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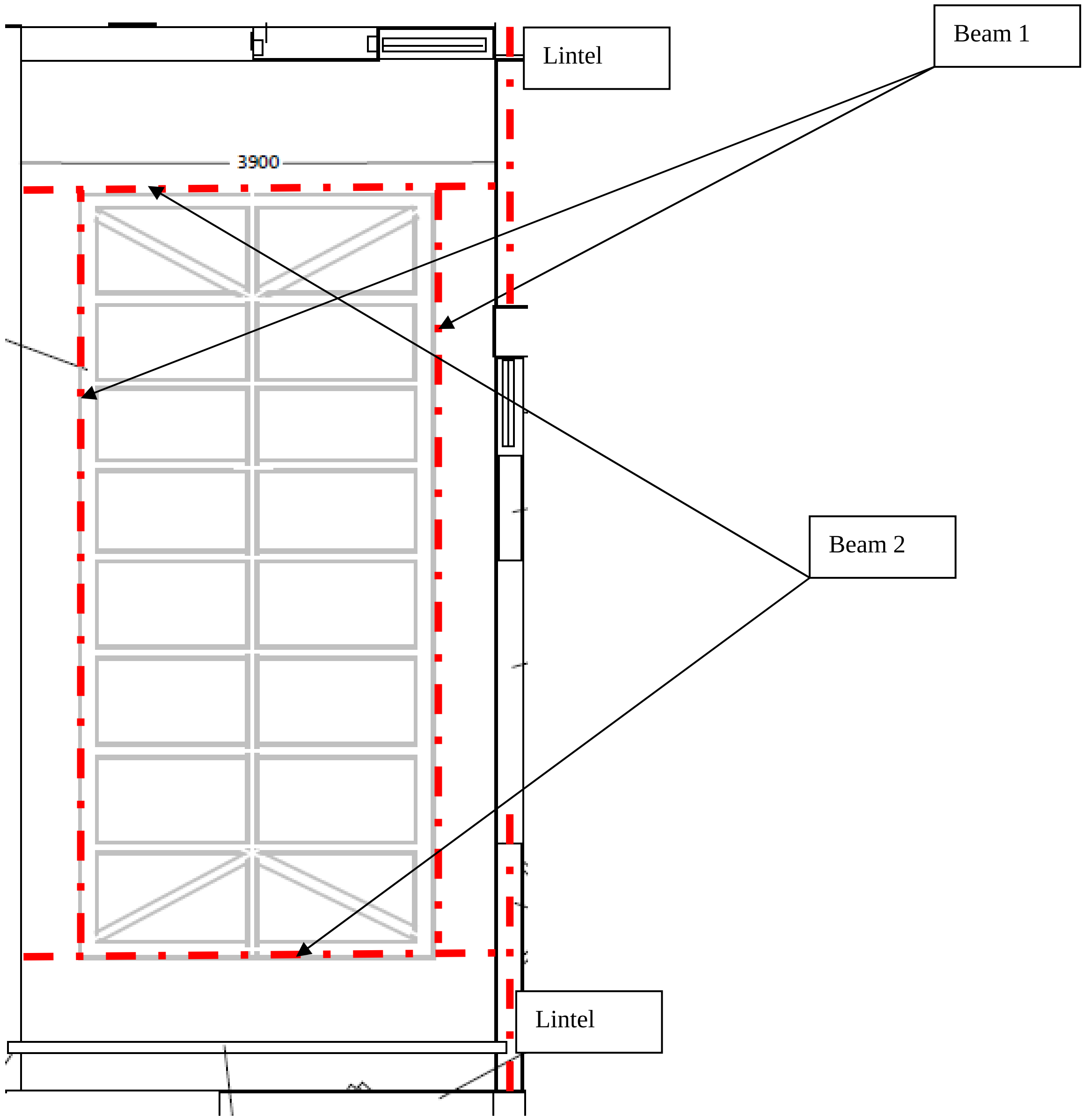
Job ref	: BD110624
Sheet	: **Structure / 1 -**
Made By	: Beam-Designs - SR
Date	: **June 2011/**
	:

Loading

Sun: lounge	Lounge	=	1.378*9.81/(6*2.5)=0.9 kN/m^2	Dead	Live
			TOTAL =	**0.9 kN/ m^2**	
	Snow loading	=	0.6*((60-35)/30)= 0.50 kN/m^2		
			TOTAL =		**0.5 kN/m^2**
Roof: (flat)	Joists & boarding , finishes	=	0.35 kN/m^2		
	Plasterboard	=	0.25 kN/m^2		
			TOTAL =	**0.60 kN/m^2**	
	Imposed	=	0.75 kN/m^2		
			TOTAL =		**0.75 kN/m^2**



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Job ref	: BD110624
Sheet	: **Structure / 2 -**
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Load to Beam 1

Lounge	2.5/2*0.9=1.13	2.5/2*0.5=0.63
Flat roof	0.7/2*0.6=0.21	0.7/2*0.75=0.26
Total	**1.34 kN/m Dead**	**0.89 kN/m Live**

Load to Beam 2
Point loads from Beam 1

Beam 1

C:\USERS\ADMINISTRATOR\DESKTOP\TEE BEAM NO1.$5

AXIAL WITH MOMENTS (MEMBER)

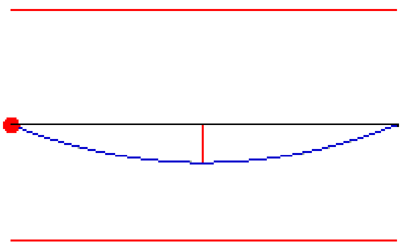
Beam 1

Member 8 (N.3-N.7) @ Level 1 in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -001.340          ( kN/m )
L1 UDLY -000.890          ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3						
Mem ber No.	Node End1 End2	Torque Moment (kN.m)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
8	3	0.000	10.203	0.000	15.305	14.332
	7	0.000	-10.203	0.000	@ 3.000	@ 3.000

Classification and Effective Area (EN 1993: 2006)

Class = Fn($b/T,d/t,f_y,N,M_y,M_z$)	6.41, 30.58, 275, 0, 15.3, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.002 / 156.396 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	15.295 / 47.108 =	0.325	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 15.3, 0.800, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 6 =	6 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.000, 137.6, 4.408, 0.009848, 210000	20.379 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	√ 171.3 x 275 / 20.379	1.520	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.520, 1.575	0.419	Curve b
χ_{LT}.mod = Fn($\chi_{LT},\lambda_{LT},k_c,f$)	0.419, 1.520, 0.942, 1.000	0.419	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.419 x 171.3 x 275 ≤ 47.108 =	19.720 kN.m	
$M_{y.Ed}/M_{b.Rd}$	15.296 / 19.72	0.776	OK

Deflection Check - Load Case 3

δ ≤ Span/360	14.33 ≤ 6000 / 360	14.33 mm	OK

Section (19.04 kg/m)   178x102 UB 19 [Grade 43]

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	Sheet	: **Structure / 3 -**
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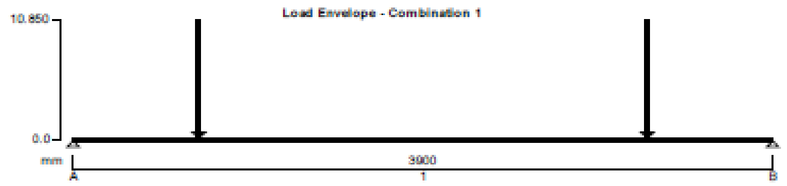
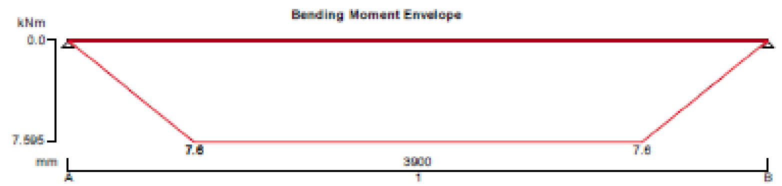
Beam 2

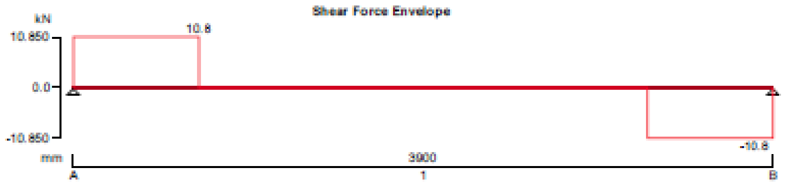
FLITCH BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.05







Applied loading

Beam loads

Permanent point load 2.670 kN at 700 mm
Variable point load 4.830 kN at 700 mm
Permanent point load 2.670 kN at 3200 mm
Variable point load 4.830 kN at 3200 mm

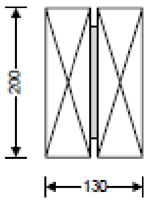
Load combinations

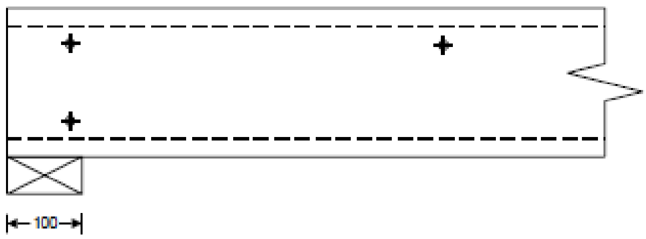
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = 7.595 kNm	M_{min} = 0.000 kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = 7.595 kNm	

Maximum shear	F_{max} = 10.850 kN	F_{min} = -10.849 kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = 10.850 kN	
Total load on beam	W_{tot} = 21.699 kN	
Reactions at support A	R_{A_max} = 10.850 kN	R_{A_min} = 10.850 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 2.670 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 4.830 kN	
Reactions at support B	R_{B_max} = 10.850 kN	R_{B_min} = 10.850 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 2.670 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 4.830 kN	





Timber section details

Breadth of timber sections	b = 60 mm
Depth of timber sections	h = 200 mm
Number of timber sections in member	N = 2
Timber strength class - EN 338:2009 Table 1	C16

Steel section details

Breadth of steel plate	b_s = 10 mm
Depth of steel plate	h_s = 150 mm
Number of steel plates in member	N_s = 1
Nominal yield stress	f_y = 275 N/mm^2
Bolt diameter	ϕ_b = 12 mm

Member details

Load duration - cl.2.3.1.2	**Long-term**
Service class of timber - cl.2.3.1.3	**1**
Length of bearing	L_b = 100 mm

Section properties

Cross sectional area of member	$A = N \times b \times h = 24000$ mm^2
Timber section modulus	$W_{yt} = N \times b \times h^2 / 6 = 800000$ mm^3
Steel section modulus	$W_{ys} = N_s \times b_s \times h_s^2 / 6 = 37500$ mm^3
Second moment of area of timber	$I_{yt} = N \times b \times h^3 / 12 = 80000000$ mm^4
Second moment of area of steel	$I_{ys} = N_s \times b_s \times h_s^3 / 12 = 2812500$ mm^4

Load proportions

Instant deflection under permanent actions	u_{instG} = 2.795 mm
Instant deflection under principal variable action	u_{instQ1} = 5.057 mm
	k_{def} = 0.6
	ψ_2 = 0.3

Final fifth percentile value of modulus of elasticity

$$E_{0.05.fin} = E_{0.05} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) = 4062 \text{ N/mm}^2$$

Final mean value of modulus of elasticity

$E_{0.mean.fin} = E_{0.mean} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) = 6017\ N/mm^2$

Proportion of applied load in timber to check bending, shear and instantaneous deflection

$k_t = E_{0.mean} \times I_{yt} / (E_{0.mean} \times I_{yt} + E_{SEC3} \times I_{ys}) = \mathbf{0.520}$

Proportion of applied load in steel to check strength of bolts and steel plate

$k_s = 1.1 \times E_{SEC3} \times I_{ys} / (E_{0.05.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) = \mathbf{0.710}$

Proportion of applied load in timber to check final deflection

$k_{t.def} = E_{0.mean.fin} \times I_{yt} / (E_{0.mean.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) = \mathbf{0.449}$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = \mathbf{1.300}$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} = \mathbf{0.700}$

Deformation factor for service classes - Table 3.2 $k_{def} = \mathbf{0.600}$

Depth factor for bending - exp.3.1 $k_{h.m} = \mathbf{1.000}$

Depth factor for tension - exp.3.1 $k_{h.t} = \mathbf{1.000}$

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = \mathbf{1.000}$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = \mathbf{0.670}$

Load configuration factor - exp.6.4 $k_{c.90} = \mathbf{1.500}$

System strength factor - cl.6.6 $k_{sys} = \mathbf{1.000}$

Effective length - Table 6.1 $L_{ef} = 1.0 \times L_{s1} + 2 \times h = \mathbf{4300}$ mm

Critical bending stress - exp.6.32 $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef}) = \mathbf{70.527}\ N/mm^2$

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]} = \mathbf{0.476}$

Lateral buckling factor - exp.6.34 $k_{crit} = \mathbf{1.000}$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{B\_max} / (N \times b \times (L_b + \min(L_b, 30\ mm))) = \mathbf{0.695}\ N/mm^2$

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M = \mathbf{1.777}\ N/mm^2$

***PASS** - Design compressive strength exceeds design compressive stress at bearing*

Bending - cl 6.1.6

Design timber bending stress $\sigma_{m.t.d} = k_t \times M / W_{yt} = \mathbf{4.937}\ N/mm^2$

Design timber bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M = \mathbf{8.615}\ N/mm^2$

***PASS** - Design timber bending strength exceeds design timber bending stress*

Design steel bending stress $\sigma_{m.s.d} = k_s \times M / W_{ys} = \mathbf{143.713}\ N/mm^2$

Design steel bending strength $f_{y.d} = f_y / \gamma_{M0} = \mathbf{275.000}\ N/mm^2$

***PASS** - Design steel bending strength exceeds design steel bending stress*

Members subjected to either bending or combined bending and compression - cl.6.3.3

Lateral torsional stability check - eq.6.33 $\sigma_{m.t.d} / (k_{crit} \times f_{m.d}) = \mathbf{0.573}$

***PASS** - Member design meets lateral torsional stability criteria*

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times k_t \times F / (2 \times k_{cr} \times A) = \mathbf{0.526}\ N/mm^2$

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M = \mathbf{1.723}\ N/mm^2$

***PASS** - Design shear strength exceeds design shear stress*

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(14\ mm, 0.003 \times L_{s1}) = \mathbf{11.700}$ mm

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Instantaneous deflection due to permanent load δ_{instG} = **2.982** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **4.772** mm

Instantaneous deflection due to variable load δ_{instQ} = **5.395** mm

Factor for quasi-permanent variable action ψ_2 = **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **6.366** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **11.138** mm

PASS - Total final deflection is less than the deflection limit

Steel-to-timber connections - cl.8.2.3

Characteristic yield moment - exp.8.30 $M_{y.R.k} = 0.3\ mm^{0.4} \times f_{u.k} \times \phi_b^{2.6}$ = **76745** Nmm

Char.embed.strength par.to grain - exp.8.32 $f_{h.0.k} = 0.082e9\ m/sec^2 \times (1\ mm - 0.01 \times \phi_b) \times \rho_k$ = **22.370** N/mm²

$k_{90} = 1.35 + 0.015 \times \phi_b / 1\ mm$ = **1.530**

Char.embed.strength perp.to grain - exp.8.31 $f_{h.90.k} = f_{h.0.k} / k_{90}$ = **14.621** N/mm²

Thickness limit for thin steel plates $b_{s.thn} = \phi_b / 2$ = **6** mm

Thickness limit for thick steel plates $b_{s.thk} = \phi_b$ = **12** mm

Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11

$F_{v.Rk.f} = f_{h.k} \times b \times \phi_b$ = **10.527** kN

$F_{v.Rk.g} = f_{h.k} \times b \times \phi_b \times (\sqrt{[2 + 4 \times M_{y.Rk} / (f_{h.k} \times \phi_b \times b^2)]} - 1)$ = **6.071** kN

$F_{v.Rk.h} = 2.3 \times \sqrt{[M_{y.Rk} \times f_{h.k} \times \phi_b]}$ = **8.440** kN

$F_{v.Rk} = Min(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h})$ = **6.071** kN

Flitch plate bolting requirements

Total load on member W_{tot} = **21.699** kN

Total load taken by steel $W_s = k_s \times W_{tot}$ = **15.398** kN

Number of interfaces $N_{int} = (N + N_s) - 1$ = **2**

Number of bolts required at supports $N_{be} = max(k_s \times R_{B\_max} / (N_{int} \times F_{v.Rk}), 2)$ = **2**

Limiting bolt spacing $S_{limit} = min(2.5 \times h, 600\ mm)$ = **500** mm

Maximum bolt spacing S_{max} = **500** mm

Minimum number of bolts along length of member $N_{bl} = W_s / (N_{int} \times F_{v.Rk})$ = **1.268**

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at maximum 500 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing - Table 8.4 $S_{end} = 7 \times \phi_b$ = **84** mm

Minimum edge spacing - Table 8.4 $S_{edge} = 4 \times \phi_b$ = **48** mm

Minimum bolt spacing - Table 8.4 $S_{bolt} = 4 \times \phi_b$ = **48** mm

Minimum washer diameter - cl.10.4.3 $\phi_w = 3 \times \phi_b$ = **36** mm

Minimum washer thickness - cl.10.4.3 $t_w = 0.3 \times \phi_b$ = **3.6** mm

Use 200x130 C16 Flitch Beam With A 10mm 150 Depth Steel Plate

Lintels

Load to Lintel

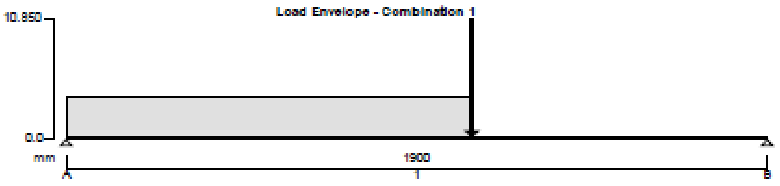
Roof (flat)	3.9/2*0.6=1.17	3.9/2*0.75=1.463
Total	**1.17 kN/m Dead**	**1.463 kN/m Live**

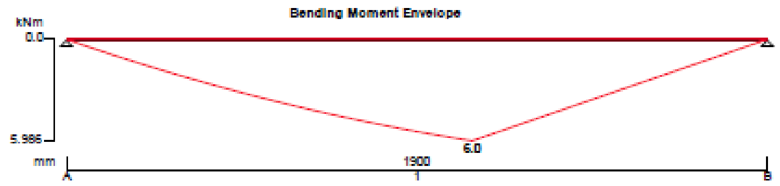
Point load from Beam 2:
2.670 kN 4.830 kN

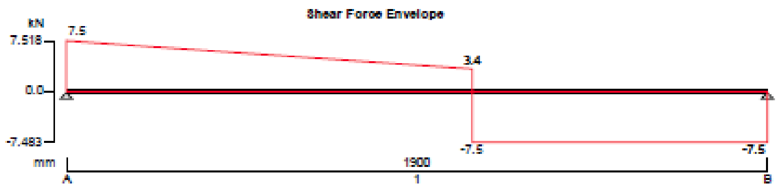
FLITCH BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.05







Applied loading

Beam loads

Permanent partial UDL 1.170 kN/m from 0 mm to 1100 mm
Variable partial UDL 1.463 kN/m from 0 mm to 1100 mm
Permanent point load 2.670 kN at 1100 mm
Variable point load 4.830 kN at 1100 mm

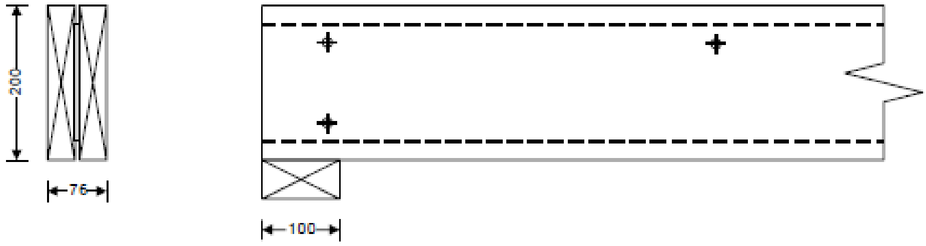
Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = 5.986 kNm	M_{min} = **0.000** kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **5.986** kNm	

Maximum shear	F_{max} = 7.518 kN	F_{min} = -7.483 kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = 7.518 kN	
Total load on beam	W_{tot} = 15.001 kN	
Reactions at support A	R_{A_max} = 7.518 kN	R_{A_min} = 7.518 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 2.039 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 3.177 kN	
Reactions at support B	R_{B_max} = 7.483 kN	R_{B_min} = 7.483 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 1.918 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 3.262 kN	



Timber section details

Breadth of timber sections — b = 35 mm

Depth of timber sections — h = 200 mm

Number of timber sections in member — N = 2

Timber strength class - EN 338:2009 Table 1 — C16

Steel section details

Breadth of steel plate — b_s = 6 mm

Depth of steel plate — h_s = 150 mm

Number of steel plates in member — N_s = 1

Nominal yield stress — f_y = 275 N/mm^2

Bolt diameter — ϕ_b = 12 mm

Member details

Load duration - cl.2.3.1.2 — **Long-term**

Service class of timber - cl.2.3.1.3 — **1**

Length of bearing — L_b = 100 mm

Section properties

Cross sectional area of member — $A = N \times b \times h = 14000$ mm^2

Timber section modulus — $W_{yt} = N \times b \times h^2 / 6 = 466667$ mm^3

Steel section modulus — $W_{ys} = N_s \times b_s \times h_s^2 / 6 = 22500$ mm^3

Second moment of area of timber — $I_{yt} = N \times b \times h^3 / 12 = 46666667$ mm^4

Second moment of area of steel — $I_{ys} = N_s \times b_s \times h_s^3 / 12 = 1687500$ mm^4

Load proportions

Instant deflection under permanent actions — u_{instG} = 0.685 mm

Instant deflection under principal variable action — u_{instQ1} = 1.142 mm

k_{def} = 0.6

ψ_2 = 0.3

Final fifth percentile value of modulus of elasticity

$$E_{0.05.fin} = E_{0.05} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) = 4038 \text{ N/mm}^2$$

Final mean value of modulus of elasticity

$E_{0.mean.fin} = E_{0.mean} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) =$ **5982 N/mm²**

Proportion of applied load in timber to check bending, shear and instantaneous deflection

$k_t = E_{0.mean} \times I_{yt} / (E_{0.mean} \times I_{yt} + E_{SEC3} \times I_{ys}) =$ **0.513**

Proportion of applied load in steel to check strength of bolts and steel plate

$k_s = 1.1 \times E_{SEC3} \times I_{ys} / (E_{0.05.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) =$ **0.718**

Proportion of applied load in timber to check final deflection

$k_{t.def} = E_{0.mean.fin} \times I_{yt} / (E_{0.mean.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) =$ **0.441**

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M =$ **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} =$ **0.700**

Deformation factor for service classes - Table 3.2 $k_{def} =$ **0.600**

Depth factor for bending - exp.3.1 $k_{h.m} =$ **1.000**

Depth factor for tension - exp.3.1 $k_{h.t} =$ **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) $k_m =$ **1.000**

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} =$ **0.670**

Load configuration factor - exp.6.4 $k_{c.90} =$ **1.500**

System strength factor - cl.6.6 $k_{sys} =$ **1.000**

Effective length - Table 6.1 $L_{ef} = 0.5 \times L_{s1} + 2 \times h =$ **1350 mm**

Critical bending stress - exp.6.32 $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef}) =$ **76.440 N/mm²**

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]} =$ **0.458**

Lateral buckling factor - exp.6.34 $k_{crit} =$ **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times (L_b + \min(L_b, 30\text{ mm}))) =$ **0.826 N/mm²**

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M =$ **1.777 N/mm²**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design timber bending stress $\sigma_{m.t.d} = k_t \times M / W_{yt} =$ **6.581 N/mm²**

Design timber bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M =$ **8.615 N/mm²**

PASS - Design timber bending strength exceeds design timber bending stress

Design steel bending stress $\sigma_{m.s.d} = k_s \times M / W_{ys} =$ **191.071 N/mm²**

Design steel bending strength $f_{y.d} = f_y / \gamma_{M0} =$ **275.000 N/mm²**

PASS - Design steel bending strength exceeds design steel bending stress

Members subjected to either bending or combined bending and compression - cl.6.3.3

Lateral torsional stability check - eq.6.33 $\sigma_{m.t.d} / (k_{crit} \times f_{m.d}) =$ **0.764**

PASS - Member design meets lateral torsional stability criteria

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times k_t \times F / (2 \times k_{cr} \times A) =$ **0.617 N/mm²**

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M =$ **1.723 N/mm²**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(14\text{ mm}, 0.003 \times L_{s1}) =$ **5.700 mm**

Instantaneous deflection due to permanent load	δ_{instG} = **0.948** mm
Final deflection due to permanent load	$\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **1.516** mm
Instantaneous deflection due to variable load	δ_{instQ} = **1.590** mm
Factor for quasi-permanent variable action	ψ_2 = **0.3**
Final deflection due to variable load	$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **1.876** mm
Total final deflection	$\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **3.392** mm

PASS - Total final deflection is less than the deflection limit

Steel-to-timber connections - cl.8.2.3

Characteristic yield moment - exp.8.30	$M_{y.R.k} = 0.3\ mm^{0.4} \times f_{u.k} \times \phi_b^{2.5}$ = **76745** Nmm
Char.embed.strength par.to grain - exp.8.32	$f_{h.0.k} = 0.082e9\ m/sec^2 \times (1\ mm - 0.01 \times \phi_b) \times \rho_k$ = **22.370** N/mm^2
	$k_{90} = 1.35 + 0.015 \times \phi_b / 1\ mm$ = **1.530**
Char.embed.strength perp.to grain - exp.8.31	$f_{h.90.k} = f_{h.0.k} / k_{90}$ = **14.621** N/mm^2
Thickness limit for thin steel plates	$b_{s.thn} = \phi_b / 2$ = **6** mm
Thickness limit for thick steel plates	$b_{s.thk} = \phi_b$ = **12** mm

Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11

$F_{v.Rk.f} = f_{h.k} \times b \times \phi_b$ = **6.141** kN

$F_{v.Rk.g} = f_{h.k} \times b \times \phi_b \times (\sqrt{[2 + 4 \times M_{y.Rk} / (f_{h.k} \times \phi_b \times b^2)]} - 1)$ = **5.229** kN

$F_{v.Rk.h} = 2.3 \times \sqrt{[M_{y.Rk} \times f_{h.k} \times \phi_b]}$ = **8.440** kN

$F_{v.Rk} = Min(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h})$ = **5.229** kN

Flitch plate bolting requirements

Total load on member	W_{tot} = **15.001** kN
Total load taken by steel	$W_s = k_s \times W_{tot}$ = **10.773** kN
Number of interfaces	$N_{int} = (N + N_s) - 1$ = **2**
Number of bolts required at supports	$N_{be} = max(k_s \times R_{A\_max} / (N_{int} \times F_{v.Rk}), 2)$ = **2**
Limiting bolt spacing	$S_{limit} = min(2.5 \times h, 600\ mm)$ = **500** mm
Maximum bolt spacing	S_{max} = **500** mm
Minimum number of bolts along length of member	$N_{bl} = W_s / (N_{int} \times F_{v.Rk})$ = **1.03**

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at maximum 500 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing - Table 8.4	$S_{end} = 7 \times \phi_b$ = **84** mm
Minimum edge spacing - Table 8.4	$S_{edge} = 4 \times \phi_b$ = **48** mm
Minimum bolt spacing - Table 8.4	$S_{bolt} = 4 \times \phi_b$ = **48** mm
Minimum washer diameter - cl.10.4.3	$\phi_w = 3 \times \phi_b$ = **36** mm
Minimum washer thickness - cl.10.4.3	$t_w = 0.3 \times \phi_b$ = **3.6** mm

Use 200x76 C16 Flitch Beam With A 6mm 150 Depth Steel Plate

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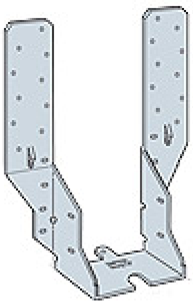
Connections

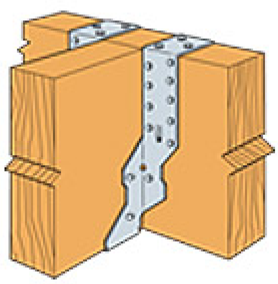
Connection between flitch beam and lintel

JHA450 SPECIFICATIONS

Installation Method	Supported Member Width	Model No.	Dimensions (mm) [1]				Number of Fasteners [2]			Safe Working Loads (kN) [4]			Characteristic Loads (kN)	
			W	H	B	C	Header		Joist	Download		Short Term Uplift	Download	Uplift
							Face	Top		Long Term	Short Term			
	1 ply 38	JHA450/38	38	481	50	191				4.2	4.8	1.6	10.0	3.13
	1 ply 44	JHA450/44	44	478	50	188				4.8	5.5	1.6	11.6	3.13
	1 ply 45	JHA450/47	47	477	50	187				5.2	5.9	1.6	12.4	3.13
	1 ply 50	JHA450/50	50	475	50	185				5.5	6.3	1.6	13.2	3.13
	1 ply 63	JHA450/63	63	469	50	179				5.5	6.3	1.6	13.2	3.13
Wrap Over	1 ply 75	JHA450/75	75	463	50	173	8	4	6	5.5	6.3	1.6	13.2	3.13
	2 ply 45	JHA450/91	91	455	50	165				5.5	6.3	1.6	13.2	3.13
	2 ply 50	JHA450/100	100	450	50	160				5.5	6.3	1.6	13.2	3.13
	2 ply 63	JHA450/125	125	453	63	163				5.7	6.5	1.6	13.6	3.13
	3 ply 45	JHA450/137	137	447	63	157				5.7	6.5	1.6	13.6	3.13
	3 ply 50	JHA450/150	150	440	63	150				5.7	6.5	1.6	13.6	3.13

Use JHA450/137 Simpson Joist Hanger





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Connection between steel beam and flitch beam