# **Design Calculations**

for

## **Internal Alterations**

to

# The Quest, West Street, Harrietsham, Maidstone.

for

## Jessica Arnold & James Whitfield

Calculations Contents A B	Information Vertical Loading
D	Superstructure Design



53/54 St Dunstans Street, Canterbury, Kent, CT2 8BS tel : 01227 464811 fax : 01227 459051 office@bsfconsultants.co.uk

Job No.



Rev: Date:

<b>OBSF</b>	CONSULTING

53/54 St Dunstans Street, Canterbury, Kent,

**SECTION A - INFORMA** 

1.0

1.1 1.2

2.0

2.1

2.2

2.3

2.4

2.5

2.6

3.0 3.1

	Job	The Quest, West Street, Harrietsham, Maidstone	Job No.	186	529
54 St Dunstans Street, Canterbury, Kent, CT2 8BS	Client	Jessica Arnold & James Whitfield	Sheet Rev:	Α	1
tel : 01227 464811 fax : 01227 459051 office@bsfconsultants.co.uk	By Chk'd	Udara Godage BSF	Date: Date:	Sep-23 Sep-23	
CTION A - INFORMATION					
Basic Information Jessica Arnold & James Whitfield James Clague Architects			Client Architect		
Codes of PracticeBS6399 : Part 1 : 1984-Code of practice for deadBS6399 : Part 2 : 1995-Code of practice for wind IBS5268 : Part 2 : 1995-Structural use of timberBS5950 : Part 1 : 1990-Structural use of steelworkBS5628 : Part 1 : 1992-Structural use of unreinforBS8110 : Part 1 : 1997-Structural use of concrete	loads k in builc ced mas	ling	Relevant and Build	Design ( ling Regs	
<b>Project Description</b> The project comprises alterations to an existing building. The with the proposed internal alterrations to the the property.	e calcul	ation and design package is concerned only	Intended element( future de	s), ir	ncluding
Fire Protection By Architect			Fire resis	stance red	qs.

			future design reqs.
4.0	Fire Protecti By Architect	on	Fire resistance reqs.
5.0	Vertical Loa Floor impose Architects sp	d loads are 1.50 kN/m <sup>2</sup> for residential loads. Roof imposed loads are 0.75 kN/m <sup>2</sup> . All dead loads to	General loading
6.0	Horizontal L Wind loading	oads as defined in BS6399.	Wind loading
7.0	<b>Geology</b> From geologi	cal survey maps the underlying sub-strate is believed to consist of Folkestone Formation.	Bearing strata
8.0	Foundations N/A	3	Foundation type
<b>9.0</b> 9.1		orm & Stability building is of traditional masonry construction, which has an inherent robustness and stability.	Structural form & stabilit
<b>10.0</b> 10.1	Materials Concrete	RC40	Material Data
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'.	



Job	The Quest, West Street,	Job No.
	Harrietsham, Maidstone	

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James Sheet

53/54 St Dunstans Street	, Canterbury, Kent, CT2 8BS
tel:01227464811	fax:01227459051

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		reet, Canterbury, Kent,					Rev:	0 00	
	tel:012274648		151		Godage		Date:	Sep-23 Sep-23	
		fconsultants.co.uk		Chk'd BSF			Date:	3ep-23	
_	CTION B	: VERTICAL		DING					
.0	Pitched Roof		kg/m <sup>2</sup>		pitch, θ =	= 40.0 °			
.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	<u>6.00</u>						
			<u>113.00</u> /	cos θ°					
		Plan load	<u>147.51</u>						
			147.51	x 1.4 ( $\gamma_{\rm f}$ ) =	206.52				
.2	Imposed Loads	Roof	50.00	x 1.6 ( $\gamma_{\rm f}$ ) =	80.00				
1.3	Imposed Loads		<u>0.00</u>	x 1.6 ( $\gamma_{\rm f}$ ) =	<u>0.00</u>				
		TOTAL	<u>197.51</u>		<u>286.52</u>				
.4	Dead Load =	1.48 kN/m <sup>2</sup> (service)		2.07 kN/m <sup>2</sup> (ultimate	)		$\gamma_f =$	1.45	
.5	Imposed Load =	0.50 kN/m <sup>2</sup> (service)		0.80 kN/m <sup>2</sup> (ultimate	)		Ser =	1.98 kN	J/n
.6	Total Load =	1.98 kN/m <sup>2</sup> (service)		2.87 kN/m <sup>2</sup> (ultimate			Ult =	2.87 kN	
2.0	Timber Floor	<b>_</b> .	kg/m <sup>2</sup>						
2.1	Dead Loads	Boards	15.00						
		Joists	13.00						
		Plasterboard	12.00						
		Skim	<u>6.00</u>						
			46.00	x 1.4 ( $\gamma_{\rm f}$ ) =	64.40				
2.2	Imposed Loads	IL	<u>150.00</u>	x 1.6 ( $\gamma_{\rm f}$ ) =	<u>240.00</u>				
		TOTAL	<u>196.00</u>		<u>304.40</u>				
2.3	Dead Load =	0.46 kN/m <sup>2</sup> (service)	AND =	0.64 kN/m <sup>2</sup> (ultimate	)		$\gamma_f =$	1.55	
2.4	Imposed Load =	1.50 kN/m <sup>2</sup> (service)	AND =	2.40 kN/m <sup>2</sup> (ultimate			Ser =	1.96 kN	√/n
2.5	Total Load =	1.96 kN/m <sup>2</sup> (service)	AND =	3.04 kN/m <sup>2</sup> (ultimate	)		Ult =	3.04 kN	
3.0	Internal Wall - L		kg/m <sup>2</sup>						
8.1	Dead Loads	Lime Plaster	60.00						
		Brick (102)	<u>195.00</u>		057.00				
		TOTAL	<u>255.00</u>	x 1.4 (γf) =	<u>357.00</u>				
.2	Dead Load =	2.55 kN/m <sup>2</sup> (service)	AND =	3.57 kN/m <sup>2</sup> (ultimate	)		$\gamma_f =$	1.40	_
3.3	Imposed Load =	0.00 kN/m <sup>2</sup> (service)	AND =	0.00 kN/m <sup>2</sup> (ultimate	)		Ser =	2.55 kN	√/n
5.3		2.55 kN/m <sup>2</sup> (service)		3.57 kN/m <sup>2</sup> (ultimate			Ult =	3.57 kN	

Client Jessica

		NSULTING	3	Job		est, West Stre ham, Maidste			Job No.	186	29
	<b>DOF</b> END	SINEERS		Client	Jessica	Arnold	&	James	Sheet	D	1
	4 St Dunstans Street, Canter tel : 01227 464811 fax : 01 office@bsfconsultants	227 45905 1	BS	By Chk'd	Whitfield Udara God BSF				Rev: Date: Date:	Sep-23 Sep-23	
SE	CTION D : SUPI	ER STRU	CTUR								
1.0	Timber Joist Design					Timber to I					
1.1	1F Floor Joists - Bedroom fl Loading UDL - over entire length Floor = 1 x 1.96 kN/m <sup>2</sup> x C =	oor - 1600 mm cle Vertical UDL,	Dead Lo <u>0.18</u>	oad <u>kN/m</u> k <b>N/m ([</b>	In	ist Length, <b>I</b> <sub>b</sub> = Centres, <b>C</b> = posed Load <u>0.60</u> kN/m <b>0.60 kN/m (IL)</b>		m mm			
		Total UDL,	w = 1.22	kN/m (l	& (L	0.78 kN/m (S)					
1.2	Analysis Shear, $\mathbf{F}_{\mathbf{v}} = wl/2 =$ Moment, $\mathbf{M} = wl^2/8 =$ Deflection, $\boldsymbol{\delta} = 5wl^4/384El =$			′kN kNm Nm <sup>3</sup> (/E	EI)	F <sub>v</sub> (U) =	1.03	kN			
1.3	Design	Try section: 1 No									
	Grade C16 Ply = 1 b = 100 mm d = 50 mm	K <sub>7</sub> = 1	1.00 1.17 1.10	K <sub>15</sub> = K <sub>19</sub> = K <sub>20</sub> =	1.00 1.00 1.00	E <sub>mean</sub> = ♂ <sub>mall</sub> = r <sub>g</sub> =	5.30	N/mm <sup>2</sup> N/mm <sup>2</sup> N/mm <sup>2</sup>			
	$\begin{array}{l} \textbf{Deflection} \\ \text{Allowable deflection, } \boldsymbol{\delta} = 0.003 \\ \text{Actual deflection, } \boldsymbol{\delta}_t = \delta_a + (19) \\ \textbf{Bending} \\ \text{Allowable bending, } \boldsymbol{\sigma}_{\textbf{macl}} = \boldsymbol{\sigma}_m \\ \text{Actual bending, } \boldsymbol{\sigma}_{\textbf{macl}} = M/Z = \\ \textbf{Shear} \\ \text{Allowable shear, } \mathbf{r}_{\textbf{g}} = r_{\textbf{g}} k_8 k_{19} = \\ \text{Actual Shear, } \mathbf{r}_{\textbf{a}} = 1.5 \text{F/A} = \end{array}$	.2M/EK <sub>20</sub> A) = <sub>all</sub> k <sub>3</sub> k <sub>7</sub> k <sub>8</sub> k <sub>15</sub> =	9.4 6.82 6.80 0.74	mm mm <sup>2</sup> N/mm <sup>2</sup> N/mm <sup>2</sup> N/mm <sup>2</sup>					f <mark>ails</mark> okay okay		
1.4	Joists satisfactory as: fail	<u>s</u>						Ex. TJ			
2.0	Note: In the absence of structura imposed load is taken as 1. have adequate bending and we would recommend that y the floor as this will make doubled up below bath loca timber spreader.	50 kN/m2 as per l   shear capacity.   vou added a 15 mi it act as a stress	BS6399 for However, it m x 11 ply F skin. BSF	residenti is failing innish b would a	ial propertie g due to ex irch plywoo ilso recomr	es. The existing cessive deflecti od 1.4 mm venee nend that the J	joist s ion. Th er, to th oists a	eems to lerefore, le top of re to be			
3.0	For the Floor Analysis & Des	sign: See Append	ix A1.1								
4.0	Timber Beam Design 1F Floor Beam - Existing - s	upport floor joist	- 3600 mm d			Timber to I					
4.1	Loading UDL - over entire length Floor = 1 x 1.96 kN/m2 x 3.5 r Self Weight = $\rho_{mean} x b x d x p$		w = 0.98		In DL) &	am Length, I <sub>b</sub> = nposed Load 2.63 kN/m <u>0.00</u> kN/m <b>2.63 kN/m (IL)</b> <b>3.60 kN/m (S)</b>		m			
4.2	Analysis Shear, $F_v = wl/2 =$ Moment, $M = wl^2/8 =$ Deflection, $\delta = 5wl^4/384EI =$			′ kN ′ kNm ⁄ Nm <sup>3</sup> (/E	:1)	F <sub>v</sub> (U) =	10.31	kN			



### Job The Quest, West Street, Job No. 18629 Harrietsham, Maidstone

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			JGINEEF		Client	Jessica Whitfield	Arnold I	& Jam	nes Sheet	D	
3/5		ans Street, Cant 464811 fax :			Ву	Udara Goda	909		Rev: Date:	Sep-23	
		ice@bsfconsultar			Chk'd	BSF	age		Date:	Sep-23	
}	Design			1 No. 185	x150 D30 timb				24.0.		
	Grade	D30	K <sub>3</sub> =	1.00	K <sub>15</sub> =	1.00	$\rho_{mean} =$	640 kg/m	n <sup>3</sup>		
	Ply =	1	K <sub>7</sub> =	1.08	K <sub>19</sub> =	1.00	E <sub>min</sub> =	6000 N/m	m²		
			K <sub>8</sub> =	1.00	K <sub>20</sub> =	1.00					
	b =	185 mm	K <sub>9</sub> =	1.00			$\sigma_{mall} =$	9.00 N/m	-		
	d =	150 mm					r <sub>g</sub> =	1.40 N/m	m²		
	Deflection	-									
	Deflection	deflection, $\delta = 0.0$	003I –		11.1 mm						
		lection, $\boldsymbol{\delta}_{t} = \delta_{a} + ($		_	28.9 mm				fails		
	Bending		(101 <u></u> 111) <u>11</u> 9. 2017)		2010 1111				iano		
		bending, $\sigma_{mall} = 0$	$\sigma_{mall}k_3k_7k_8k_{15} =$		9.71 N/mm <sup>2</sup>						
		nding, $\sigma_{macll} = M/2$			8.89 N/mm <sup>2</sup>				okay		
	Shear										
		shear, $\mathbf{r}_{g} = r_{g}k_{8}k_{19}$			1.40 N/mm <sup>2</sup>						
	Actual She	ear, <b>r</b> <sub>a</sub> = 1.5F/A =			0.36 N/mm <sup>2</sup>				okay		
4	Continn	tiofootor:	iaila					E	тв		
+	Section Sa	tisfactory as: f	allS					<u> </u>			
)											
0	For the St	teel Beam Analys	sis & Design: S	ee Appen	dix A1.2						
0	Timber B										
	Existing \	cam Deargn					Timber to I	3S5268 : Pa	irt 2		
1	Loading	Wall Beam - Lint					Timber to I	3S5268 : Pa	irt 2		
		Wall Beam - Linte		ear span		Effec	tive length, <b>I =</b>	3S5268 : Pa 1.68 m	irt 2		
		Wall Beam - Linte	el - 1525 mm clo	ear span	Dead Load	Effec	tive length, <b>I =</b> posed Load		irt 2		
		Wall Beam - Linte er entire length < 1.96 kN/m2 x 1.3	<b>el - 1525 mm cl</b> e 7 m x 50% =	ear span	Dead Load 0.39 kN/m	Effec	tive length, <b>I =</b> posed Load 1.28 kN/m		irt 2		
	Wall = 1 x	Wall Beam - Lint er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9	el - 1525 mm clo 7 m x 50% = 9 m x 100% =	ear span	Dead Load 0.39 kN/m 2.30 kN/m	Effec	tive length, <b>I =</b> posed Load 1.28 kN/m 0.00 kN/m		irt 2		
	Wall = 1 x	Wall Beam - Linte er entire length < 1.96 kN/m2 x 1.3	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply =	ear span [	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m	Effec Im	tive length, <b>I =</b> posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m		irt 2		
	Wall = 1 x	Wall Beam - Lint er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical U	ear span [	Dead Load 0.39 kN/m 2.30 kN/m	Effec Im DL) &	tive length, <b>I =</b> posed Load 1.28 kN/m 0.00 kN/m		irt 2		
	Wall = 1 x Self Weigl	Wall Beam - Lint er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical U	ear span [ JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b>	Effec Im DL) &	tive length, <b>I =</b> posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b>		irt 2		
2	Wall = 1 x Self Weigl Analysis	Wall Beam - Linteger entire length $x = ntire length$ $x = 0.96 \text{ kN/m2 x } 1.325 \text{ kN/m2 x } 0.825 \text$	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical U	ear span [ JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b> 5.92 kN/m (I	Effec Im DL) &	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S)	1.68 m	irt 2		
2	Wall = 1 x Self Weigl <b>Analysis</b> Shear, <b>F</b> v	Wall Beam - Linterer entire length $x 1.96 \text{ kN/m2 x 1.}$ $2.55 \text{ kN/m2 x 0.9}$ $x 1.55 \text{ kN/m2 x 0.9}$ $x 1 = \rho_{mean} x b x d 3$	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical U	ear span [ JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b>	Effec Im DL) &	tive length, <b>I =</b> posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b>		irt 2		
2	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I	Wall Beam - Linte er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = ρ <sub>mean</sub> x b x d 3 = wl/2 = M = wl <sup>2</sup> /8 =	el - 1525 mm cle 7 m x 50% = 9 m x 100% = x ply = Vertical U Total U	ear span [ JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> 1.42 kNm	Effec Im DL) & J) &	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S)	1.68 m	irt 2		
2	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I	Wall Beam - Linterer entire length $x 1.96 \text{ kN/m2 x 1.}$ $2.55 \text{ kN/m2 x 0.9}$ $x 1.55 \text{ kN/m2 x 0.9}$ $x 1 = \rho_{mean} x b x d 3$	el - 1525 mm cle 7 m x 50% = 9 m x 100% = x ply = Vertical U Total U	ear span [ JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b>	Effec Im DL) & J) &	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S)	1.68 m	irt 2		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I	Wall Beam - Linte er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = ρ <sub>mean</sub> x b x d 3 = wl/2 = M = wl <sup>2</sup> /8 =	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L	ear span JDL, w = JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m <u>0.09</u> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> 1.42 kNm	Effec Im DL) & J) &	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S)	1.68 m	irt 2		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection	Wall Beam - Linte er entire length < 1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = ρ <sub>mean</sub> x b x d 3 = wl/2 = M = wl <sup>2</sup> /8 =	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L	ear span JDL, w = JDL, w =	Dead Load 0.39 kN/m 2.30 kN/m 0.09 kN/m 2.77 kN/m (I 5.92 kN/m (I 3.40 kN 1.42 kNm 417 Nm <sup>3</sup> (/E	Effec Im DL) & J) &	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <u>0.00</u> kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S)	1.68 m			
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $wl/2 =$ M = $wl^2/8 =$ , $\delta = 5wl^4/384EI =$	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: K <sub>3</sub> = K <sub>7</sub> =	ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> = K <sub>19</sub> =	Effec Im DL) & J) & I)	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S) F <sub>v</sub> (U) =	<b>1.68 m</b> 4.97 kN	n <sup>3</sup>		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply =	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $\mu_{mean} x b x d =$ M = wl/2 = $M = wl^2/8 =$ , $\delta = 5wl^4/384EI =$ D30 1	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: K <sub>3</sub> = K <sub>7</sub> = K <sub>8</sub> =	Ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13 1.00	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> =	Effec Im DL) & J) & I) & I)	tive length, <b>I</b> = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> <b>4.05 kN/m (S)</b> F <sub>v</sub> (U) = ρ <sub>mean</sub> = E <sub>min</sub> =	1.68 m 4.97 kN 640 kg/m 6000 N/m	n <sup>3</sup> m <sup>2</sup>		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply = b =	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $\mu_{mean} x b x d =$ <b>M</b> = wl/2 = <b>M</b> = wl <sup>2</sup> /8 = , $\delta = 5wl^4/384EI =$ D30 1 140 mm	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: K <sub>3</sub> = K <sub>7</sub> =	ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> = K <sub>19</sub> =	Effec Im DL) & J) & I) & I) Sr 1.00 1.00	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S) $F_v$ (U) = $\rho_{mean} = E_{min} =$ $\sigma_{mall} =$	1.68 m 4.97 kN 640 kg/n 6000 N/m 9.00 N/m	n <sup>3</sup> m <sup>2</sup> m <sup>2</sup>		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply =	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $\mu_{mean} x b x d =$ M = wl/2 = $M = wl^2/8 =$ , $\delta = 5wl^4/384EI =$ D30 1	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: K <sub>3</sub> = K <sub>7</sub> = K <sub>8</sub> =	Ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13 1.00	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> = K <sub>19</sub> =	Effec Im DL) & J) & I) & I) Sr 1.00 1.00	tive length, <b>I</b> = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> <b>4.05 kN/m (S)</b> F <sub>v</sub> (U) = ρ <sub>mean</sub> = E <sub>min</sub> =	1.68 m 4.97 kN 640 kg/m 6000 N/m	n <sup>3</sup> m <sup>2</sup> m <sup>2</sup>		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply = b = d =	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $\mu_{mean} x b x d =$ <b>M</b> = wl/2 = <b>M</b> = wl <sup>2</sup> /8 = , $\delta = 5wl^4/384EI =$ D30 1 140 mm 100 mm	el - 1525 mm clo 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: K <sub>3</sub> = K <sub>7</sub> = K <sub>8</sub> =	Ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13 1.00	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> = K <sub>19</sub> =	Effec Im DL) & J) & I) & I) Sr 1.00 1.00	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S) $F_v$ (U) = $\rho_{mean} = E_{min} =$ $\sigma_{mall} =$	1.68 m 4.97 kN 640 kg/n 6000 N/m 9.00 N/m	n <sup>3</sup> m <sup>2</sup> m <sup>2</sup>		
	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply = b = d = Deflection	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} \times b \times d =$ mt = $wl/2 =$ M = $wl^2/8 =$ , $\delta = 5wl^4/384EI =$ D30 1 140 mm 100 mm	el - 1525 mm cle 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: $K_3 =$ $K_7 =$ $K_8 =$ $K_9 =$	Ear span JDL, w = JDL, w = 1 No. 140 1.00 1.13 1.00	Dead Load 0.39 kN/m 2.30 kN/m <b>0.09</b> kN/m <b>2.77 kN/m (I</b> <b>5.92 kN/m (I</b> <b>3.40 kN</b> <b>1.42 kNm</b> <b>417 Nm<sup>3</sup> (/E</b> <b>x100 D30 timb</b> K <sub>15</sub> = K <sub>19</sub> =	Effec Im DL) & J) & I) & I) Sr 1.00 1.00	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S) $F_v$ (U) = $\rho_{mean} = E_{min} =$ $\sigma_{mall} =$	1.68 m 4.97 kN 640 kg/n 6000 N/m 9.00 N/m	n <sup>3</sup> m <sup>2</sup> m <sup>2</sup>		
2	Wall = 1 x Self Weigl Analysis Shear, F <sub>v</sub> Moment, I Deflection Design Grade Ply = b = d = Deflection Allowable	<b>Wall Beam - Linte</b> er entire length (1.96 kN/m2 x 1. 2.55 kN/m2 x 0.9 ht = $\rho_{mean} x b x d =$ ht = $\mu_{mean} x b x d =$ <b>M</b> = wl/2 = <b>M</b> = wl <sup>2</sup> /8 = , $\delta = 5wl^4/384EI =$ D30 1 140 mm 100 mm	el - 1525 mm cle 7 m x 50% = 9 m x 100% = x ply = Vertical L Total L = Try section: $K_3 =$ $K_7 =$ $K_8 =$ $K_9 =$ 0031 =	Ear span JDL, w = JDL, w = JDL, w = 1.00 1.13 1.00 1.00	Dead Load 0.39 kN/m 2.30 kN/m 2.77 kN/m (I 5.92 kN/m (I 3.40 kN 1.42 kNm 417 Nm <sup>3</sup> (/E x100 D30 timbe $K_{15} =$ $K_{19} =$ $K_{20} =$	Effec Im DL) & J) & I) & I) Sr 1.00 1.00	tive length, I = posed Load 1.28 kN/m 0.00 kN/m <b>1.28 kN/m (IL)</b> 4.05 kN/m (S) $F_v$ (U) = $\rho_{mean} = E_{min} =$ $\sigma_{mall} =$	1.68 m 4.97 kN 640 kg/n 6000 N/m 9.00 N/m	n <sup>3</sup> m <sup>2</sup> m <sup>2</sup>		

BendingAllowable bending,  $\sigma_{mall} = \sigma_{mall} k_3 k_7 k_8 k_{15} =$ 10.16 N/mm²Actual bending,  $\sigma_{macll} = M/Z =$ 6.10 N/mm²

okay

	BSES	NSULTIN	йG	Job		est, West Stro sham, Maidsto			Job No.	186	529
		GINEER	5	Client	Jessica Whitfiel		& Ja	ames		D	3
	4 St Dunstans Street, Cante tel : 01227 464811 fax : 0		2882	By	Udara Go	dane			Rev: Date:	Sep-23	
	office@bsfconsultant			Chk'd	BSF	augo			Date:	Sep-23	
	Shear										
	Allowable shear, $\mathbf{r_g} = \mathbf{r_g} \mathbf{k_8} \mathbf{k_{19}}$ Actual Shear, $\mathbf{r_a} = 1.5 \text{F/A} =$	=		1.40 N/mm <sup>2</sup> 0.36 N/mm <sup>2</sup>					okay		
4	Section satisfactory as: fa	ils					E	x. TLB	·		
0	Timber Beam Design					Timber to	BS5268 :	Part 2			
	Existing Wall Beam - Lintel	- 1400 mm clea	r span								
1	Loading					ective length, I =	1.54 n	n			
	UDL - over entire length		D	ead Load	I	nposed Load					
	Floor = 1 x 1.96 kN/m2 x 1.7			0.39 kN/m		1.28 kN/m					
	$Wall = 1 \times 2.55 \text{ kN/m} 2 \times 0.8 \text{ m}$			2.04 kN/m		0.00 kN/m					
	Self Weight = $\rho_{mean} x b x d x$	Vertical UE	) w _	0.09 kN/m	۹ <b>ו וח</b>	<u>0.00</u> kN/m <b>1.28 kN/m (IL)</b>					
		venical OL	)L, <b>W</b> =	2.52 kN/m (		1.20 KIN/III (IL)	•				
		Total UE	)L, <b>w</b> =	5.57 kN/m (	U) &	3.79 kN/m (S)					
2	Analysis					<b>F</b> (1)					
	Shear, $\mathbf{F_v} = wl/2 =$			2.92 kN 1.12 kNm		$F_v(U) =$	4.29 k	N			
	Moment, $\mathbf{M} = wl^2/8 =$ Deflection, $\boldsymbol{\delta} = 5wl^4/384El =$			278 Nm <sup>3</sup> (/I	=1)						
	Define tion, $0 = 5 \text{WI} / 384 \text{EI} =$			270 NM* (/I	=1)						
3	Design	Try section: 1									
	Grade D30	K <sub>3</sub> =	1.00	K <sub>15</sub> =	1.00	$\rho_{\text{mean}} =$	640 k				
	Ply = 1	K <sub>7</sub> =	1.13	K <sub>19</sub> =	1.00	E <sub>min</sub> =	6000 N	l/mm²			
	h 140 mm	K <sub>8</sub> = K <sub>9</sub> =	1.00	K <sub>20</sub> =	1.00	đ	0.00				
	b = 140 mm d = 100 mm	r\ <sub>9</sub> =	1.00			$\sigma_{mall} = r_{g} =$	9.00 N 1.40 N				
	Deflection										
	<b>Deflection</b> Allowable deflection, $\delta = 0.00$	13I –		4.6 mm							
	Actual deflection, $\delta_t = \delta_a + (19)$ Bending			4.2 mm					okay		
	Allowable bending, $\sigma_{mall} = \sigma_{r}$	mallk3k7k8k15 =		10.16 N/mm <sup>2</sup>							
	Actual bending, $\sigma_{macll} = M/Z$ Shear			4.82 N/mm <sup>2</sup>					okay		
	Allowable shear, $\mathbf{r}_{a} = r_{a}k_{8}k_{19}$	=		1.40 N/mm <sup>2</sup>							
	Actual Shear, $\mathbf{r_a} = 1.5 F/A =$			0.31 N/mm <sup>2</sup>					okay		
4	Section satisfactory as: 1	No. 140x100 D3	) timber				E	x. TLB			
0											
•	Note:										
	In the absence of structura										
	D30. The existing timber be										
	of 1.525m clear span. How	wever, the exist	ing bear	n have adequ	late capaci	ty to support ar	n clear s	pan of			
	1 (1)		rooflas	lingo The be	am 600		a timba	r noct			
	1.4m. Please note that we have n	not allowed any	1001 1080	•			•				
	Please note that we have n	· · · · · · · · · · · · · · · · · · ·		o allowance							
	Please note that we have n The extent of this post is	unknown and	hence n					e post			
	Please note that we have n	unknown and opened up prio	hence n r to com					e post			
	Please note that we have n The extent of this post is recommend that this to be and finding to be reported	unknown and opened up prio	hence n r to com			etermine the ext	tent of th		·		
0.0	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design	unknown and opened up prio back to the BSF	hence n r to com	mencement o	f work to d	etermine the exit	tent of th BS5268 :	Part 2			
<b>0.0</b>	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design New Floor Joists - new bat	unknown and opened up prio back to the BSF	hence n r to com	mencement o	f work to d	Timber to lective length, I =	tent of th BS5268 : 2.40 n	Part 2			
<b>0.0</b>	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design	unknown and opened up prio back to the BSF	hence n r to com	mencement o	f work to d	etermine the exit	tent of th BS5268 :	Part 2			
	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design New Floor Joists - new bat Loading	unknown and opened up prio back to the BSF 	hence n r to com	mencement o	f work to d	Timber to ective length, I = Centres, C =	tent of th BS5268 : 2.40 n	Part 2			
	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design New Floor Joists - new bat Loading UDL - over entire length	unknown and opened up prio back to the BSF 	hence n r to com	mencement o	f work to d	Timber to Timber to ective length, I = Centres, C = nposed Load	tent of th BS5268 : 2.40 n 400 n	Part 2			
	Please note that we have n The extent of this post is recommend that this to be and finding to be reported Timber Joist Design New Floor Joists - new bat Loading UDL - over entire length	unknown and opened up prio back to the BSF 	hence n r to com 00 mm c D DL, w =	lear span ead Load 0.18 kN/m	f work to d Effe In DL) &	Timber to I ctive length, I = Centres, C = nposed Load <u>0.60</u> kN/m	tent of th BS5268 : 2.40 n 400 n	Part 2			

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TING RS	

#### Job The Quest, West Street, Job No. 18629 Harrietsham, Maidstone Client James Sheet Jessica Arnold & D 4 Whitfield Rev: Sep-23 Sep-23 Date:

1.46 kN

8800 N/mm<sup>2</sup>

FJ

Date:

53/5	i4 St Dunst	ans Street, Canter	rbury, Kent, C	T2 885		whittield	
	tel:01227	7464811 fax:0	1227 45905	i1	By	Udara Godage	9
	of	fice@bsfconsultant	s.co.uk		Chk'd	BSF	
10.2	Analysis						
	Shear, <b>F</b> ,	, = wl/2 =			0.94 kN		$F_v(U) =$
	Moment,	$M = wl^2/8 =$			0.56 kNm		
	Deflection	n, $\delta = 5 \text{wl}^4 / 384 \text{EI} =$			339 Nm <sup>3</sup> (/E	EI)	
10.3	Design		Try section:	1 No. 47x1	50 C16 timbe	r at 400 mm ce	ntres
	Grade	C16	K <sub>3</sub> =	1.00	K <sub>15</sub> =	1.00	E <sub>mean</sub> =
	Ply =	1	K <sub>7</sub> =	1.08	K <sub>19</sub> =	1.00	$\sigma_{mall} =$
	b =	47 mm	K <sub>8</sub> =	1.10	$K_{20} =$	1.00	$r_{\alpha} =$

arauc	010	13 -	1.00	15 -	1.00	-mean -		1
Ply =	1	K <sub>7</sub> =	1.08	K <sub>19</sub> =	1.00	$\sigma_{mall} =$	5.30 N/mm <sup>2</sup>	
b =	47 mm	K <sub>8</sub> =	1.10	K <sub>20</sub> =	1.00	r <sub>g</sub> =	0.67 N/mm <sup>2</sup>	
d =	150 mm							
Deflectio	on							
Allowable	e deflection, $\boldsymbol{\delta} = 0.0$	031 =		7.2 mm				
Actual de	eflection, $\boldsymbol{\delta}_{t} = \delta_{a} + (1)$	$19.2M/EK_{20}A) =$		3.1 mm				okay
Bending								_
Allowable	e bending, $\sigma_{mall}$ = $\sigma$	<sub>mall</sub> k <sub>3</sub> k <sub>7</sub> k <sub>8</sub> k <sub>15</sub> =		6.29 N/mm <sup>2</sup>				
Actual be	ending, $\sigma_{macll} = M/Z$	=		3.20 N/mm <sup>2</sup>				okay
Shear								
Allowable	e shear, $\mathbf{r}_{\mathbf{q}} = \mathbf{r}_{\mathbf{q}}\mathbf{k}_{8}\mathbf{k}_{19}$	=		0.74 N/mm <sup>2</sup>				ĺ
	near, <b>r</b> , = 1.5F/A =			0.20 N/mm <sup>2</sup>				okay

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

Tekla. Tedds	Project The Que	Job no. 18629				
	Calcs for App	endix A1.1 - Stre	ss Skin Panel	Design	Start page no./Revision 1	
	Calcs by U.G	Approved by	Approved date			

STRESS SKIN PANEL DESIGN (BS5268-2	<u>2:2002)</u>
	TEDDS calculation version 1.0.0
Single-skin panel details	
Effective span of panel	L <sub>ef</sub> = <b>1700</b> mm
Panel width	b <sub>panel</sub> = <b>3700</b> mm
Web member depth	h = <b>50</b> mm
Web member breadth	b = <b>100</b> mm
Web member spacing	s = <b>400</b> 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 <sub>mins_top</sub> = <b>14.3</b> mm
Panel depth	t <sub>panel</sub> = t <sub>mins_top</sub> + h = <b>64</b> mm
9	
<b>▲</b> 400 <b>→▲</b> 400 <b>→▲</b> 400 <b>→▲</b>	

#### Section properties

• •	
Top skin partial flange width	$b_{se\_top} = min(25 \times t_{mins\_top}, 0.1 \times L_{ef}, s - b) = 170 \text{ mm}$
Top skin effective width	$b_{s\_top} = b_{panel} - (N - 1) \times (s - b - b_{se\_top}) = 2530 \text{ mm}$
Top skin area	$A_{s\_top} = b_{s\_top} \times t_{mins\_top} = 36179 mm^2$
Product of EA for top skin	$EA_{top} = E_{t\_pars\_top} \times A_{s\_top} = 151951.800 \text{ kN}$
Distance from centroid to top surface	$y_{s_{top}} = (t_{mins_{top}} / 2) = 7.2 mm$
Web member area	$A = N \times b \times h = 50000 \text{ mm}^2$
Product of EA for web member	EA = E <sub>mean</sub> × A = <b>440000.000</b> kN
Distance from centroid to top surface	$y = (t_{mins\_top} + h / 2) = 39.3 mm$
Summation of product EA for panel	$\Sigma EA = EA_{top} + EA = 591952 \text{ kN}$
Summation of product EAy for panel	$\Sigma EAy = EA_{top} \times y_{s_{top}} + EA \times y = $ <b>18378</b> kNm
Neutral axis depth	$\overline{y} = \Sigma EAy / \Sigma EA = 31.0 \text{ mm}$
Distance from NA to centroid of top skin	$h_{xs_{top}} = abs(y_{s_{top}} - \overline{y}) = 23.897 \text{ mm}$
Distance from NA to centroid of web members	$h_x = abs(y - \overline{y}) = 8.253 mm$
Bending rigidity of panel	$EI = EA \times h^2 / 12 + EA_{top} \times h_{xs\_top}^2 + EA \times h_x^2 = \textbf{208} \text{ kN/m}^{-2}$
Loading details	
Panel self weight	$F_{swt} = (N \times b \times h \times \rho + s_{wts\_top} \times b_{panel}) \times g_{acc} = 0.559 \text{ kN/m}$
Dead load	$F_{d_udl} = 0.50 \text{ kN/m}^2$
Imposed UDL	F <sub>i_udl</sub> = <b>1.50</b> kN/m <sup>2</sup>
Imposed point load	F <sub>i_pt</sub> = <b>1.40</b> kN
Modification factors	
Section depth factor	K <sub>7</sub> = <b>1.17</b>
Load sharing factor	K <sub>8</sub> = 1.10
Stress concentration modification factor	K <sub>37</sub> = <b>0.50</b>
Nail glue modification factor	K <sub>70</sub> = <b>0.90</b>
Consider long term loads	
Load duration factor for timber	K <sub>3</sub> = <b>1.00</b>
Load duration factor for plywood	K <sub>36</sub> = <b>1.00</b>

<b>Tekla</b> Tedds	Project The Qu	est, West Street	, Harrietsham,	Maidstone	Job no. 1	8629	
	Calcs for		,		Start page no./	Revision	
	Арр	endix A1.1 - Stre	ess Skin Panel	Design		2	
	Calcs by U.G	Calcs date 21/09/2023	Checked by B.S.F	Checked date 21/09/2023	Approved by	Approved	
Total UDL		$W = F_{swt} +$	(Fd_udl + Fi_udl)	× b <sub>panel</sub> = <b>7.959</b> kN	l/m		
Check bending stresses							
Permissible compressive stress	s in top skin	<b>σ</b> ms_top_adm =	= $\sigma_{c_{pars_{top}}} \times K_{s}$	<sub>36</sub> = <b>9.700</b> N/mm <sup>2</sup>			
Maximum bending moment		$M = W * L_e$	<sub>f</sub> ² / 8 = <b>2.875</b> k	Nm			
Compressive stress at extreme	fibre of top skir	$\sigma_{ms_{top}} = M$	$\times \overline{y} \times E_{t\_pars\_top}$	/ EI = <b>1.799</b> N/m	m²		
	PASS - Compre	essive stress at	extreme fibre	of top skin is le	ss than pern	nissible st	
Permissible bending stress in v	veb	$\sigma_{mw_adm} = \sigma$	$m  imes K_3  imes K_7  imes k$	K <sub>8</sub> = <b>6.821</b> N/mm <sup>2</sup>			
Bending stress at upper extrem	ne fibre of web	$\sigma_{mw_{top}} = M$	$\times$ ( $\overline{y}$ - t <sub>mins_top</sub> )	× E <sub>mean</sub> / EI = <b>2.0</b>	<b>33</b> N/mm <sup>2</sup>		
	PASS - Bena	ling stress at up	oper extreme	fibre of web is le	ss than pern	nissible st	
Bending stress at lower extrem	e fibre of web	$\sigma_{mw\_bot} = M$	$ imes$ (t <sub>panel</sub> - $\overline{y}$ ) $ imes$	E <sub>mean</sub> / EI = <b>4.037</b>	N/mm <sup>2</sup>		
	PASS - Bend	ding stress at lo	wer extreme	fibre of web is le	ss than pern	nissible st	
Check horizontal shear stres	ses in web me	mbers					
Permissible shear stress			K <sub>3</sub> × K <sub>8</sub> = <b>0.737</b>	N/mm <sup>2</sup>			
Maximum shear force			/ 2 = <b>6.765</b> kN				
Product of moment of elasticity	and first mome			-			
· · · · · · · · · · · · · · · · · · ·				$\mathbf{x} \times (\overline{\mathbf{y}} - \mathbf{t}_{s_{top}})^2 \times \mathbf{E}$	- mean / 2 = <b>476</b>	<b>4</b> kNm	
Maximum horizontal shear stre	\$5						
		$\tau_{max} = V \times E_s / (EI \times N \times b) = 0.155 \text{ N/mm}^2$ Maximum horizontal shear stress is less than permissible shear str					
<b>.</b>					ii periiissioi	e shear st	
Check rolling shear stress be	etween top skir						
Web contact length			9 = <b>1000</b> mm				
Permissible rolling shear stress	s at top skin			$\times K_{37} \times K_{70} = 0.55$			
Maximum rolling shear stress				$top / (EI \times b_{con}) = 0$			
PASS -	Maximum rolli	ing shear stress	s at top skin is	s less than perm	issible rollin	g shear st	
Check deflection							
Permissible deflection		$\delta_{\text{adm}}=0.00$	3 × L <sub>ef</sub> = <b>5.100</b>	mm			
Bending deflection		$\delta_{\text{bending}} = 5$	* W * L <sub>ef</sub> <sup>4</sup> / (38	4 * El) = <b>4.153</b> mi	m		
Shear deflection		$\delta_{shear} = 12^{-5}$	* W * L <sub>ef</sub> ² / (5 *	$\Sigma EA) = 0.093 mm$	n		
Total deflection		$\delta_{\text{max}} = \delta_{\text{bend}}$	ing + $\delta_{\text{shear}} = 4.2$	2 <b>46</b> mm			
		P	ASS - Total de	flection is less t	han permiss	ible deflec	
Consider medium term loads	i						
Load duration factor for timber	-	K <sub>3</sub> = <b>1.25</b>					
Load duration factor for plywoo	d	K <sub>36</sub> = <b>1.33</b>					
Total UDL		W = F <sub>swt</sub> +	$F_{d_{udl}} \times b_{panel} =$	<b>2.409</b> kN/m			
Total point load		$P = F_{i_p t} = 1.400 \text{ kN}$					
Check bending stresses		-					
Permissible compressive stress	e in top ekin	<b>.</b>	- <del>.</del>	<sub>36</sub> = <b>12.901</b> N/mm	2		
•	s in top skin			4 = <b>1.465</b> kNm			
Maximum bending moment	fibro of top akir				m <sup>2</sup>		
Comproseive atrace at autrama				/ EI = <b>0.917</b> N/m • of top skin is le		niecihla at	
Compressive stress at extreme	-ASS - Compre			-	-	11351010 31	
	voh		m ^ r\3 ^ r\7 X ľ	K <sub>8</sub> = <b>8.526</b> N/mm <sup>2</sup>			
Permissible bending stress in v					26 N/mm2		
	ne fibre of web	$\sigma_{mw\_top} = M$	$\times$ ( $\overline{y}$ - t <sub>mins_top</sub> )	$\times E_{mean} / EI = 1.0$		alaalti	
Permissible bending stress in v	ne fibre of web PASS - Bend	თ <sub>mw_top</sub> = M ling stress at up	$\times (\overline{y} - t_{mins_{top}})$	<pre>&gt; × E<sub>mean</sub> / EI = 1.0 fibre of web is le E<sub>mean</sub> / EI = 2.057</pre>	ss than pern	nissible st	

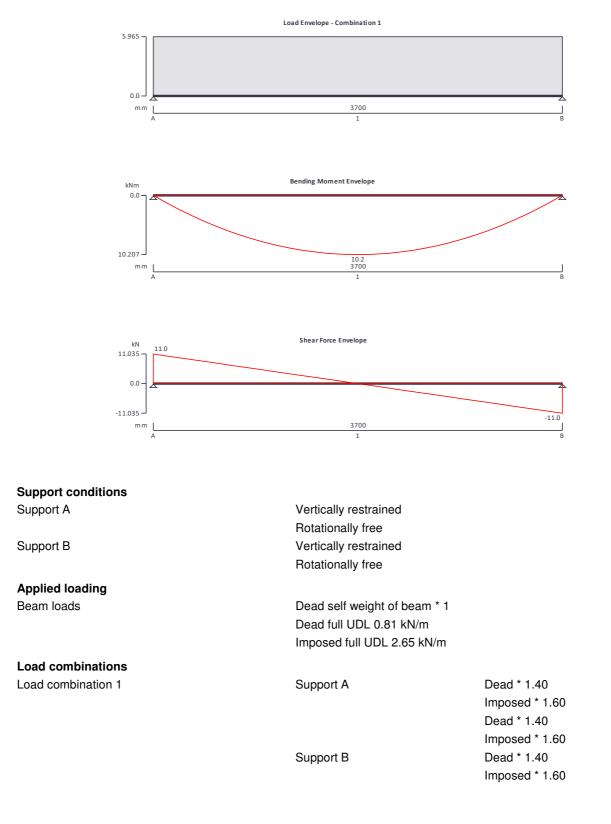
<b>Tekla</b> Tedds	Project The Quest, West Street, Harrietsham, Maidstone				Job no. 1	8629		
	Calcs for		· · ·		Start page no./Revision			
	Ар	oendix A1.1 - Stre	ess Skin Panel	Design		3		
	Calcs by U.G	Calcs date 21/09/2023	Checked by B.S.F	Checked date 21/09/2023	Approved by	Approved da		
Check horizontal shear stress	es in web me	embers						
Permissible shear stress		$\tau_{adm} = \tau \times K$	K₃ × Kଃ = <b>0.921</b>	N/mm <sup>2</sup>				
Maximum shear force	$V = W * L_{ef}$	/2+P/2= <b>2</b>	.747 kN					
Product of moment of elasticity a	and first mome	ent of area about	neutral axis					
		$E_s = EA_{top}$	$h_{xs_{top}} + N \times b$	$0 \times (\overline{y} - t_{s_{top}})^2 \times E$	mean / 2 = <b>476</b>	<b>4</b> kNm		
Maximum horizontal shear stres	S	$\tau_{max} = V \times E$	$E_s / (EI \times N \times b)$	) = <b>0.063</b> N/mm <sup>2</sup>				
	PASS	- Maximum horiz	zontal shear s	tress is less tha	n permissibl	e shear str		
Check rolling shear stress bet	ween top ski	n and web mem	bers					
Web contact length	$b_{con} = N * b$	b <sub>con</sub> = N * b = <b>1000</b> mm						
Permissible rolling shear stress	at top skin	$\tau_{r\_top\_adm} = \tau$	$\tau_{r\_top\_adm} = \tau_{r\_backs\_top} \times K_{36} \times K_{37} \times K_{70} = \textbf{0.736} \text{ N/mm}^2$					
Maximum rolling shear stress		$\tau_{r\_top\_max} = V$	$\tau_{r\_top\_max} = V \times EA_{top} \times h_{xs\_top} / (EI \times b_{con}) = 0.048 \text{ N/mm}^2$					
PASS -	Maximum rol	ling shear stress	at top skin is	s less than perm	issible rolling	g shear str		
Check deflection								
Permissible deflection		$\delta_{adm}=0.00$	3 × L <sub>ef</sub> = <b>5.100</b>	mm				
Bending deflection		$\delta_{bending} = [(5)]$	$\delta_{bending} = [(5 * W * L_{ef} / 8) + P] * L_{ef}^3 / (48 * EI) = 1.944 \text{ mm}$					
Shear deflection		$\delta_{\text{shear}} = 12$	* L <sub>ef</sub> * (W * L <sub>ef</sub> -	+ 2 * P) / (5 * ΣΕΑ	.) = <b>0.048</b> mm			
Total deflection			ing + $\delta_{\text{shear}} = 1.9$		-			
		PA	ASS - Total de	flection is less t	han permiss	ible deflect		
Splice plate design								
Load duration factor for plywood	I	K <sub>36</sub> = <b>1.00</b>						
Splice to top skin								
Compressive stress at extreme	fibre of top ski	n $\sigma_{ms_{top}} = 1.7$	<b>799</b> N/mm²					
Permissible rolling shear stress	at top skin spl	ice	m = $\tau_{r\_backs\_top} \times$	K <sub>36</sub> × K <sub>70</sub> = <b>1.10</b>	7 N/mm <sup>2</sup>			
Minimum length of splice plate f	or top okin			ıs_top / τr top_splice adr	16 mm			

Tekla. Tedds	Project The Que	Job no. 18629				
	Calcs for Append	dix A1.2 - Steel E	Beam Analysis 8	& Design	Start page no./Revision 1	
	Calcs by U.G	Calcs date 21/09/2023	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



	The Ques	st, West Street	, Harrietsham,	Maidstone	Job no.	8629
	Calcs for		Doom Analysia	<sup>9</sup> Decign	Start page no./	
			Beam Analysis	_		2
	Calcs by U.G	Calcs date 21/09/2023	Checked by B.S.F	Checked date 21/09/2023	Approved by	Approved da
Analysis results						
Maximum moment		M <sub>max</sub> = 10.2	2 kNm	$M_{min} = 0$	) kNm	
Maximum shear		V <sub>max</sub> = <b>11</b> k		V <sub>min</sub> = -		
Deflection		δ <sub>max</sub> = <b>8.5</b> r		$\delta_{\min} = 0$		
Maximum reaction at support A		R <sub>A_max</sub> = <b>11</b>		R <sub>A min</sub> =		
Unfactored dead load reaction a	at support A	$R_{A_{Dead}} = 2$		••••		
Unfactored imposed load reaction	•••	RA Imposed =				
Maximum reaction at support B		R <sub>B_max</sub> = <b>11</b>		R <sub>B_min</sub> =	11 kN	
Unfactored dead load reaction a	at support B	$R_{B_{Dead}} = 2$		· (b_11011 —		
Unfactored imposed load reaction		RB_Imposed =				
	ση αι συρροτί Β	i is_imposed =				
Section details						
Section type			02x43 (Tata S	iteel Advance)		
Steel grade		S355				
From table 9: Design strength	py					
Thickness of element		max(T, t) =				
Design strength		p <sub>y</sub> = <b>345</b> N/				
Modulus of elasticity		E = <b>205000</b>	<b>)</b> N/mm²			
=						
			2.7 -			
- 20.5 →			2./ ←			
			09.1			
Lateral restraint						
<b> 4</b>		2	09.1	nt at supports only	→	
lateral restraint		2	09.1	nt at supports only	►	
uteral restraint Effective length factors	axis	2	09.1	nt at supports only	<b>→</b>	
lateral restraint		2 Span 1 has	09.1	nt at supports only	►	
► Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a	axis	22 Span 1 has K <sub>x</sub> = <b>1.00</b> K <sub>y</sub> = <b>1.00</b>	09.1	nt at supports only	<b>→</b>	
Lateral restraint Effective length factors Effective length factor in major a	axis	22 Span 1 has K <sub>x</sub> = <b>1.00</b> K <sub>y</sub> = <b>1.00</b>	09.1 s lateral restrain 0 + 2 * D	nt at supports only	, y	
► Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a	axis -torsional bucklin	22 Span 1 has K <sub>x</sub> = 1.00 K <sub>y</sub> = 1.00 g K <sub>LT.A</sub> = 1.20 K <sub>LT.B</sub> = 1.20	09.1 5 lateral restrain 0 + 2 * D 0 + 2 * D		→	
Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for lateral	axis -torsional bucklin	22 Span 1 has K <sub>x</sub> = 1.00 K <sub>y</sub> = 1.00 g K <sub>LT.A</sub> = 1.20 K <sub>LT.B</sub> = 1.20	09.1 s lateral restrain 0 + 2 * D		►	
Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for lateral Classification of cross section	axis -torsional bucklin	22 Span 1 has K <sub>x</sub> = 1.00 K <sub>y</sub> = 1.00 g K <sub>LT.A</sub> = 1.20 K <sub>LT.B</sub> = 1.20	09.1 5 lateral restrain 0 + 2 * D 0 + 2 * D		→	
Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for lateral	axis -torsional bucklin	22 Span 1 has K <sub>x</sub> = 1.00 K <sub>y</sub> = 1.00 g K <sub>LT.A</sub> = 1.20 K <sub>LT.B</sub> = 1.20	09.1 b lateral restrain 0 + 2 * D 0 + 2 * D 1/mm <sup>2</sup> / p <sub>y</sub> ] = 0.		→	
Lateral restraint Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for lateral Classification of cross section Outstand flanges - Table 11	axis -torsional bucklin	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $g$ $K_{LT,A} = 1.20$ $K_{LT,B} = 1.20$ $\epsilon = \sqrt{275}$ N $b = D = 11^{-1}$	09.1 b lateral restrain 0 + 2 * D 0 + 2 * D 1/mm <sup>2</sup> / p <sub>y</sub> ] = 0.	89	→ y semi-compa	ct

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	Calcs for Apper	ndix A1.2 - Steel	Beam Analysis	& Design	Start page no./I	Revision 3		
	Calcs by U.G	Calcs date 21/09/2023	Checked by B.S.F	Checked date 21/09/2023	Approved by	Approved da		
Shear capacity - Section 4.2.3								
Design shear force		F <sub>v</sub> = max(a	ubs(V <sub>max</sub> ), abs(V	V <sub>min</sub> )) = <b>11</b> kN				
		(D - T - r) /	t < 70 * ε					
			Web does	not need to be a	checked for s	hear buckl		
Shear area		A <sub>v</sub> = t * D =	= <b>1410</b> mm <sup>2</sup>					
Design shear resistance			by * A <sub>v</sub> = <b>291.8</b>					
		PAS	SS - Design sh	ear resistance e	exceeds desig	gn shear fo		
Moment capacity - Section 4.2	5							
Design bending moment		M = max(a	bs(M <sub>s1_max</sub> ), ab	$s(M_{s1_min})) = 10.2$	kNm			
Effective plastic modulus - Se	ction 3.5.6							
Effective plastic modulus - cl.3.5		$S_{eff} = Z_{xx} =$	<b>41867</b> mm <sup>3</sup>					
Moment capacity low shear - cl.	4.2.5.2	$M_c = p_y * m$	nin(Z <sub>xxflange</sub> , Z <sub>xxto</sub>	<sub>be</sub> ) = <b>14.4</b> kNm				
Effective length for lateral-tors	sional bucklir	ng - Section 4.3.	5					
Effective length for lateral torsio		-	- _ <sub>s1</sub> + 2 * D = <b>46</b>	62 mm				
Slenderness ratio	iai baoini ig		$\lambda = L_E / r_{yy} = 87.278$					
			•••••					
Equivalent slenderness - Sect	101 4.3.6.7	u = <b>0.000</b>						
Buckling parameter Torsional index		u = 0.000 x = 5.115						
Moment of inertia compression	flance minor a		4					
Moment of inertia tension flange	-		/ 12 = <b>156183</b> 3	<b>34</b> mm <sup>4</sup>				
Flange ratio			$(+ I_{yt}) = 0.000$					
Flange ratio factor		k <sub>η</sub> = <b>1.000</b>						
Monosymmetry index		•	×η - 1) = <b>-1.0</b>	n				
Slenderness factor		•		$0.05 \times (\lambda / x)^2 + \psi^2$	) <sup>0.5</sup> ⊥ )µ] <sup>0.5</sup> – <b>∩</b>	583		
Ratio - cl.4.3.6.9			S <sub>xx</sub> = <b>0.495</b>		<b>γ</b> + ψ] = <b>υ</b>			
	c 7	•	$ \times \lambda \times \sqrt{[\beta w]} = 0.495 $	000				
Equivalent slenderness - cl.4.3.								
Limiting slenderness - Annex B.	2.2		$(\pi^2 \times E / p_y)^{0.5}$					
			vo allowance l	need be made fo	n ialerai-lors	IUTIAI DUCKI		
Buckling resistance moment -	Section 4.3.6							
Bending strength		$p_b = p_y = 3$						
Buckling resistance moment		-	e <sub>ff</sub> = <b>14.4</b> kNm					
		PA	ASS - Moment	capacity exceed	as design bei	nding mom		
Check vertical deflection - See	ction 2.5.2							
Consider deflection due to impo	sed loads							
		$\delta_{\rm m} = {\rm min}(1)$	$1  \text{mm}  \text{I}_{\text{st}} / 36$	0) = <b>10.278</b> mm				
Limiting deflection			l <del>+</del> mm, ∟si / 50	•)				
				nin)) = <b>8.465</b> mm				