL	**196.00**		**304.40**

2.3	Dead Load =	**0.46 kN/m^2** (service) AND =	**0.64 kN/m^2** (ultimate)	γ_f = 1.55
2.4	Imposed Load =	**1.50 kN/m^2** (service) AND =	**2.40 kN/m^2** (ultimate)	Ser = 1.96 kN/m^2
2.5	Total Load =	**1.96 kN/m^2** (service) AND =	**3.04 kN/m^2** (ultimate)	Ult = 3.04 kN/m^2

3.0	**Internal Wall** - Lime Plaster		kg/m^2		
3.1	Dead Loads	Lime Plaster	60.00		
		Brick (102)	195.00		
		TOTAL	**255.00**	x 1.4 (γf) =	**357.00**

3.2	Dead Load =	**2.55 kN/m^2** (service) AND =	**3.57 kN/m^2** (ultimate)	γ_f = 1.40
3.3	Imposed Load =	**0.00 kN/m^2** (service) AND =	**0.00 kN/m^2** (ultimate)	Ser = 2.55 kN/m^2
3.4	Total Load =	**2.55 kN/m^2** (service) AND =	**3.57 kN/m^2** (ultimate)	Ult = 3.57 kN/m^2

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SECTION D : SUPER STRUCTURE DESIGN

1.0 Timber Joist Design — **Timber to BS5268 : Part 2**

1F Floor Joists - Bedroom floor - 1600 mm clear span — Effective joist Length, l_b = **1.70 m**

1.1 Loading — Centres, **C** = **400 mm**

	Dead Load	Imposed Load
UDL - over entire length		
Floor = 1 x 1.96 kN/m^2 x C =	0.18 kN/m	0.60 kN/m
Vertical UDL, **w** =	**0.18 kN/m (DL)** &	**0.60 kN/m (IL)**
Total UDL, **w** =	**1.22 kN/m (U)** &	**0.78 kN/m (S)**

1.2 Analysis

Shear, F_v = wl/2 =	**0.67 kN**	F_v (U) =	1.03 kN
Moment, **M** = wl^2/8 =	**0.28 kNm**		
Deflection, δ = 5wl^4/384EI =	**85 Nm^3 (/EI)**		

1.3 Design — Try section: **1 No. 100x50 C16 timber at 400 mm centres**

Grade	C16	K_3 =	1.00	K_{15} =	1.00	E_{mean} =	8800 N/mm^2
Ply =	1	K_7 =	1.17	K_{19} =	1.00	σ_{mall} =	5.30 N/mm^2
b =	100 mm	K_8 =	1.10	K_{20} =	1.00	r_g =	0.67 N/mm^2
d =	50 mm						

Deflection		
Allowable deflection, δ = 0.003l =	**5.1 mm**	
Actual deflection, $\delta_t = \delta_a + (19.2M/EK_{20}A)$ =	**9.4 mm**	fails
Bending		
Allowable bending, $\sigma_{mall} = \sigma_{mall}k_3k_7k_8k_{15}$ =	**6.82 N/mm^2**	
Actual bending, σ_{macll} = M/Z =	**6.80 N/mm^2**	okay
Shear		
Allowable shear, $r_g = r_g k_8 k_{19}$ =	**0.74 N/mm^2**	
Actual Shear, r_a = 1.5F/A =	**0.20 N/mm^2**	okay

1.4 Joists satisfactory as: **fails** — **Ex. TJ**

2.0

Note:
In the absence of structural information, the grade of timber is conservatively taken as C16. The floor-imposed load is taken as 1.50 kN/m2 as per BS6399 for residential properties. The existing joist seems to have adequate bending and shear capacity. However, it is failing due to excessive deflection. Therefore, we would recommend that you added a 15 mm x 11 ply Finnish birch plywood 1.4 mm veneer, to the top of the floor as this will make it act as a stress skin. BSF would also recommend that the Joists are to be doubled up below bath locations, or spread the bath loading over minimum three number of joists using a timber spreader.

3.0 For the Floor Analysis & Design: See Appendix A1.1

4.0 Timber Beam Design — **Timber to BS5268 : Part 2**

1F Floor Beam - Existing - support floor joist - 3600 mm clear span

4.1 Loading — Effective Beam Length, l_b = **3.70 m**

	Dead Load	Imposed Load
UDL - over entire length		
Floor = 1 x 1.96 kN/m2 x 3.5 m x 50% =	0.81 kN/m	2.63 kN/m
Self Weight = ρ_{mean} x b x d x ply =	0.17 kN/m	0.00 kN/m
Vertical UDL, **w** =	**0.98 kN/m (DL)** &	**2.63 kN/m (IL)**
Total UDL, **w** =	**5.57 kN/m (U)** &	**3.60 kN/m (S)**

4.2 Analysis

Shear, F_v = wl/2 =	**6.67 kN**	F_v (U) =	10.31 kN
Moment, **M** = wl^2/8 =	**6.17 kNm**		
Deflection, δ = 5wl^4/384EI =	**8795 Nm^3 (/EI)**		

4.3 **Design** Try section: **1 No. 185x150 D30 timber**

Grade	D30	K_3 =	1.00	K_{15} =	1.00	ρ_{mean} =	640 kg/m^3
Ply =	1	K_7 =	1.08	K_{19} =	1.00	E_{min} =	6000 N/mm^2
		K_8 =	1.00	K_{20} =	1.00		
b =	185 mm	K_9 =	1.00			σ_{mall} =	9.00 N/mm^2
d =	150 mm					r_g =	1.40 N/mm^2

Deflection
Allowable deflection, $\boldsymbol{\delta}$ = 0.003l = **11.1 mm**
Actual deflection, $\boldsymbol{\delta_t} = \delta_a + (19.2M/Ek_9K_{20}A)$ = **28.9 mm** — fails

Bending
Allowable bending, $\sigma_{\mathbf{mall}} = \sigma_{mall}k_3k_7k_8k_{15}$ = **9.71 N/mm^2**
Actual bending, $\sigma_{\mathbf{macll}}$ = M/Z = **8.89 N/mm^2** — okay

Shear
Allowable shear, $\mathbf{r_g} = r_gk_8k_{19}$ = **1.40 N/mm^2**
Actual Shear, $\mathbf{r_a}$ = 1.5F/A = **0.36 N/mm^2** — okay

4.4 Section satisfactory as: fails **Ex. TB**

5.0

Note:
In the absence of structural information, the grade of the timber beam has been conservatively taken as D30. The floor's imposed load is taken as 1.50 kN/m2 as per BS6399 for residential properties. The existing timber beam is failing due to excessive deflection. Furthermore, during our visit, we noticed that the beams condition has deteriorated over the years and repairs have been carried out in several positions along the beams. Therefore, BSF would recommend that you strengthen the floor beam with a single steel T section as per BSF drawings.

6.0 **For the Steel Beam Analysis & Design: See Appendix A1.2**

7.0 **Timber Beam Design** **Timber to BS5268 : Part 2**
Existing Wall Beam - Lintel - 1525 mm clear span

7.1 **Loading** Effective length, **l** = **1.68 m**

UDL - over entire length	Dead Load	Imposed Load
Floor = 1 x 1.96 kN/m2 x 1.7 m x 50% =	0.39 kN/m	1.28 kN/m
Wall = 1 x 2.55 kN/m2 x 0.9 m x 100% =	2.30 kN/m	0.00 kN/m
Self Weight = ρ_{mean} x b x d x ply =	0.09 kN/m	0.00 kN/m
Vertical UDL, **w** =	**2.77 kN/m (DL)** &	**1.28 kN/m (IL)**
Total UDL, **w** =	**5.92 kN/m (U)** &	**4.05 kN/m (S)**

7.2 **Analysis**
Shear, $\mathbf{F_v}$ = wl/2 = **3.40 kN** F_v (U) = 4.97 kN
Moment, **M** = $wl^2/8$ = **1.42 kNm**
Deflection, $\boldsymbol{\delta} = 5wl^4/384EI$ = **417 Nm^3 (/EI)**

7.3 **Design** Try section: **1 No. 140x100 D30 timber**

Grade	D30	K_3 =	1.00	K_{15} =	1.00	ρ_{mean} =	640 kg/m^3
Ply =	1	K_7 =	1.13	K_{19} =	1.00	E_{min} =	6000 N/mm^2
		K_8 =	1.00	K_{20} =	1.00		
b =	140 mm	K_9 =	1.00			σ_{mall} =	9.00 N/mm^2
d =	100 mm					r_g =	1.40 N/mm^2

Deflection
Allowable deflection, $\boldsymbol{\delta}$ = 0.003l = **5.0 mm**
Actual deflection, $\boldsymbol{\delta_t} = \delta_a + (19.2M/Ek_9K_{20}A)$ = **6.3 mm** — fails

Bending
Allowable bending, $\sigma_{\mathbf{mall}} = \sigma_{mall}k_3k_7k_8k_{15}$ = **10.16 N/mm^2**
Actual bending, $\sigma_{\mathbf{macll}}$ = M/Z = **6.10 N/mm^2** — okay

Shear
Allowable shear, $r_g = r_g k_8 k_{19}$ = **1.40 N/mm²**
Actual Shear, r_a = 1.5F/A = **0.36 N/mm²** okay

7.4 Section satisfactory as: **fails** **Ex. TLB**

8.0 Timber Beam Design **Timber to BS5268 : Part 2**
Existing Wall Beam - Lintel - 1400 mm clear span

8.1 **Loading** Effective length, **l = 1.54 m**

UDL - over entire length	Dead Load	Imposed Load
Floor = 1 x 1.96 kN/m2 x 1.7 m x 50% =	0.39 kN/m	1.28 kN/m
Wall = 1 x 2.55 kN/m2 x 0.8 m x 100% =	2.04 kN/m	0.00 kN/m
Self Weight = ρ_{mean} x b x d x ply =	0.09 kN/m	0.00 kN/m
Vertical UDL, **w** =	**2.52 kN/m (DL)** &	**1.28 kN/m (IL)**
Total UDL, **w** =	**5.57 kN/m (U)** &	**3.79 kN/m (S)**

8.2 **Analysis**
Shear, F_v = wl/2 = **2.92 kN** F_v (U) = 4.29 kN
Moment, **M** = $wl^2/8$ = **1.12 kNm**
Deflection, $\delta = 5wl^4/384EI$ = **278 Nm³ (/EI)**

8.3 **Design** Try section: **1 No. 140x100 D30 timber**

Grade	D30	K_3 =	1.00	K_{15} =	1.00	ρ_{mean} =	640 kg/m³
Ply =	1	K_7 =	1.13	K_{19} =	1.00	E_{min} =	6000 N/mm²
		K_8 =	1.00	K_{20} =	1.00		
b =	140 mm	K_9 =	1.00			σ_{mall} =	9.00 N/mm²
d =	100 mm					r_g =	1.40 N/mm²

Deflection
Allowable deflection, δ = 0.003l = **4.6 mm**
Actual deflection, $\delta_t = \delta_a + (19.2M/Ek_9K_{20}A)$ = **4.2 mm** okay
Bending
Allowable bending, $\sigma_{mall} = \sigma_{mall} k_3 k_7 k_8 k_{15}$ = **10.16 N/mm²**
Actual bending, σ_{macll} = M/Z = **4.82 N/mm²** okay
Shear
Allowable shear, $r_g = r_g k_8 k_{19}$ = **1.40 N/mm²**
Actual Shear, r_a = 1.5F/A = **0.31 N/mm²** okay

8.4 Section satisfactory as: **1 No. 140x100 D30 timber** **Ex. TLB**

9.0
Note:
In the absence of structural information, the grade of the timber beam has been conservatively taken as D30. The existing timber beam does not have adequate capacity to act as a lintel for the proposed opening of 1.525m clear span. However, the existing beam have adequate capacity to support an clear span of 1.4m.
Please note that we have not allowed any roof loadings. The beam seems to be supporting a timber post. The extent of this post is unknown and hence no allowance was made in our calculation. BSF would recommend that this to be opened up prior to commencement of work to determine the extent of the post and finding to be reported back to the BSF.

10.0 Timber Joist Design **Timber to BS5268 : Part 2**
New Floor Joists - new bathroom floor - 2300 mm clear span Effective length, **l = 2.40 m**
10.1 **Loading** Centres, **C = 400 mm**

UDL - over entire length	Dead Load	Imposed Load
Floor = 1 x 1.96 kN/m² x C =	0.18 kN/m	0.60 kN/m
Vertical UDL, **w** =	**0.18 kN/m (DL)** &	**0.60 kN/m (IL)**
Total UDL, **w** =	**1.22 kN/m (U)** &	**0.78 kN/m (S)**

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10.2 **Analysis**

Shear, $\mathbf{F_v}$ = wl/2 =	**0.94 kN**	F_v (U) =	1.46 kN
Moment, **M** = $wl^2/8$ =	**0.56 kNm**		
Deflection, $\boldsymbol{\delta} = 5wl^4/384EI$ =	**339 Nm^3 (/EI)**		

10.3 **Design** Try section: **1 No. 47x150 C16 timber at 400 mm centres**

Grade	C16	K_3 =	1.00	K_{15} =	1.00	E_{mean} =	8800 N/mm^2
Ply =	1	K_7 =	1.08	K_{19} =	1.00	σ_{mall} =	5.30 N/mm^2
b =	47 mm	K_8 =	1.10	K_{20} =	1.00	r_g =	0.67 N/mm^2
d =	150 mm						

Deflection

Allowable deflection, $\boldsymbol{\delta}$ = 0.003l =	**7.2 mm**	
Actual deflection, $\boldsymbol{\delta_t} = \delta_a + (19.2M/EK_{20}A)$ =	**3.1 mm**	okay

Bending

Allowable bending, $\boldsymbol{\sigma_{mall}} = \sigma_{mall}k_3k_7k_8k_{15}$ =	**6.29 N/mm^2**	
Actual bending, $\boldsymbol{\sigma_{macll}}$ = M/Z =	**3.20 N/mm^2**	okay

Shear

Allowable shear, $\mathbf{r_g} = r_gk_8k_{19}$ =	**0.74 N/mm^2**	
Actual Shear, $\mathbf{r_a}$ = 1.5F/A =	**0.20 N/mm^2**	okay

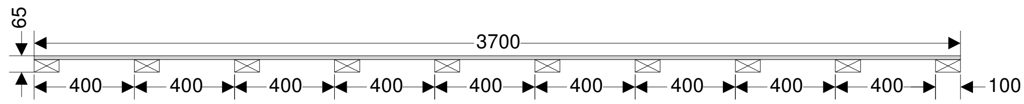
10.4 Joists satisfactory as: **1 No. 47x150 C16 timber at 400 mm centres** **FJ**

STRESS SKIN PANEL DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03

Single-skin panel details

Effective span of panel	L_{ef} = **1700** mm
Panel width	b_{panel} = **3700** mm
Web member depth	h = **50** mm
Web member breadth	b = **100** mm
Web member spacing	s = **400** mm
Number of web members per panel	N = **10**
Timber strength class	**C16**
Top skin	15 mm x 11 ply Finnish birch plywood 1.4 mm veneer:Sanded
Minimum thickness	t_{mins_top} = **14.3** mm
Panel depth	$t_{panel} = t_{mins\_top} + h$ = **64** mm



Section properties

Top skin partial flange width	$b_{se\_top} = \min(25 \times t_{mins\_top}, 0.1 \times L_{ef}, s - b)$ = **170** mm
Top skin effective width	$b_{s\_top} = b_{panel} - (N - 1) \times (s - b - b_{se\_top})$ = **2530** mm
Top skin area	$A_{s\_top} = b_{s\_top} \times t_{mins\_top}$ = **36179** mm^2
Product of EA for top skin	$EA_{top} = E_{t\_pars\_top} \times A_{s\_top}$ = **151951.800** kN
Distance from centroid to top surface	$y_{s\_top} = (t_{mins\_top} / 2)$ = **7.2** mm
Web member area	$A = N \times b \times h$ = **50000** mm^2
Product of EA for web member	$EA = E_{mean} \times A$ = **440000.000** kN
Distance from centroid to top surface	$y = (t_{mins\_top} + h / 2)$ = **39.3** mm
Summation of product EA for panel	$\Sigma EA = EA_{top} + EA$ = **591952** kN
Summation of product EAy for panel	$\Sigma EAy = EA_{top} \times y_{s\_top} + EA \times y$ = **18378** kNm
Neutral axis depth	$\overline{y} = \Sigma EAy / \Sigma EA$ = **31.0** mm
Distance from NA to centroid of top skin	$h_{xs\_top} = \text{abs}(y_{s\_top} - \overline{y})$ = **23.897** mm
Distance from NA to centroid of web members	$h_x = \text{abs}(y - \overline{y})$ = **8.253** mm
Bending rigidity of panel	$EI = EA \times h^2 / 12 + EA_{top} \times h_{xs\_top}^2 + EA \times h_x^2$ = **208** kN/m^{-2}

Loading details

Panel self weight	$F_{swt} = (N \times b \times h \times \rho + s_{wts\_top} \times b_{panel}) \times g_{acc}$ = **0.559** kN/m
Dead load	F_{d_udl} = **0.50** kN/m^2
Imposed UDL	F_{i_udl} = **1.50** kN/m^2
Imposed point load	F_{i_pt} = **1.40** kN

Modification factors

Section depth factor	K_7 = **1.17**
Load sharing factor	K_8 = **1.10**
Stress concentration modification factor	K_{37} = **0.50**
Nail glue modification factor	K_{70} = **0.90**

Consider long term loads

Load duration factor for timber	K_3 = **1.00**
Load duration factor for plywood	K_{36} = **1.00**

Total UDL $W = F_{swt} + (F_{d\_udl} + F_{i\_udl}) \times b_{panel} = \mathbf{7.959}$ kN/m

Check bending stresses

Permissible compressive stress in top skin $\sigma_{ms\_top\_adm} = \sigma_{c\_pars\_top} \times K_{36} = \mathbf{9.700}$ N/mm^2

Maximum bending moment $M = W * L_{ef}^2 / 8 = \mathbf{2.875}$ kNm

Compressive stress at extreme fibre of top skin $\sigma_{ms\_top} = M \times \bar{y} \times E_{t\_pars\_top} / EI = \mathbf{1.799}$ N/mm^2

PASS - Compressive stress at extreme fibre of top skin is less than permissible stress

Permissible bending stress in web $\sigma_{mw\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{6.821}$ N/mm^2

Bending stress at upper extreme fibre of web $\sigma_{mw\_top} = M \times (\bar{y} - t_{mins\_top}) \times E_{mean} / EI = \mathbf{2.033}$ N/mm^2

PASS - Bending stress at upper extreme fibre of web is less than permissible stress

Bending stress at lower extreme fibre of web $\sigma_{mw\_bot} = M \times (t_{panel} - \bar{y}) \times E_{mean} / EI = \mathbf{4.037}$ N/mm^2

PASS - Bending stress at lower extreme fibre of web is less than permissible stress

Check horizontal shear stresses in web members

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

Maximum shear force $V = W * L_{ef} / 2 = \mathbf{6.765}$ kN

Product of moment of elasticity and first moment of area about neutral axis

$E_s = EA_{top} \times h_{xs\_top} + N \times b \times (\bar{y} - t_{s\_top})^2 \times E_{mean} / 2 = \mathbf{4764}$ kNm

Maximum horizontal shear stress $\tau_{max} = V \times E_s / (EI \times N \times b) = \mathbf{0.155}$ N/mm^2

PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between top skin and web members

Web contact length $b_{con} = N * b = \mathbf{1000}$ mm

Permissible rolling shear stress at top skin $\tau_{r\_top\_adm} = \tau_{r\_backs\_top} \times K_{36} \times K_{37} \times K_{70} = \mathbf{0.554}$ N/mm^2

Maximum rolling shear stress $\tau_{r\_top\_max} = V \times EA_{top} \times h_{xs\_top} / (EI \times b_{con}) = \mathbf{0.118}$ N/mm^2

PASS - Maximum rolling shear stress at top skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{ef} = \mathbf{5.100}$ mm

Bending deflection $\delta_{bending} = 5 * W * L_{ef}^4 / (384 * EI) = \mathbf{4.153}$ mm

Shear deflection $\delta_{shear} = 12 * W * L_{ef}^2 / (5 * \Sigma EA) = \mathbf{0.093}$ mm

Total deflection $\delta_{max} = \delta_{bending} + \delta_{shear} = \mathbf{4.246}$ mm

PASS - Total deflection is less than permissible deflection

Consider medium term loads

Load duration factor for timber $K_3 = \mathbf{1.25}$

Load duration factor for plywood $K_{36} = \mathbf{1.33}$

Total UDL $W = F_{swt} + F_{d\_udl} \times b_{panel} = \mathbf{2.409}$ kN/m

Total point load $P = F_{i\_pt} = \mathbf{1.400}$ kN

Check bending stresses

Permissible compressive stress in top skin $\sigma_{ms\_top\_adm} = \sigma_{c\_pars\_top} \times K_{36} = \mathbf{12.901}$ N/mm^2

Maximum bending moment $M = W * L_{ef}^2 / 8 + P * L_{ef} / 4 = \mathbf{1.465}$ kNm

Compressive stress at extreme fibre of top skin $\sigma_{ms\_top} = M \times \bar{y} \times E_{t\_pars\_top} / EI = \mathbf{0.917}$ N/mm^2

PASS - Compressive stress at extreme fibre of top skin is less than permissible stress

Permissible bending stress in web $\sigma_{mw\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.526}$ N/mm^2

Bending stress at upper extreme fibre of web $\sigma_{mw\_top} = M \times (\bar{y} - t_{mins\_top}) \times E_{mean} / EI = \mathbf{1.036}$ N/mm^2

PASS - Bending stress at upper extreme fibre of web is less than permissible stress

Bending stress at lower extreme fibre of web $\sigma_{mw\_bot} = M \times (t_{panel} - \bar{y}) \times E_{mean} / EI = \mathbf{2.057}$ N/mm^2

PASS - Bending stress at lower extreme fibre of web is less than permissible stress

Check horizontal shear stresses in web members

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

Maximum shear force $V = W * L_{ef} / 2 + P / 2 =$ **2.747** kN

Product of moment of elasticity and first moment of area about neutral axis

$E_s = EA_{top} \times h_{xs\_top} + N \times b \times (\bar{y} - t_{s\_top})^2 \times E_{mean} / 2 =$ **4764** kNm

Maximum horizontal shear stress $\tau_{max} = V \times E_s / (EI \times N \times b) =$ **0.063** N/mm^2

PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between top skin and web members

Web contact length $b_{con} = N * b =$ **1000** mm

Permissible rolling shear stress at top skin $\tau_{r\_top\_adm} = \tau_{r\_backs\_top} \times K_{36} \times K_{37} \times K_{70} =$ **0.736** N/mm^2

Maximum rolling shear stress $\tau_{r\_top\_max} = V \times EA_{top} \times h_{xs\_top} / (EI \times b_{con}) =$ **0.048** N/mm^2

PASS - Maximum rolling shear stress at top skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{ef} =$ **5.100** mm

Bending deflection $\delta_{bending} = [(5 * W * L_{ef} / 8) + P] * L_{ef}^3 / (48 * EI) =$ **1.944** mm

Shear deflection $\delta_{shear} = 12 * L_{ef} * (W * L_{ef} + 2 * P) / (5 * \Sigma EA) =$ **0.048** mm

Total deflection $\delta_{max} = \delta_{bending} + \delta_{shear} =$ **1.992** mm

PASS - Total deflection is less than permissible deflection

Splice plate design

Load duration factor for plywood $K_{36} =$ **1.00**

Splice to top skin

Compressive stress at extreme fibre of top skin $\sigma_{ms\_top} =$ **1.799** N/mm^2

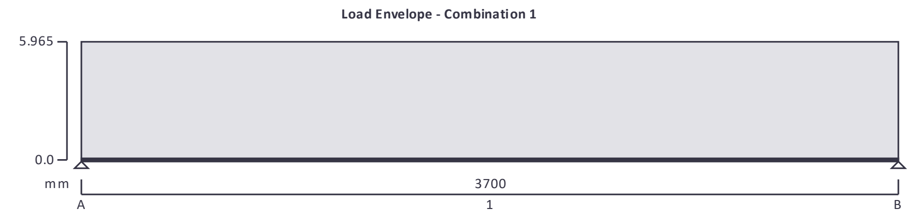
Permissible rolling shear stress at top skin splice $\tau_{r\_top\_splice\_adm} = \tau_{r\_backs\_top} \times K_{36} \times K_{70} =$ **1.107** N/mm^2

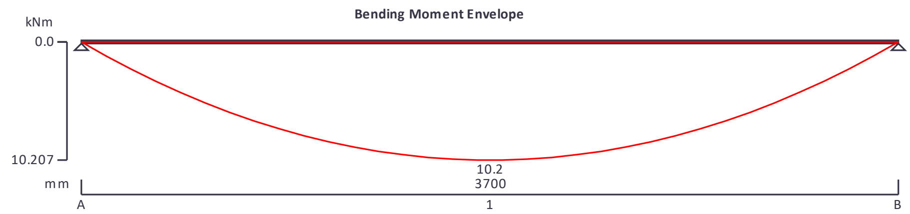
Minimum length of splice plate for top skin $L_{top\_splice} = 2 \times \sigma_{ms\_top} \times t_{mins\_top} / \tau_{r\_top\_splice\_adm} =$ **46** mm

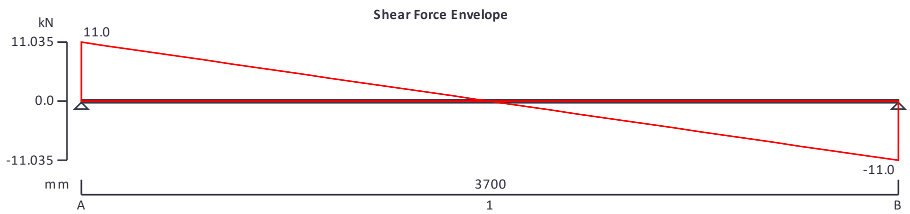
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam * 1
	Dead full UDL 0.81 kN/m
	Imposed full UDL 2.65 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} = **10.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **11** kN	V_{min} = **-11** kN
Deflection	δ_{max} = **8.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **11** kN	R_{A_min} = **11** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **2.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.9** kN	
Maximum reaction at support B	R_{B_max} = **11** kN	R_{B_min} = **11** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.9** kN	

Section details

Section type **STC 203x102x43 (Tata Steel Advance)**

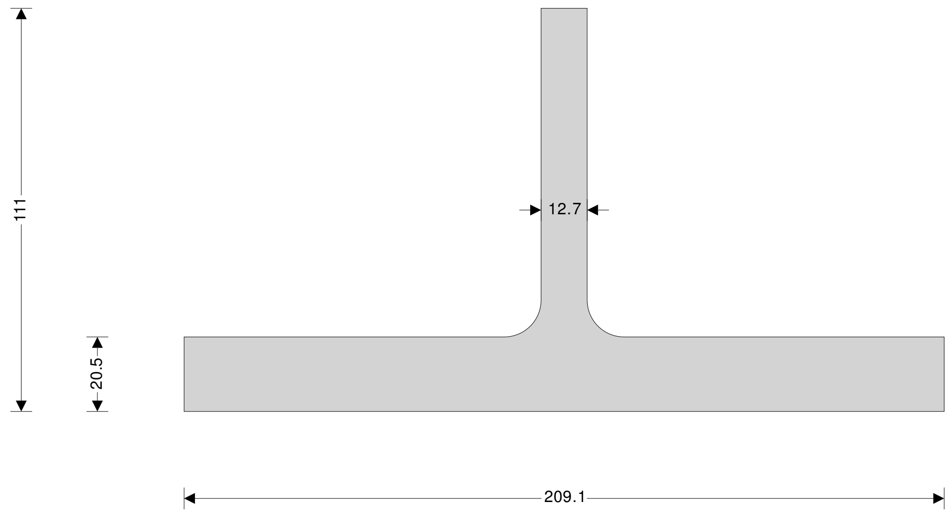
Steel grade **S355**

From table 9: Design strength p_y

Thickness of element max(T, t) = **20.5** mm

Design strength p_y = **345** N/mm^2

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



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

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

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.20** + 2 * D

$K_{LT.B}$ = **1.20** + 2 * D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.89**

Outstand flanges - Table 11

Width of section b = D = **111** mm

b / t = 9.8 * ε <= 18 * ε Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **11** kN

$(D - T - r) / t < 70 * \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t * D$ = **1410** mm^2

Design shear resistance $P_v = 0.6 * p_y * A_v$ = **291.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **10.2** kNm

Effective plastic modulus - Section 3.5.6

Effective plastic modulus - cl.3.5.6.1 $S_{eff} = Z_{xx}$ = **41867** mm^3

Moment capacity low shear - cl.4.2.5.2 $M_c = p_y * \min(Z_{xxflange}, Z_{xxtoe})$ = **14.4** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.2 * L_{s1} + 2 * D$ = **4662** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **87.278**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.000**

Torsional index x = **5.115**

Moment of inertia compression flange minor axis I_{yc} = **0** mm^4

Moment of inertia tension flange minor axis $I_{yt} = T * B^3 / 12$ = **15618334** mm^4

Flange ratio $\eta = I_{yc} / (I_{yc} + I_{yt})$ = **0.000**

Flange ratio factor k_η = **1.000**

Monosymmetry index $\psi = k_\eta \times (2 \times \eta - 1)$ = **-1.000**

Slenderness factor $v = 1 / [(4 \times \eta \times (1 - \eta) + 0.05 \times (\lambda/ x)^2 + \psi^2)^{0.5} + \psi]^{0.5}$ = **0.583**

Ratio - cl.4.3.6.9 $\beta_W = S_{eff} / S_{xx}$ = **0.495**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **0.000**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.632**

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength $p_b = p_y$ = **345** N/mm^2

Buckling resistance moment $M_b = p_b * S_{eff}$ = **14.4** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = \min(14\ mm, L_{s1} / 360)$ = **10.278** mm

Maximum deflection span 1 $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **8.465** mm

PASS - Maximum deflection does not exceed deflection limit