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CALCULATIONS

Job No: 16240

Job Title: CARN GWAVEL

ISLES OF SCILLY

SCHOOL EXTENSION

Page Numbers:	Date:	Checked by:
C09 - C91	13/11/17	MSH

CALCULATIONS

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JOB NO : 16240 **SHEET NO :** 1

BRITISH STANDARDS

The British Standards listed below have been used in the preparation of these calculations. All British Standards incorporate the latest revision and amendments.

☑ **BS648 :** **1964 : Schedule of Weights of Buildings Materials.**

BS5268 : **: Structural Use of Timber.**

☑ Part 2 :2002 : Code of Practice for Permissible stress design , materials and workmanship.

☐ Part 3 :1998 : Code of Practice for Trussed Rafter Roofs.

☐ Part 4(4.1) 1978 : Fire Resistance of Timber structures.

BS5628 : **: Code of Practice for Use of Masonry.**

☐ Part 1 :1992 : Structural Use of Unreinforced Masonry.

☐ Part 2 :2000 : Structural Use of Reinforced and Prestressed Masonry.

☐ Part 3 :2001 : Materials and components, design and workmanship.

BS5950 : **: Structural Use of Steel in Building.**

☑ Part 1 : 2000 : Code of Practice for design in simple and continuous construction : hot rolled sections

BS6399 : **: Loading for Buildings.**

☑ Part 1 :1996 : Code of Practice for Dead and Imposed Loads.

☑ Part 2 : 1997 : Code of Practice for Wind Loads

☑ Part 3 :1988 : Code of Practice for Imposed Roof Loads.

☑ **BS8004 :** **1986 : Code of Practice for Foundations.**

BS8110 : **: Structural Use of Concrete.**

☑ Part 1 :1997 : Code of Practice for Design and Construction.

☐ Part 2 :1985 : Code of Practice for Special Circumstances

☑ Tick as necessary

Other British Standards used in the calculations :

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LOADING SHEET.

FLAT ROOF

	DL (kN/m²)	IL (kN/m²)
GRP SHEATHING	0.05	
18mm PLY SHEATHING	0.15	
PURLINS	0.15	
RAFTERS	0.10	
CEILING & SERVICES	0.25	
INSULATION	0.05	
18mm PLY SHEATHING	0.15	
SNOW		0.6
	0.90	0.6

TIMBER WALLS [EXTERNAL]

	DL (kN/m²)
TIMBER CLADDING (18mm THK)	0.15
BATTENS	0.10
STUD TIMBERS	0.15
9mm PLYWOOD	0.10
INSULATION	0.05
GTEC FIREBOARD	0.20

0.75 kN/m².

Made by EP	Job Title: Carn Gwavel - Isles Of Scilly		
Checked by	Job No. 16240	Sheet 3	Date Oct '17

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Loading Sheet Cont.

Slate Roof	DL (kN/m^2)	IL (kN/m^2)
Slate Tiles	0.65	
9mm Plywood Sheet	0.10	
Battons	0.05	
Timber Truss	0.14	
Insulation	0.05	
Ceiling & Services	0.25	
Snow		0.6
30° Roof Angle	2.15	
Total	2.15 kN/m^2	0.6 kN/m^2

External Wall	DL
Render	0.1
Ext. 100mm Block	2.0
Int. 100mm Block	2.0
Insulation	0.05
Plaster	0.1
Total	4.25 kN/m^2

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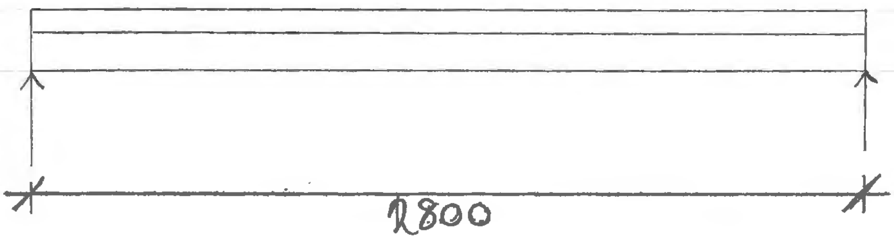
ROOF JOISTS.

LOADINGS. kN/m^2

FLAT ROOF – DL = 0.75/Cos(10) = 0.77

IL = 0.6



PROVIDE
150 x 50 C16
TIMBER JOISTS
@ 400mm c/c.

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Project				Job no.	
Carn Gwavel, Isles of Scilly				16240	
Calcs for				Start page no./Revision	
Roof Joists				5	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	13/11/2017				

TIMBER JOIST DESIGN (BS5268-2:2002)

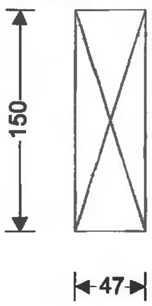
Tedds calculation version 1.1.04

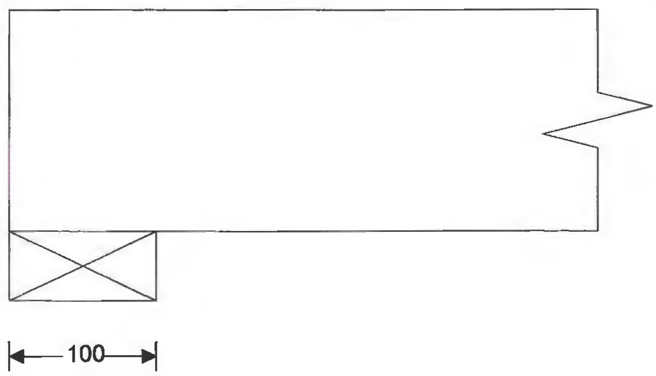
Joist details

Joist breadth	b = **47** mm	Joist depth	h = **150** mm
Joist spacing	s = **400** mm	Service class of timber	**1**
Timber strength class	**C16**		

Span details

Number of spans	N_{span} = **1**	Length of bearing	L_b = **100** mm
Clear length of span	L_{s1} = **2800** mm		





Section properties

Second moment of area	I = **13218750** mm^4	Section modulus	Z = **176250** mm^3

Loading details

Joist self weight	F_{swt} = **0.02** kN/m	Dead load	F_{d_udl} = **0.77** kN/m^2
Imposed UDL(Medium term)	F_{i_udl} = **0.60** kN/m^2		
Imposed point load (Short)	F_{i_pt} = **0.90** kN		

Consider medium term loads

Design bending moment	M = **0.558** kNm	Design shear force	V = **0.797** kN
Design support reaction	R = **0.797** kN	Design deflection	δ = **4.090** mm

Check bending stress

Permissible bending stress	σ_{m_adm} = **7.865** N/mm^2	Applied bending stress	σ_{m_max} = **3.166** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress	τ_{adm} = **0.921** N/mm^2	Applied shear stress	τ_{max} = **0.170** N/mm^2

PASS - Applied shear stress within permissible limits

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Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.025** N/mm^2 Applied bearing stress σ_{c_max} = **0.170** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **8.400** mm Actual deflection δ = **4.090** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Design bending moment M = **0.953** kNm Design shear force V = **1.361** kN

Design support reaction R = **1.361** kN Design deflection δ = **6.100** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **9.438** N/mm^2 Applied bending stress σ_{m_max} = **5.406** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.106** N/mm^2 Applied shear stress τ_{max} = **0.290** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.630** N/mm^2 Applied bearing stress σ_{c_max} = **0.290** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **8.400** mm Actual deflection δ = **6.100** mm

PASS - Actual deflection within permissible limits

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	16240	7	SEPT 17

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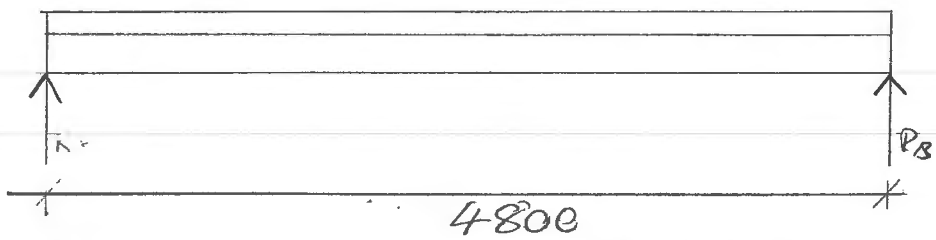
TIMBER PURLINS.

LOADINGS

kN/m

FLAT ROOF - DL $= \frac{0.75}{\cos(10)} \times \frac{4.6}{2} =$ 1.77

IL $= 0.6 \times \frac{4.6}{2}$ 1.38



Provide 3 No. 225 x 75 C24 TIMBER PURLINS

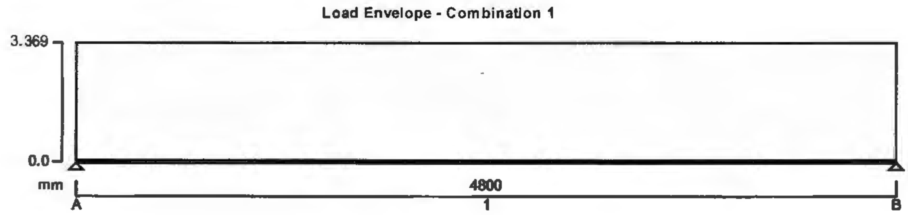
TEDDS OUTPUT.

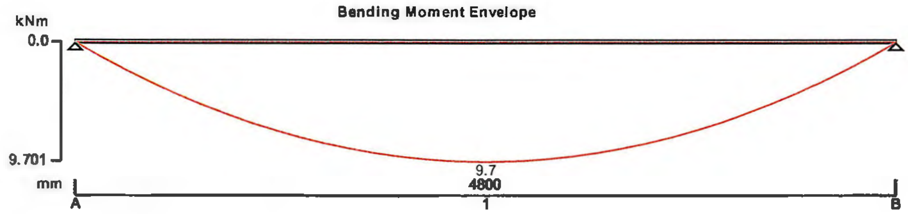
$R_A (= R_B)$ - DL = 4.75 kN

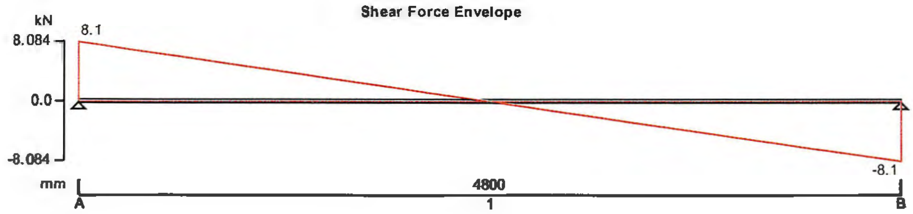
IL = 3.34 kN

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 1.770 kN/m

Imposed full UDL 1.390 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **9.701** kNm	Design shear	F = **8.084** kN
Total load on beam	W_{tot} = **16.169** kN		
Reactions at support A	R_{A_max} = **8.084** kN		R_{A_min} = **8.084** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **4.748** kN		
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **3.336** kN		

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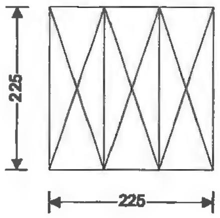
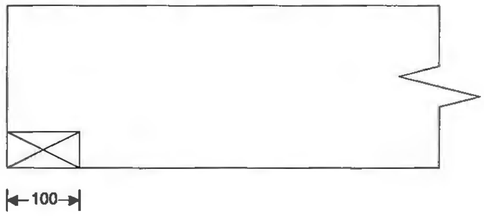
Reactions at support B R_{B_max} = **8.084** kN R_{B_min} = **8.084** kN

Unfactored dead load reaction at support B R_{B_Dead} = **4.748** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **3.336** kN





Timber section details

Breadth of section	b = **75** mm	Depth of section	h = **225** mm
Number of sections	N = **3**	Breadth of beam	b_b = **225** mm
Timber strength class	**C24**		

Member details

Service class of timber	**1**	Load duration	**Short term**
Length of bearing	L_b = **100** mm		

Underside of beam notched at all supports

Beam depth at notch h_e = **175** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **5.00** Actual depth-to-breadth ratio **1.00**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.960** N/mm^2 Applied bearing stress σ_{c_a} = **0.359** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **12.773** N/mm^2 Applied bending stress σ_{m_a} = **5.110** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain at notched support

Permissible shear stress τ_{adm} = **0.911** N/mm^2 Applied shear stress τ_a = **0.308** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **13.995** mm Total deflection δ_a = **12.936** mm

PASS - Total deflection is less than permissible deflection

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Sheet No. 10

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Date **Tuesday, 10, October, 2017**

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Wind Assessment to BS6399-2

Data Entry:-

Site Altitude	*4.750*	*m*	Reference Height (Hr)			*Diagonal Dimension (a)*		
Basic Wind Speed	*23.900*		Roof	*4.680*	*m*	*Roof*	*12.500*	*m*
Seasonal Factor (Ss)	*1.000*	*m/s*	Side Walls	*4.680*	*m*	*Sides*	*11.050*	*m*
Probability Factor(Sp)	*1.000*		Gables	*4.680*	*m*	*Gables*	*10.000*	*m*
Site ID								

Dynamic Pressure Results

Wind Direction (deg)		0	30	60	90	120	150	180	210	240	270	300	330
Direction Factor Sd		0.78	0.73	0.73	0.74	0.73	0.80	0.85	0.93	1.00	0.99	0.91	0.82
Topographic Location Factor s		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Altitude Factor Sa		1.005	1.005	1.005	1.005	1.005	1.005	1.005	1.005	1.005	1.005	1.005	1.005
Effective Height He (m)	Roof	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68
	Sides	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68
	Gable	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.68
Terrain & Building Factor Sb	Roof	1.611	1.591	1.591	1.591	1.611	1.611	1.632	1.632	1.632	1.611	1.632	1.632
	Sides	1.611	1.591	1.591	1.591	1.611	1.611	1.632	1.632	1.632	1.611	1.632	1.632
	Gable	1.611	1.591	1.591	1.591	1.611	1.611	1.632	1.632	1.632	1.611	1.632	1.632
Site Wind Speed Vs (m/s)	Roof	18.73	17.53	17.53	17.77	17.53	19.21	20.41	22.33	24.01	23.77	21.85	19.69
	Sides	18.73	17.53	17.53	17.77	17.53	19.21	20.41	22.33	24.01	23.77	21.85	19.69
	Gable	18.73	17.53	17.53	17.77	17.53	19.21	20.41	22.33	24.01	23.77	21.85	19.69
Effective Wind Speed Ve (m/s)	Roof	30.17	27.89	27.89	28.27	28.24	30.94	33.31	36.45	39.19	38.29	35.67	32.14
	Sides	30.17	27.89	27.89	28.27	28.24	30.94	33.31	36.45	39.19	38.29	35.67	32.14
	Gable	30.17	27.89	27.89	28.27	28.24	30.94	33.31	36.45	39.19	38.29	35.67	32.14
Dynamic Pressure q (kN/m^2)	Roof	0.558	0.477	0.477	0.490	0.489	0.587	0.680	0.814	0.942	0.899	0.780	0.633
	Sides	0.558	0.477	0.477	0.490	0.489	0.587	0.680	0.814	0.942	0.899	0.780	0.633
	Gable	0.558	0.477	0.477	0.490	0.489	0.587	0.680	0.814	0.942	0.899	0.780	0.633
Size Effect Factor Cae	Roof	0.949	0.938	0.938	0.938	0.949	0.949	0.949	0.949	0.949	0.949	0.949	0.949
	Sides	0.955	0.945	0.945	0.945	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955
	Gable	0.960	0.950	0.950	0.950	0.960	0.960	0.960	0.960	0.960	0.960	0.960	0.960

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Designer
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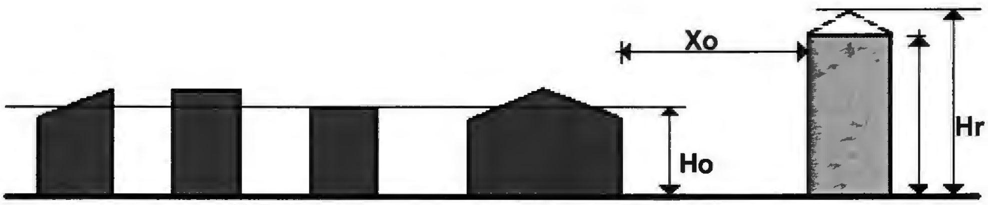
Wind Assessment to BS6399-2



Ho= Obstruction Height or average height
of roof tops upwind of building under consideration

Hr= Maximum height of building (REFERENCE HEIGHT)

Xo= Distance to obstruction

Terrain Data

Wind Direction (deg)	0	30	60	90	120	150	180	210	240	270	300	330
Smallest Obstruction Height Ho(m)	0	0	0	0	0	0	0	0	0	0	0	0
Distance to Obstruction Xo(m)	20	20	20	20	20	20	20	20	20	20	20	20
Upwind distance to sea (km)	2	3	3	3	2	2	0	0	0	2	0	0
Upwind Distance from Edge of Town(km)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

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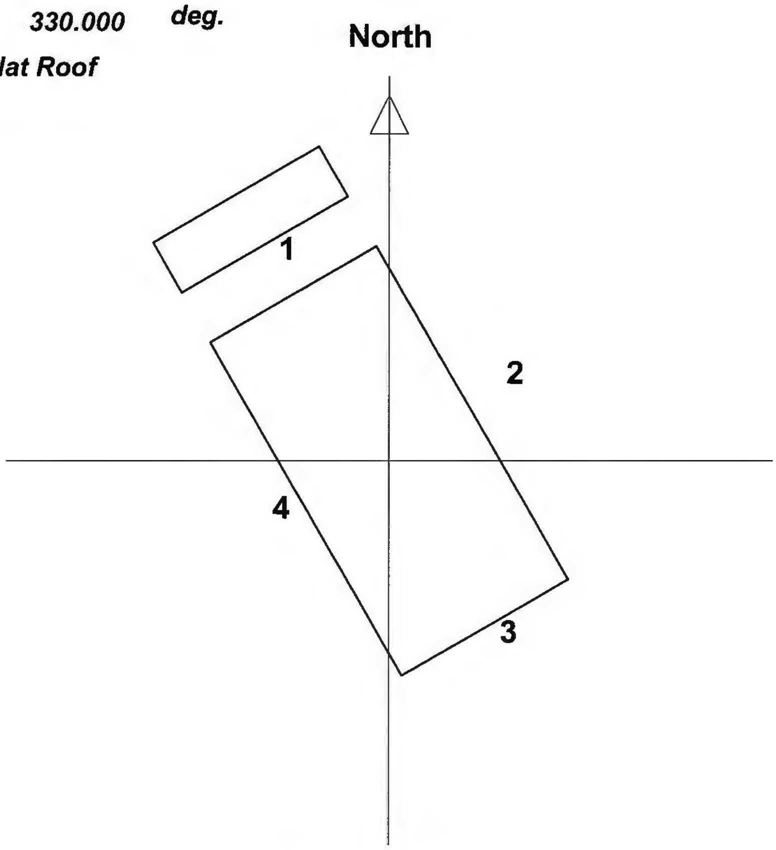
Wind Assessment to BS6399-2

Roof Orientation *330.000* *deg.*
Roof Type *Flat Roof*



Dynamic Pressure kN/m²

Orthogonal Direction 1	Orthogonal Direction 2	Orthogonal Direction 3	Orthogonal Direction 4
0.780	0.490	0.680	0.942

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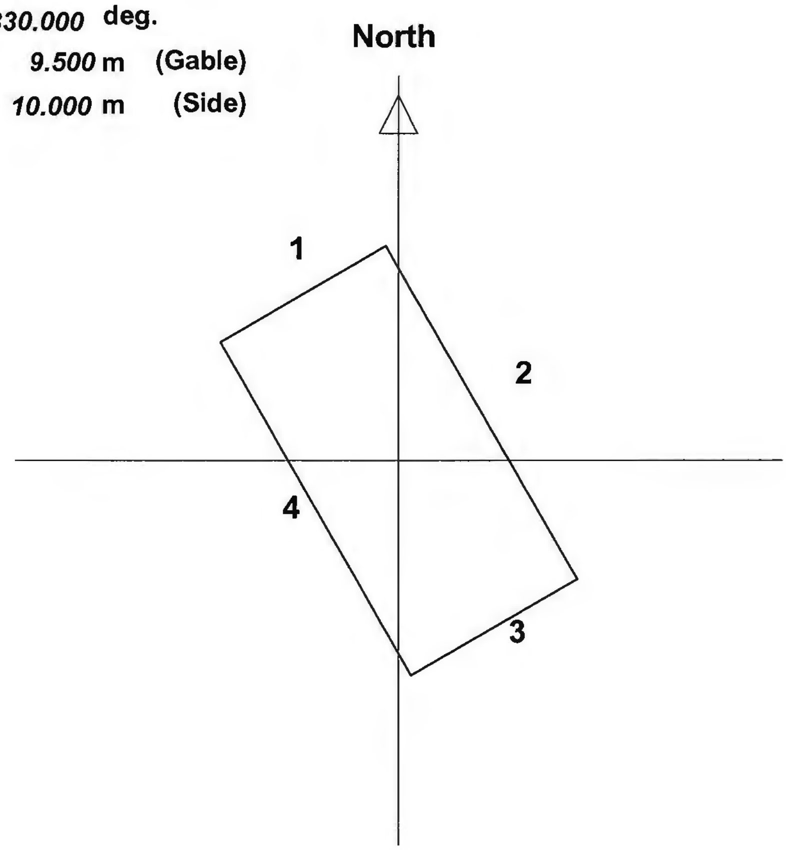
Wind Assessment to BS6399-2

Wall Orientation	330.000	deg.	
Short Face 1 or 3	9.500	m	(Gable)
Long Face 2 or 4	10.000	m	(Side)



Dynamic Pressure kN/m²

Face 1	Face 2	Face 3	Face 4
0.780	0.490	0.680	0.942

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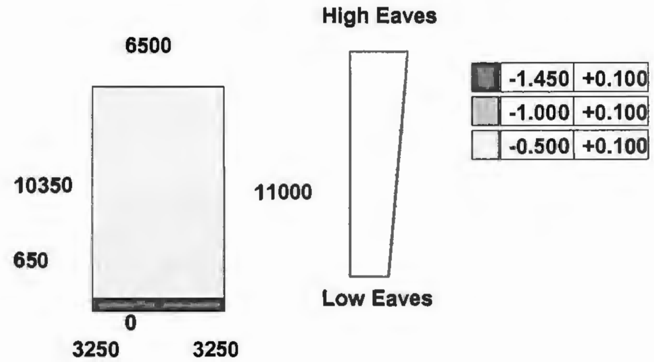
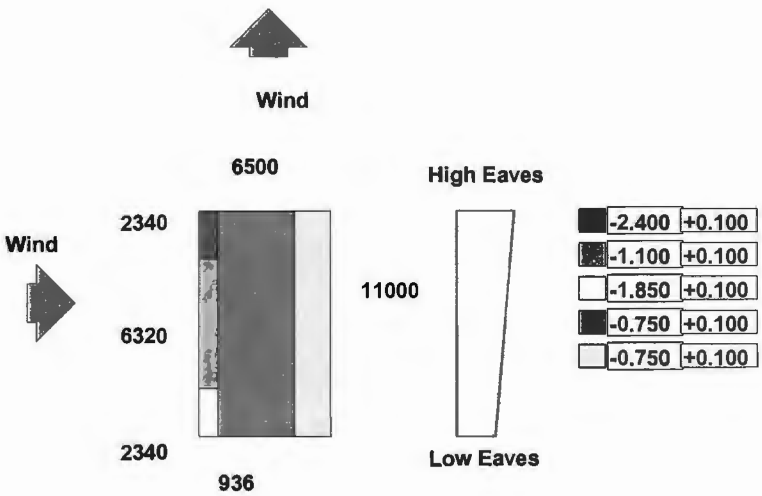
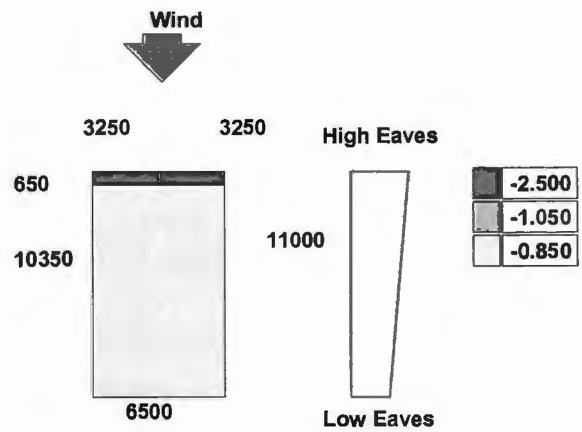
Wind Analysis to BS6399-2 - Cpe Results for Roofs

DATA ENTRY:-			
Width of Bay	11.000 m	**Reference Height**	4.680m
Length of Bay	6.500 m	**Roof Pitch**	10.000 deg.
Roof Type	Monopitch roof		
Bay type	Single bay building		







NB: All dimensions are in millimetres, except Cpe values

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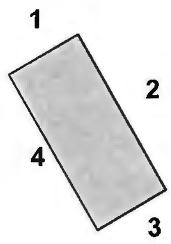
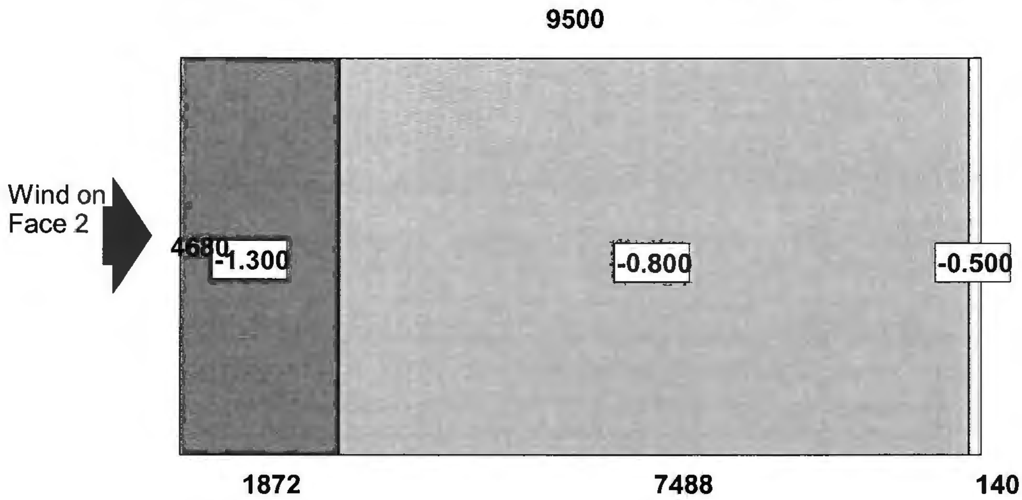
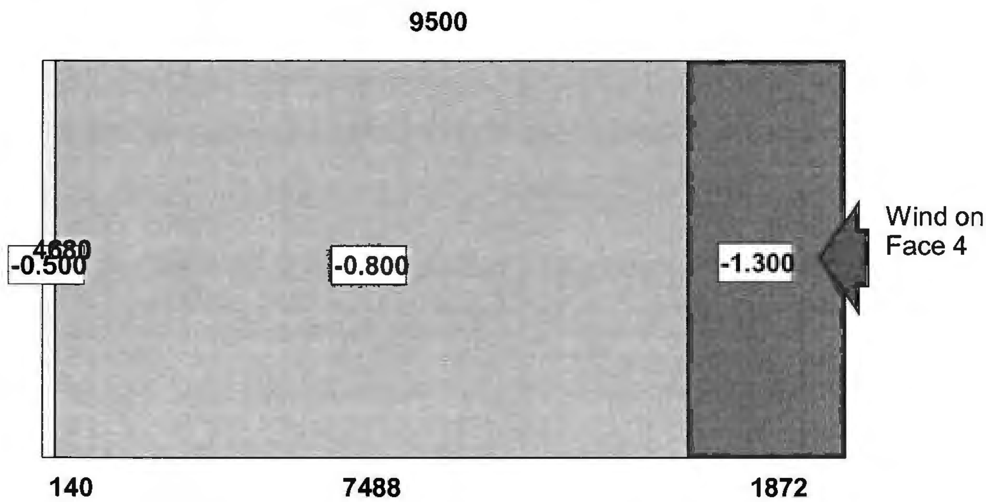
Wind Analysis to BS6399-2 - Cpe Results for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 1 (Gable)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>-Cpe On Reference Face1</u>
+Cpe On Reference Face =+0.755
-Cpe On Opposite(Leeward) Face =0.500

NB: All dimensions are in millimetres, except Cpe values

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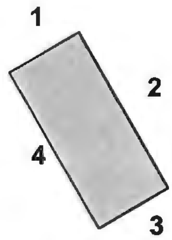
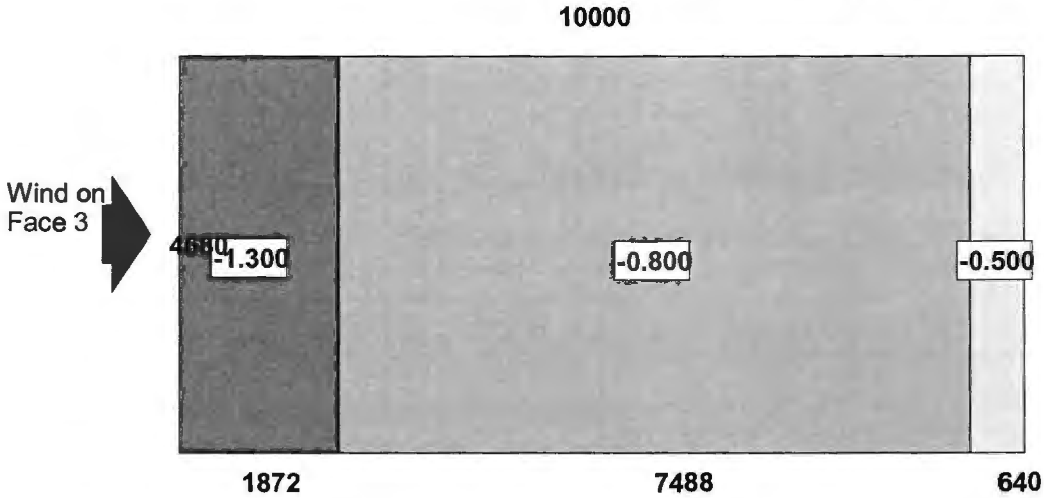
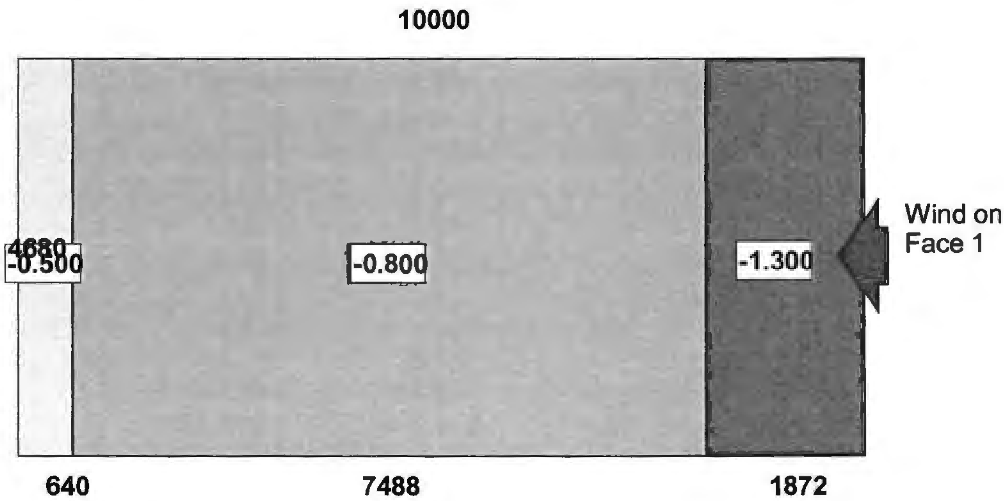
Wind Analysis to BS6399-2 - Cpe Results for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 2 (Side)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>-Cpe On Reference Face2</u>
+Cpe On Reference Face =+0.764
-Cpe On Opposite(Leeward) Face =0.500

NB: All dimensions are in millimetres, except Cpe values

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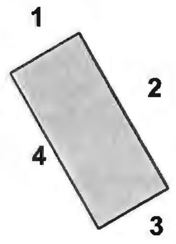
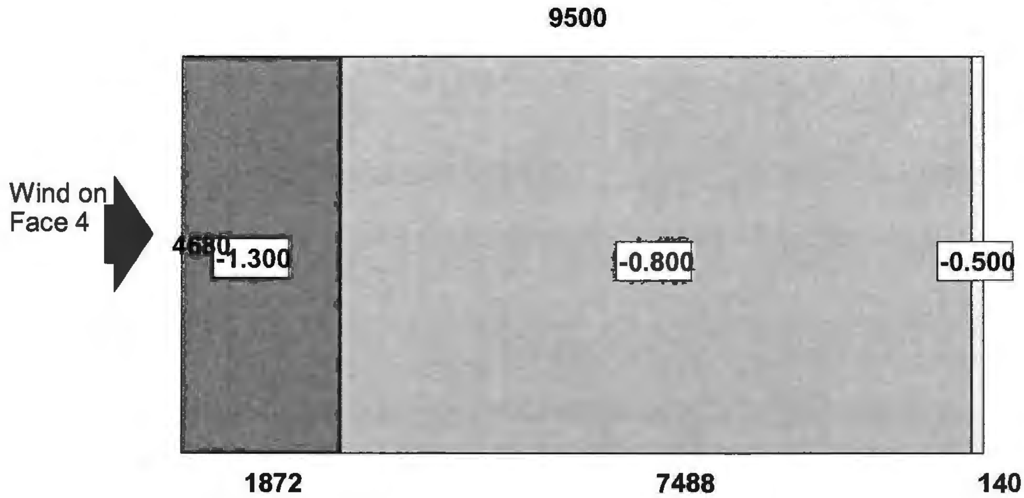
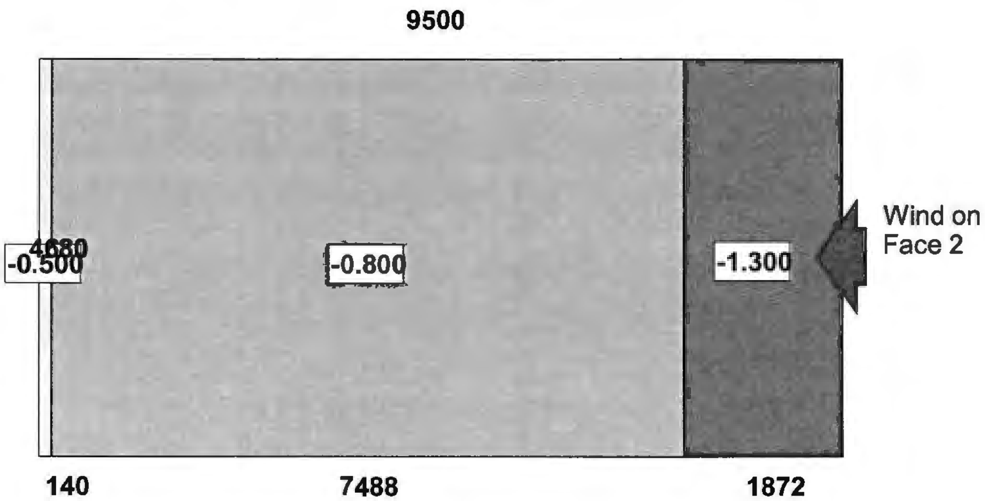
Wind Analysis to BS6399-2 - Cpe Results for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 3 (Gable)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>-Cpe On Reference Face3</u>
+Cpe On Reference Face =+0.755
-Cpe On Opposite(Leeward) Face =0.500

NB: All dimensions are in millimetres, except Cpe values

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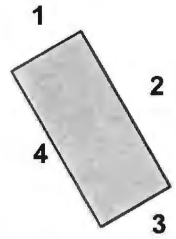
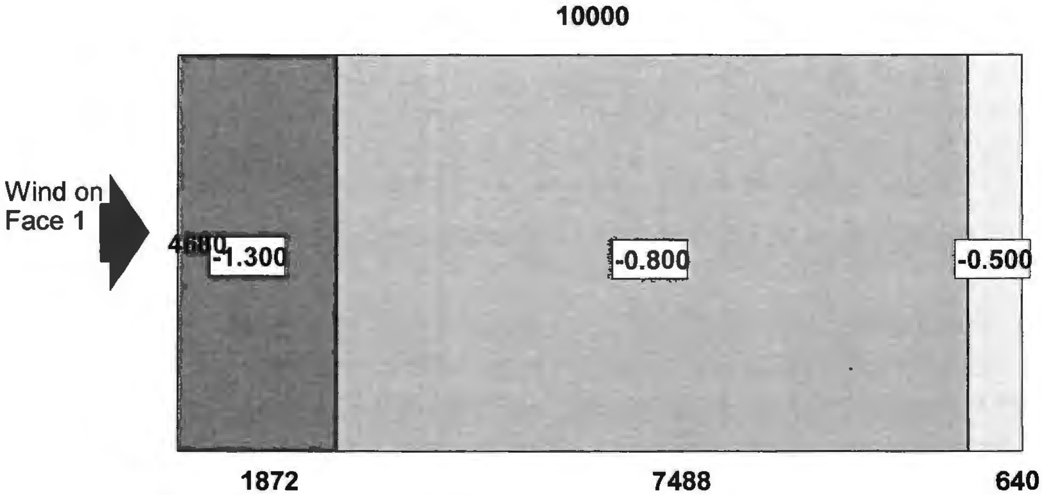
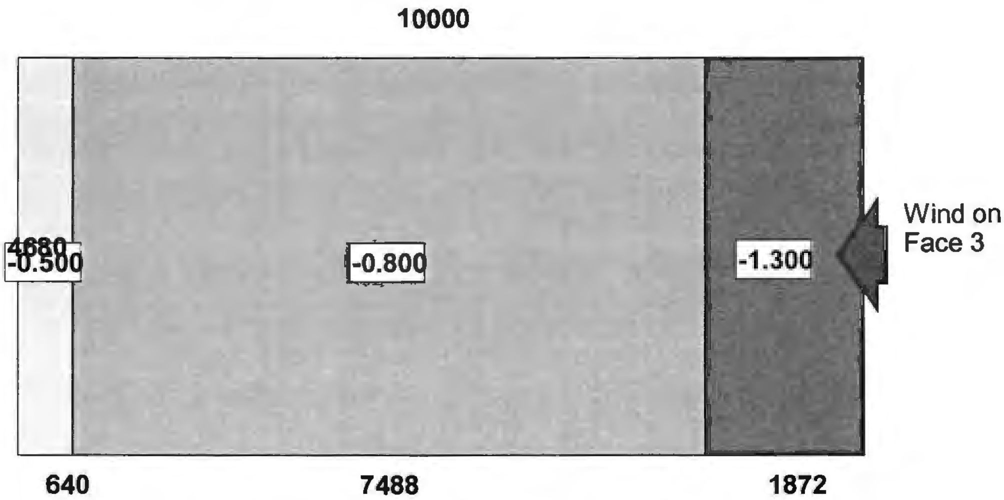
Wind Analysis to BS6399-2 - Cpe Results for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 4 (Side)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>-Cpe On Reference Face4</u>
+Cpe On Reference Face =+0.764
-Cpe On Opposite(Leeward) Face =0.500

NB: All dimensions are in millimetres, except Cpe values

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Wind Assessment to BS6399-2

Data Entry:-

Condition		*Without Dominant Opening*		**Reference Height (Hr)**	
Internal Volume		*177.000*	m^3	Roof	4.680 m
				Side Walls	4.680 m
				Gables	4.680 m
Cpi	-ve	*0.300*			
	+ve	*0.200*			

Size Effect Factors For Cpi

Wind Direction (deg)	0	30	60	90	120	150	180	210	240	270	300	330
Size Effect Factor Cai (Roof)	0.849	0.819	0.819	0.819	0.849	0.849	0.849	0.849	0.849	0.849	0.849	0.849

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Wind Assessment to BS6399-2

Data Entry:-

Condition	*Without Dominant Opening*	
Internal Volume	*177.000*	m^3
Cpi -ve	*0.300*	
Cpi +ve	*0.000*	

Reference Height (Hr)

Roof	4.680	m
Side Walls	4.680	m
Gables	4.680	m

Size Effect Factors For Cpi

Wind Direction (deg)		0	30	60	90	120	150	180	210	240	270	300	330
Size Effect Factor Cai (Walls)	Side	0.849	0.819	0.819	0.819	0.849	0.849	0.849	0.849	0.849	0.849	0.849	0.849
	Gable	0.849	0.819	0.819	0.819	0.849	0.849	0.849	0.849	0.849	0.849	0.849	0.849

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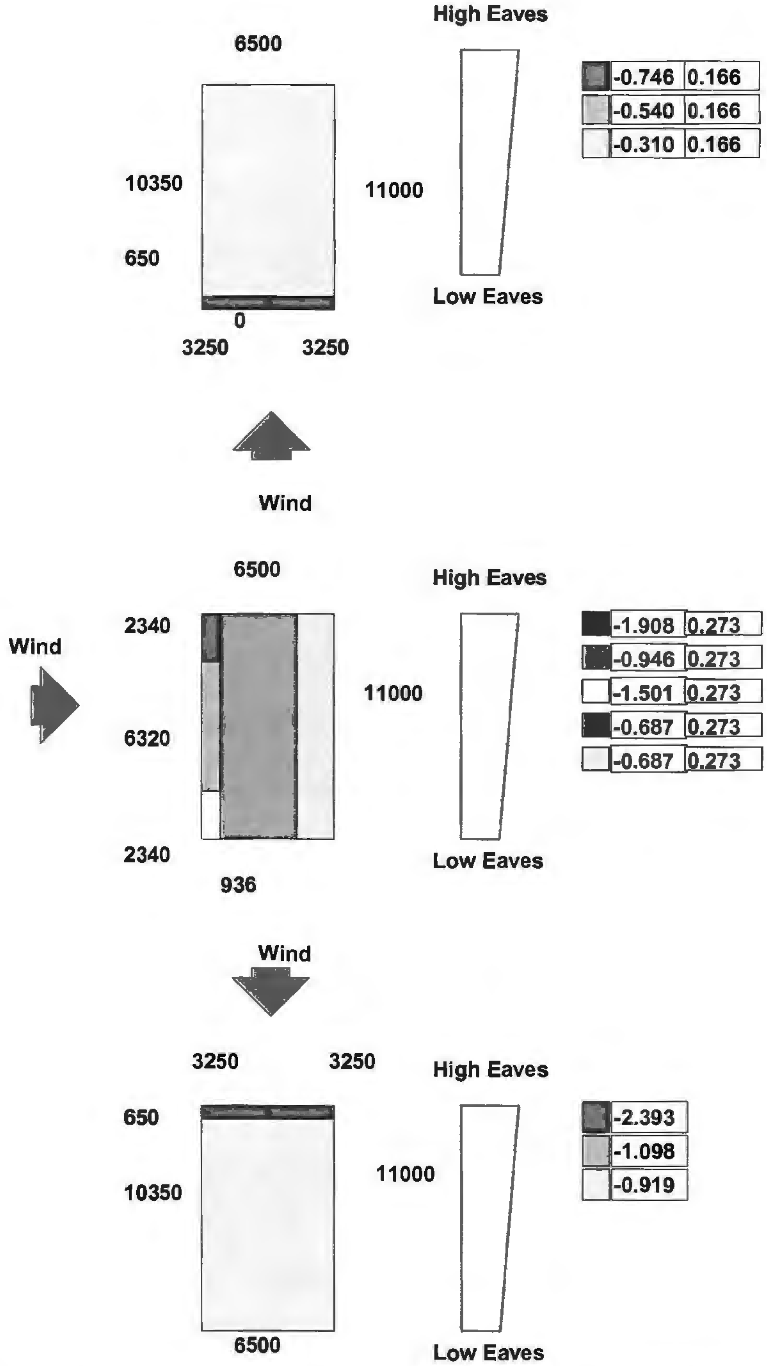
Wind Analysis to BS6399-2 - Wind Loads for Roofs

DATA ENTRY:-

Width of Bay	11.000 m	**Reference Height**	4.680m
Length of Bay	6.500 m	**Roof Pitch**	10.000 deg.
Roof Type	Monopitch roof		
Bay type	Single bay building		



NB: All dimensions are in millimetres and wind loads in kN/m^2

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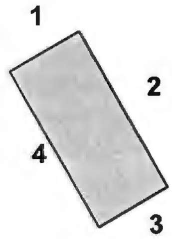
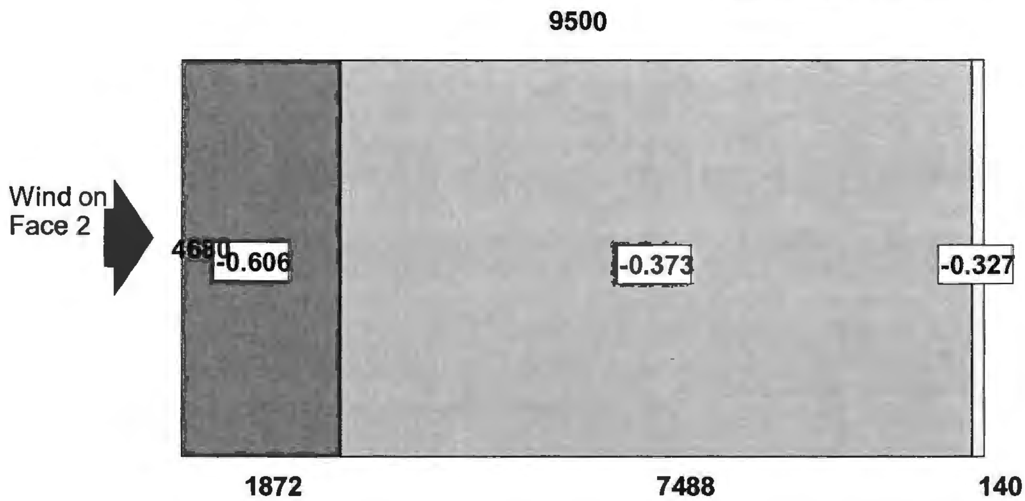
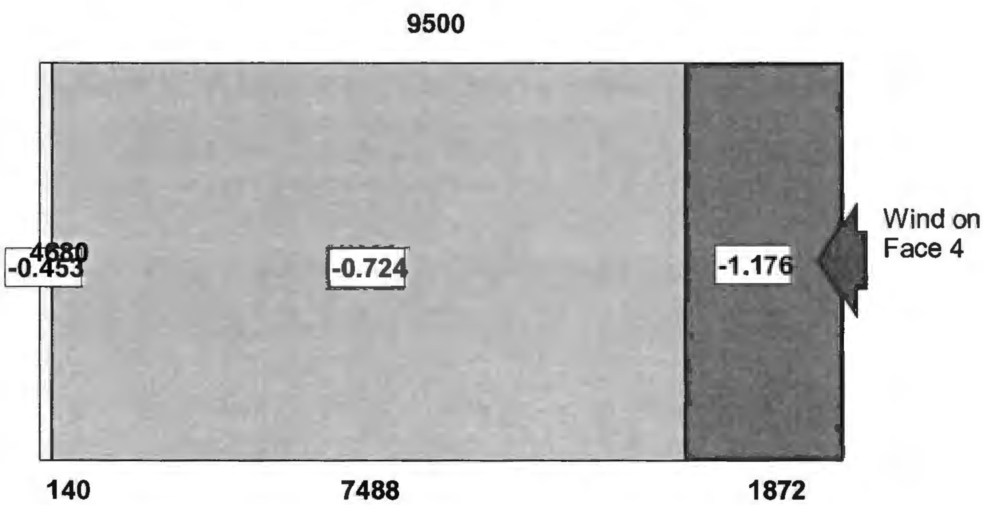
Wind Analysis to BS6399-2 - Wind Loads for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 1 (Gable)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>Suction(kN/m²) On Reference Face1</u>
Pressure (kN/m²) On Reference Face = +0.764
Pressure/Suction (kN/m²) On Opposite(Leeward) Face = -0.375

Note: The above loads are not applicable to parapets which must be designed separately.

NB: All dimensions are in millimetres and wind loads in kN/m²

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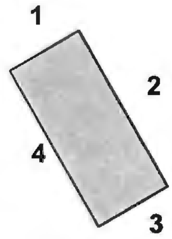
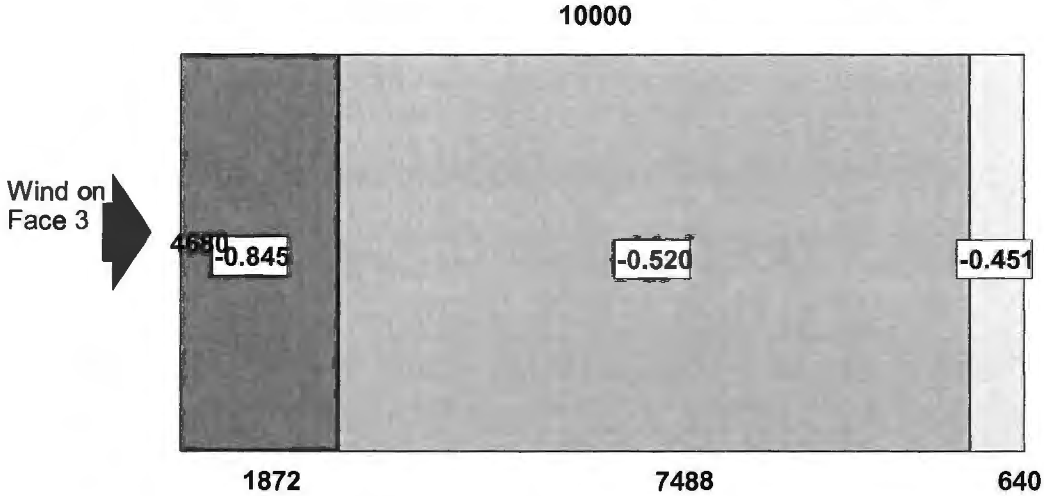
Wind Analysis to BS6399-2 - Wind Loads for Walls

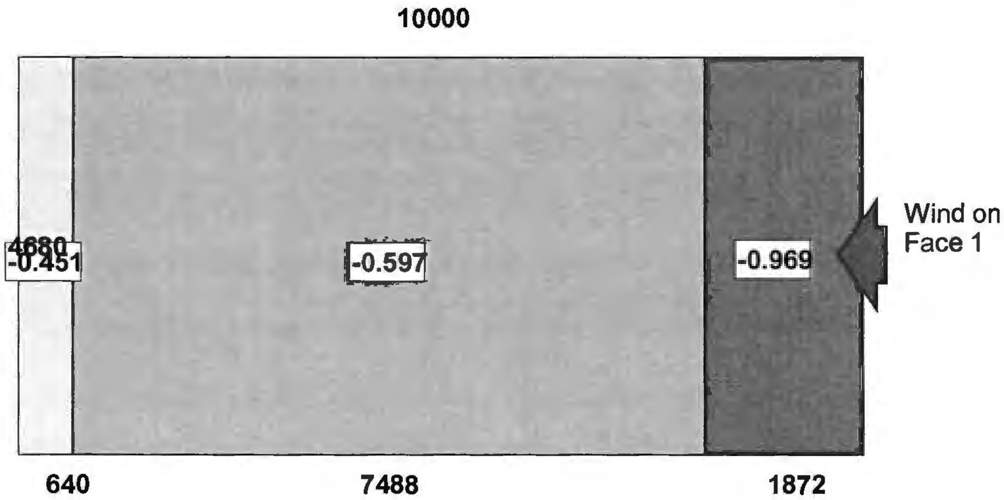
DATA ENTRY:-

Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 2 (Side)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





Suction(kN/m²) On Reference Face2
Pressure (kN/m²) On Reference Face = +0.474
Pressure/Suction (kN/m²) On Opposite(Leeward) Face = -0.232

Note: The above loads are not applicable to parapets which must be designed separately.

NB: All dimensions are in millimetres and wind loads in kN/m²

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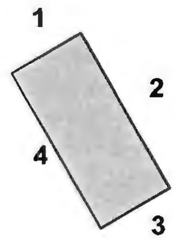
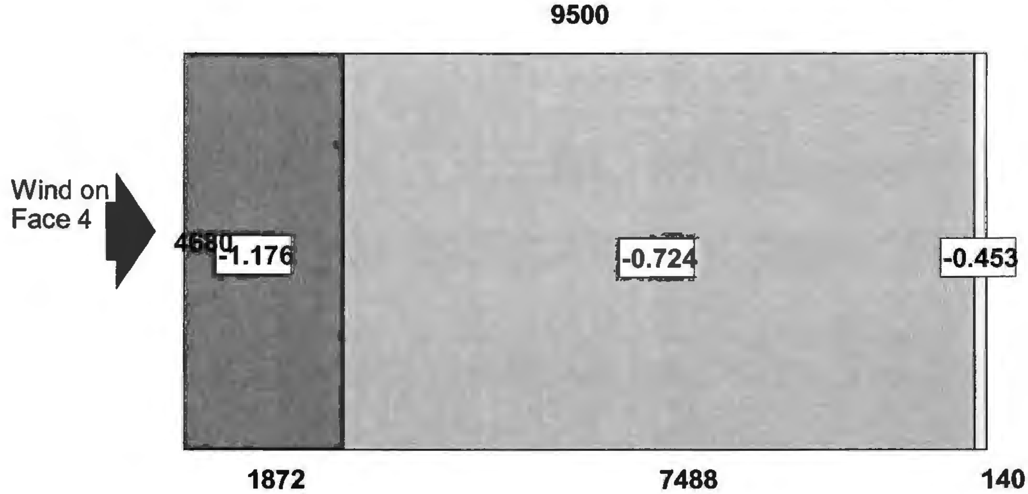
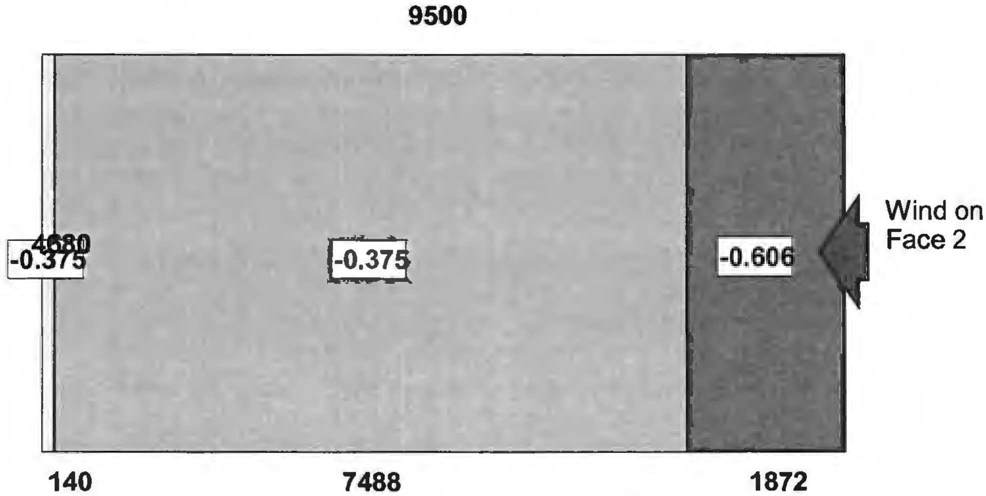
Wind Analysis to BS6399-2 - Wind Loads for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 3 (Gable)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





<u>Suction(kN/m²) On Reference Face3</u>
Pressure (kN/m²) On Reference Face = +0.667
Pressure/Suction (kN/m²) On Opposite(Leeward) Face = -0.327

Note: The above loads are not applicable to parapets which must be designed separately.

NB: All dimensions are in millimetres and wind loads in kN/m²

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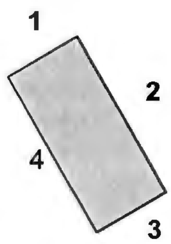
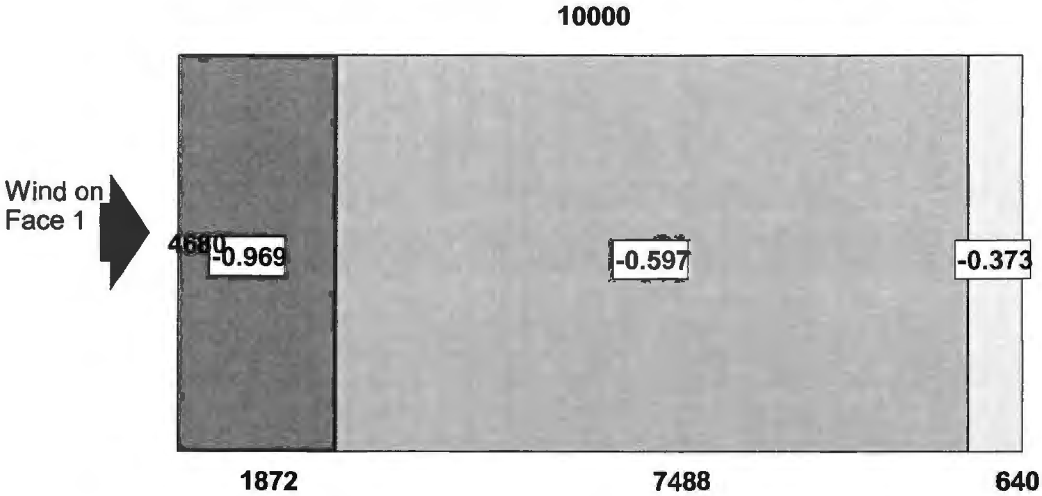
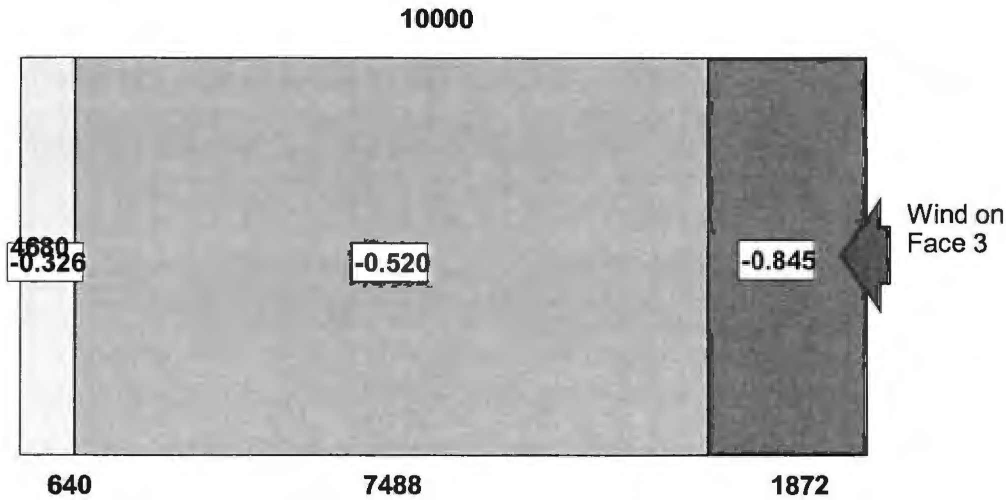
Wind Analysis to BS6399-2 - Wind Loads for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 4 (Side)
Reference Height	4.680 m
Gap Between Buildings	0.000 m



Reference Face





Suction(kN/m²) On Reference Face4
Pressure (kN/m²) On Reference Face = +0.927
Pressure/Suction (kN/m²) On Opposite(Leeward) Face = -0.451

Note: The above loads are not applicable to parapets which must be designed separately.

NB: All dimensions are in millimetres and wind loads in kN/m²

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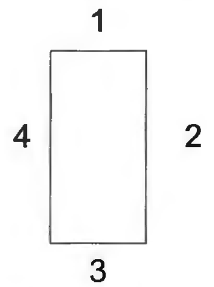
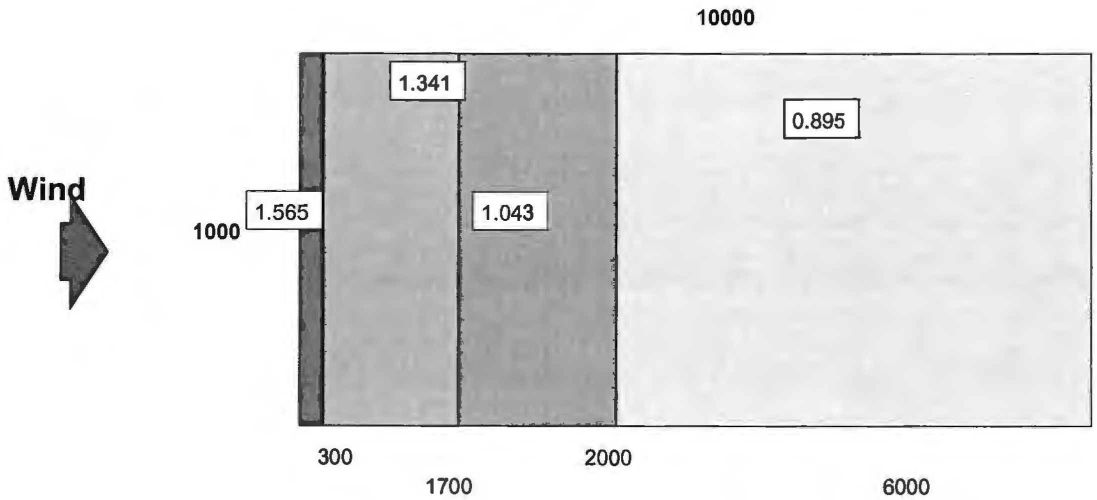
Wind Analysis to BS6399-2 - Wind Loads for Parapets

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	10.000 m
Reference Face	Face 4 (Side)
Reference Height	4.680 m
Solidity	1.000
Parapet Height	1.000 m
Corner Condition	Without return corners



Reference Face



Suction/Pressure(kN/m²) On Reference Face

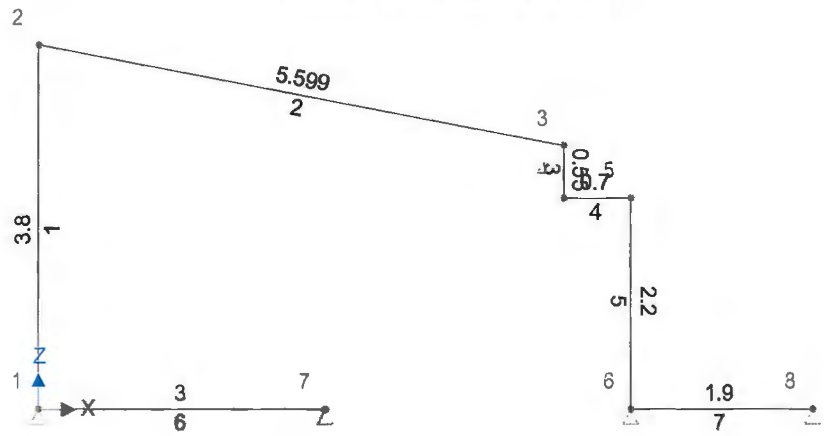
NB: All dimensions are in millimetres and wind loads in kN/m²

ANALYSIS

Tedds calculation version 1.0.18

Geometry

Geometry (m) - Steel (BS5950)



Loading

Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
Def (Strength)	1.00	1.00	1.00	1.00
Strength 1 (Strength)	1.40	1.40	1.60	
Strength 2 (Strength)	1.40	1.40		1.40
Strength 3 (Strength)	1.20	1.20	1.20	1.20

Node loads

Node	Load case	Force X (kN)	Force Z (kN)	Moment (kNm)
2	Wind	9.6	0	0

Element point loads

Element	Load case	Position Type	Position Start	Load (kN)	Orientation
2	Permanent	Absolute	1 m	3.9	GlobalZ
2	Permanent	Absolute	3.4 m	7.8	GlobalZ
2	Permanent	Absolute	5.5 m	7.8	GlobalZ
2	Imposed	Absolute	1 m	3	GlobalZ
2	Imposed	Absolute	3.4 m	6	GlobalZ
2	Imposed	Absolute	5.5 m	6	GlobalZ

Element UDL loads

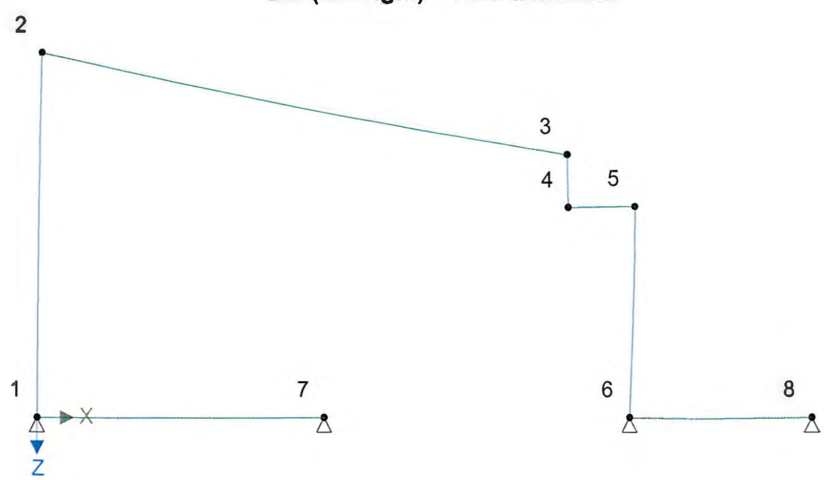
MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly			Job no.: 16240	
	Calcs for: Extension Frame (Base Plate Resistance Considered)			Start page no./Revision: 28	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

Element	Load case	Position			Load	Orientation
		Type	Start	End	(kN/m)	
1	Permanent	Ratio	0	1	0	GlobalZ

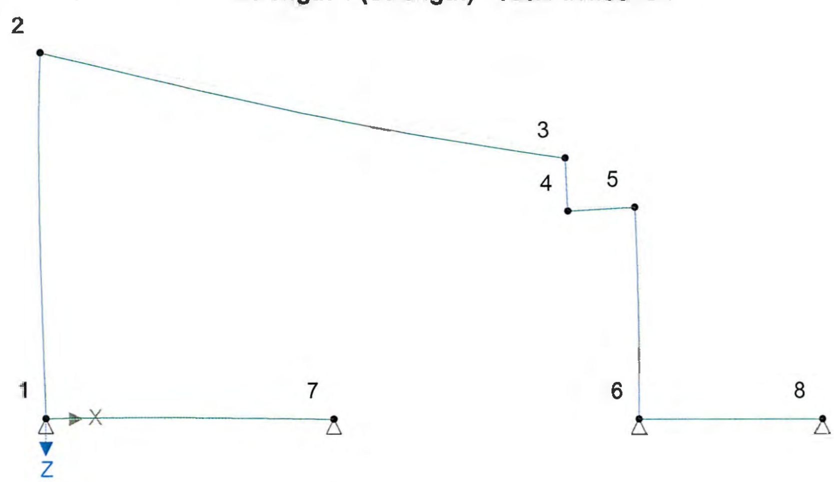
Results

Total deflection

Def (Strength) - Total deflection



Strength 1 (Strength) - Total deflection

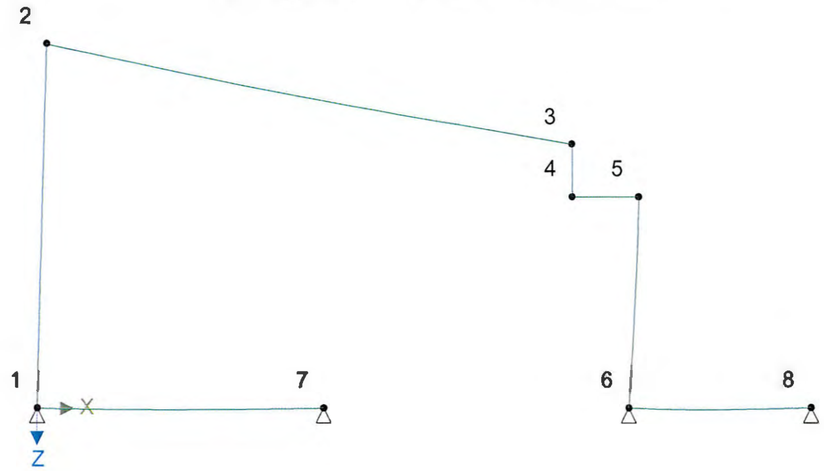


MBA CONSULTING

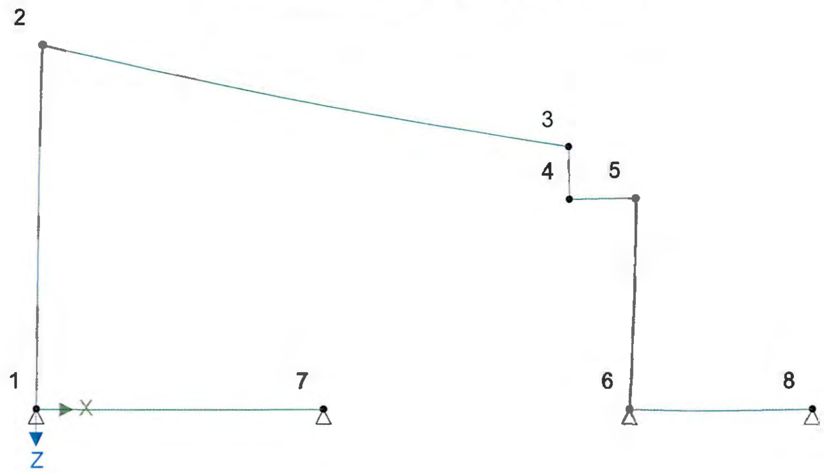
Tel: [illegible] 260962
Fax: [illegible]

Project				Job no.	
Carn Gwavel, Isles of Scilly				16240	
Calcs for				Start page no./Revision	
Extension Frame (Base Plate Resistance Considered)				29	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	13/11/2017				

Strength 2 (Strength) - Total deflection



Strength 3 (Strength) - Total deflection



Node deflections

Load combination: Def (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.03075	
2	4.7	0	0.14889	
3	4.4	1.1	-0.12797	
4	5.5	1.1	-0.10494	
5	5.5	0.1	-0.0182	
6	0	0	0.18756	
7	0	0	-0.02906	
8	0	0	-0.09674	

Load combination: Strength 1 (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	-0.19915	
2	-6.8	0.1	0.18289	
3	-7.6	3.9	-0.2582	
4	-4.9	3.9	-0.30471	
5	-4.9	0.1	-0.2757	
6	0	0	-0.04751	
7	0	0	0.08014	
8	0	0	0.01872	

Load combination: Strength 2 (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.12346	
2	9.3	0	0.13935	
3	9.2	0	-0.08049	
4	9.7	0	-0.02941	
5	9.7	0.1	0.08197	
6	0	0	0.28435	
7	0	0	-0.0808	
8	0	0	-0.14626	

Load combination: Strength 3 (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.0369	
2	5.6	0.1	0.17867	
3	5.3	1.3	-0.15357	
4	6.6	1.3	-0.12593	
5	6.6	0.1	-0.02184	
6	0	0	0.22507	
7	0	0	-0.03487	
8	0	0	-0.11609	

Element end forces

Load combination: Def (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.8	1	-11.7	1.3	0.1
		2	10.3	-1.3	-4.9
2	5.599	2	-8.7	-12.1	4.9
		3	15.6	-24.3	-6.3
3	0.55	3	-26.8	-10.8	6.3
		4	27	10.8	-0.3

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
4	0.7	4	-10.8	27	0.3
		5	10.8	-27.2	-19.3
5	2.2	5	-27.2	-10.8	19.3
		6	28	10.8	4.5
6	3	1	0	-0.6	-0.1
		7	0	-0.6	0
7	1.9	6	0	2	-4.5
		8	0	-2.7	0

Load combination: Strength 1 (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.8	1	-23.9	6.2	-3.8
		2	21.9	-6.2	-19.7
2	5.599	2	-1.9	-22.7	19.7
		3	12.2	-31.2	5
3	0.55	3	-32.9	-6.2	-5
		4	33.2	6.2	8.4
4	0.7	4	-6.2	33.2	-8.4
		5	6.2	-33.5	-15
5	2.2	5	-33.5	-6.2	15
		6	34.7	6.2	-1.4
6	3	1	0	-2.1	3.8
		7	0	0.4	0
7	1.9	6	0	-1.3	1.4
		8	0	0.2	0

Load combination: Strength 2 (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.8	1	-8.1	-0.5	1.3
		2	6.2	0.5	0.7
2	5.599	2	-11.4	-8.5	-0.7
		3	17.2	-21.8	-10.9
3	0.55	3	-24.7	-12.8	10.9
		4	24.9	12.8	-3.8
4	0.7	4	-12.8	24.9	3.8
		5	12.8	-25.3	-21.4
5	2.2	5	-25.3	-12.8	21.4
		6	26.4	12.8	6.8
6	3	1	0	-0.4	-1.3
		7	0	-1.3	0
7	1.9	6	0	3.1	-6.8
		8	0	-4.1	0

Load combination: Strength 3 (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.8	1	-14	1.5	0.1

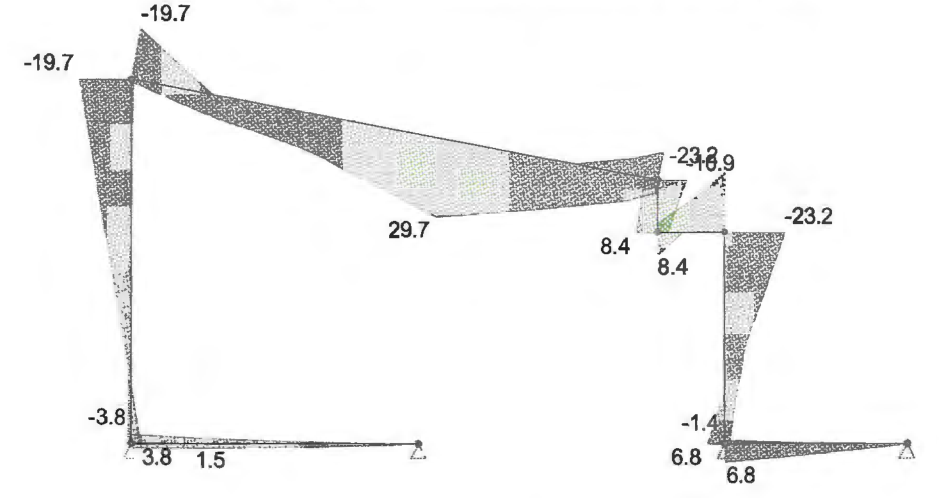
MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly	Job no.: 16240
	Calcs for: Extension Frame (Base Plate Resistance Considered)	Start page no./Revision: 32

Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	13/11/2017				

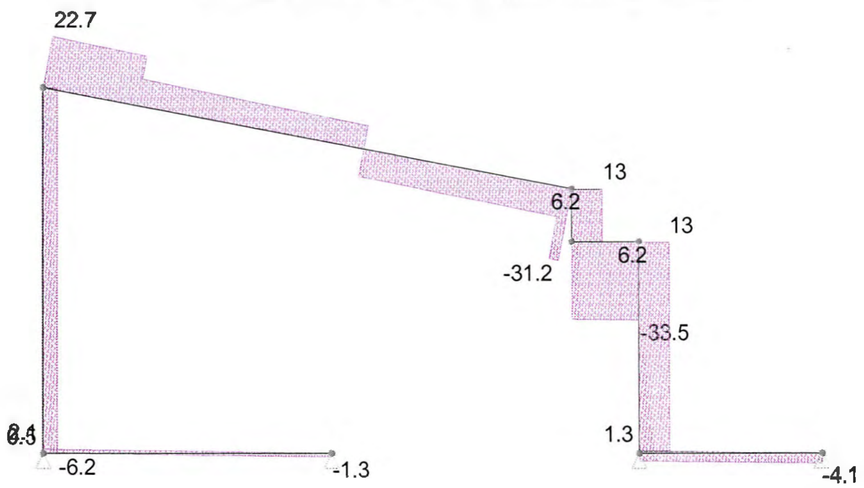
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
		2	12.3	-1.5	-5.8
2	5.599	2	-10.4	-14.5	5.8
		3	18.8	-29.1	-7.5
3	0.55	3	-32.1	-13	7.5
		4	32.3	13	-0.4
4	0.7	4	-13	32.3	0.4
		5	13	-32.7	-23.2
5	2.2	5	-32.7	-13	23.2
		6	33.6	13	5.4
6	3	1	0	-0.7	-0.1
		7	0	-0.7	0
7	1.9	6	0	2.4	-5.4
		8	0	-3.3	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Element results

Envelope - Strength combinations

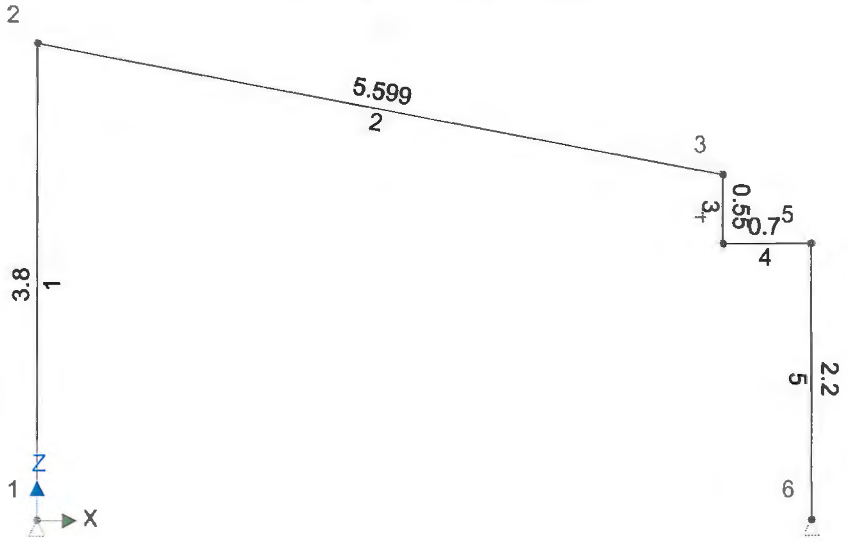
Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	-6.2	0	3.8	3.8	-19.7
2	5.599	-31.2	3.4	29.7	0	-19.7
3	0	13	0.55	8.4	0	-10.9
4	0.7	-33.5	0	8.4	0.7	-23.2
5	0	13	2.2	6.8	0	-23.2
6	0	2.1	0.668	1.5	0	-3.8
7	1.9	-4.1	0	6.8	0	-1.4

ANALYSIS

Tedds calculation version 1.0.18

Geometry

Geometry (m) - Steel (BS5950)



Loading

Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
Def (Strength)	1.00	1.00	1.00	1.00
Strength 1 (Strength)	1.40	1.40	1.60	
Strength 2 (Strength)	1.40	1.40		1.40
Strength 3 (Strength)	1.20	1.20	1.20	1.20
Uplift (Strength)	1.00	1.00		1.40

Node loads

Node	Load case	Force X (kN)	Force Z (kN)	Moment (kNm)
2	Wind	9.6	0	0

Element point loads

Element	Load case	Position Type	Position Start	Load (kN)	Orientation
2	Permanent	Absolute	1 m	3.28	GlobalZ
2	Permanent	Absolute	3.4 m	3.28	GlobalZ
2	Permanent	Absolute	5.5 m	3.28	GlobalZ
2	Imposed	Absolute	1 m	2.7	GlobalZ
2	Imposed	Absolute	3.4 m	2.7	GlobalZ
2	Imposed	Absolute	5.5 m	2.7	GlobalZ

Element UDL loads

Element	Load case	Position			Load	Orientation
		Type	Start	End	(kN/m)	
1	Permanent	Ratio	0	1	0	GlobalZ

Results

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
Self Weight	0	5.2
Permanent	0	9.8
Imposed	0	8.1
Wind	-9.6	0

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.3	2.8	0
6	-0.3	2.3	0

Load case: Permanent

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.9	4.7	0
6	-0.9	5.1	0

Load case: Imposed

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.8	3.9	0
6	-0.8	4.2	0

Load case: Wind

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-3.2	-5.9	0
6	-6.4	5.9	0

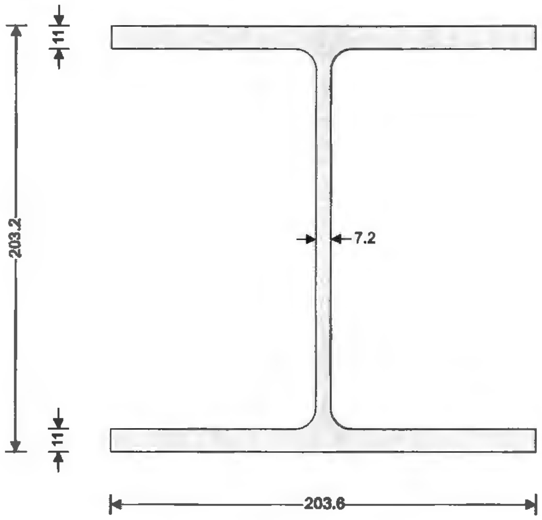
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.05

Section details

Section type **UKC 203x203x46 (Tata Steel Advance)** Steel grade **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **17.1** kN Design shear resistance $P_{y,v}$ = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Moment capacity - Section 4.2.5

Design bending moment M = **16.5** kNm Moment capacity low shear M_c = **174.1** kNm

PASS - Moment capacity exceeds design bending moment

MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly			Job no.: 16240	
	Calcs for: Steel Column			Start page no./Revision: 37	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

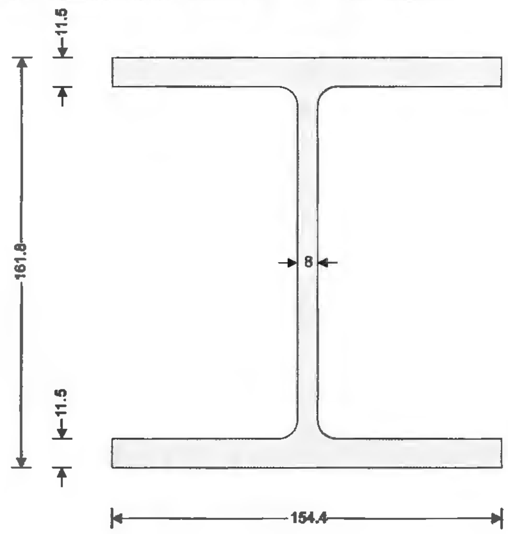
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.05

Section details

Section type **UC 152x152x37 (BS4-1)** Steel grade **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **20.1** kN Design shear resistance $P_{y,v}$ = **275.7** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Moment capacity - Section 4.2.5

Design bending moment M = **23.5** kNm Moment capacity low shear M_c = **109.6** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **21.1** kN Compression resistance P_{cx} = **1560.6** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check $F_c / (A \times p_y) + M / M_c$ = **0.227**

PASS - Combined bending and compression check is satisfied

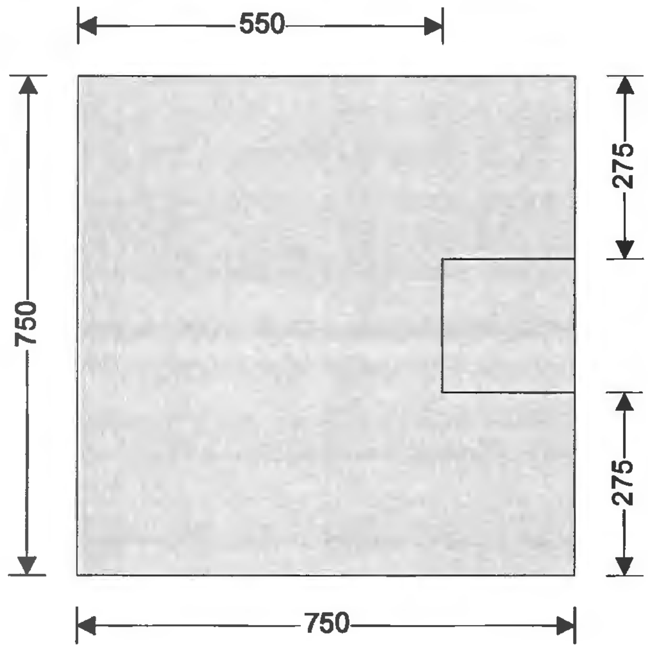
Member buckling resistance - cl.4.8.3.3.2

Buckling resistance check $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx})$ = **0.229**

PASS - Member buckling resistance checks are satisfied

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

TEDDS calculation version 2.0.07



Pad footing details

Length of pad footing	L = **750** mm	Width of pad footing	B = **750** mm
Depth of pad footing	h = **600** mm	Depth of soil over pad footing	h_{soil} = **200** mm
Density of concrete	ρ_{conc} = **23.6** kN/m^3		

Column details

Column base length	l_A = **200** mm	Column base width	b_A = **200** mm
Column eccentricity in x	e_{PxA} = **275** mm	Column eccentricity in y	e_{PyA} = **0** mm

Soil details

Depth of soil over pad footing	h_{soil} = **200** mm	Density of soil	ρ_{soil} = **20.0** kN/m^3
Allowable bearing pressure	$P_{bearing}$ = **150** kN/m^2		

Axial loading on column

Dead axial load	P_{GA} = **8.5** kN	Imposed axial load	P_{QA} = **5.0** kN
Wind axial load	P_{WA} = **3.2** kN	Total axial load	P_A = **16.7** kN

Foundation loads

Dead surcharge load	F_{Gsur} = **0.000** kN/m^2	Imposed surcharge load	F_{Qsur} = **0.000** kN/m^2
Pad footing self weight	F_{swt} = **14.160** kN/m^2		
Soil self weight	F_{soil} = **4.000** kN/m^2	Total foundation load	F = **10.2** kN

Calculate pad base reaction

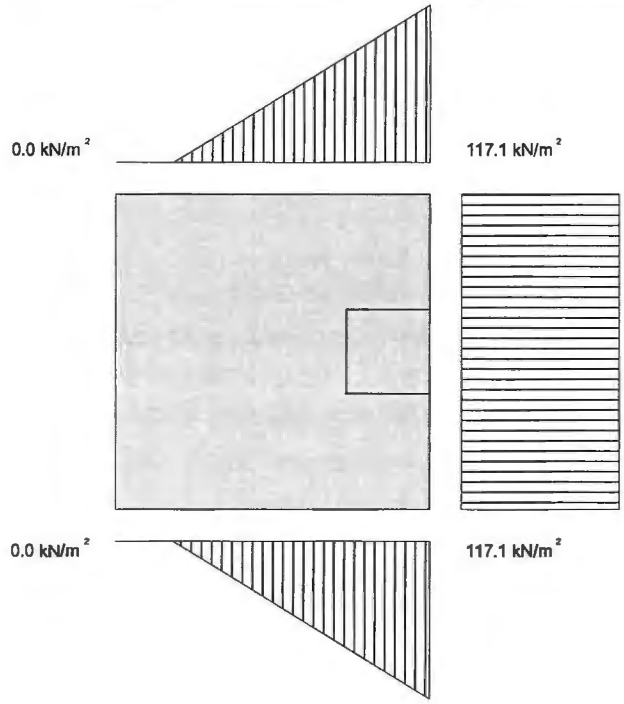
Total base reaction	T = **26.9** kN		
Base reaction eccentricity in x	e_{Tx} = **171** mm	Base reaction eccentricity in y	e_{Ty} = **0** mm

Base reaction acts outside of middle third of base

Calculate pad base pressures

q_1 = **0.000** kN/m^2	q_2 = **0.000** kN/m^2	q_3 = **117.064** kN/m^2	q_4 = **117.064** kN/m^2
Minimum base pressure	q_{min} = **0.000** kN/m^2	Maximum base pressure	q_{max} = **117.064** kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure



Material details

Char.strength of concrete f_{cu} = **20** N/mm^2

Calculate minimum depth of unreinforced pad footing

Ave.pressure to left of footing q_L = **-29.623** kN/m^2

Ave.pressure to right of footing q_R = **117.064** kN/m^2

Ave.pressure to top of footing q_T = **58.532** kN/m^2

Ave.pressure to btm of footing q_B = **58.532** kN/m^2

Min.depth unreinforced footing h_{min} = **550** mm

Min.depth to left of footing h_{Lmin} = **550** mm

Min.depth to right of footing h_{Rmin} = **0** mm

Min.depth to top of footing h_{Tmin} = **275** mm

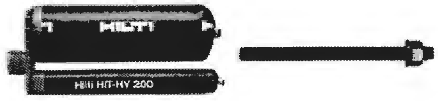
Min.depth to btm of footing h_{Bmin} = **275** mm

PASS - Unreinforced pad footing depth is greater than minimum

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	18/10/2017
E-Mail:			

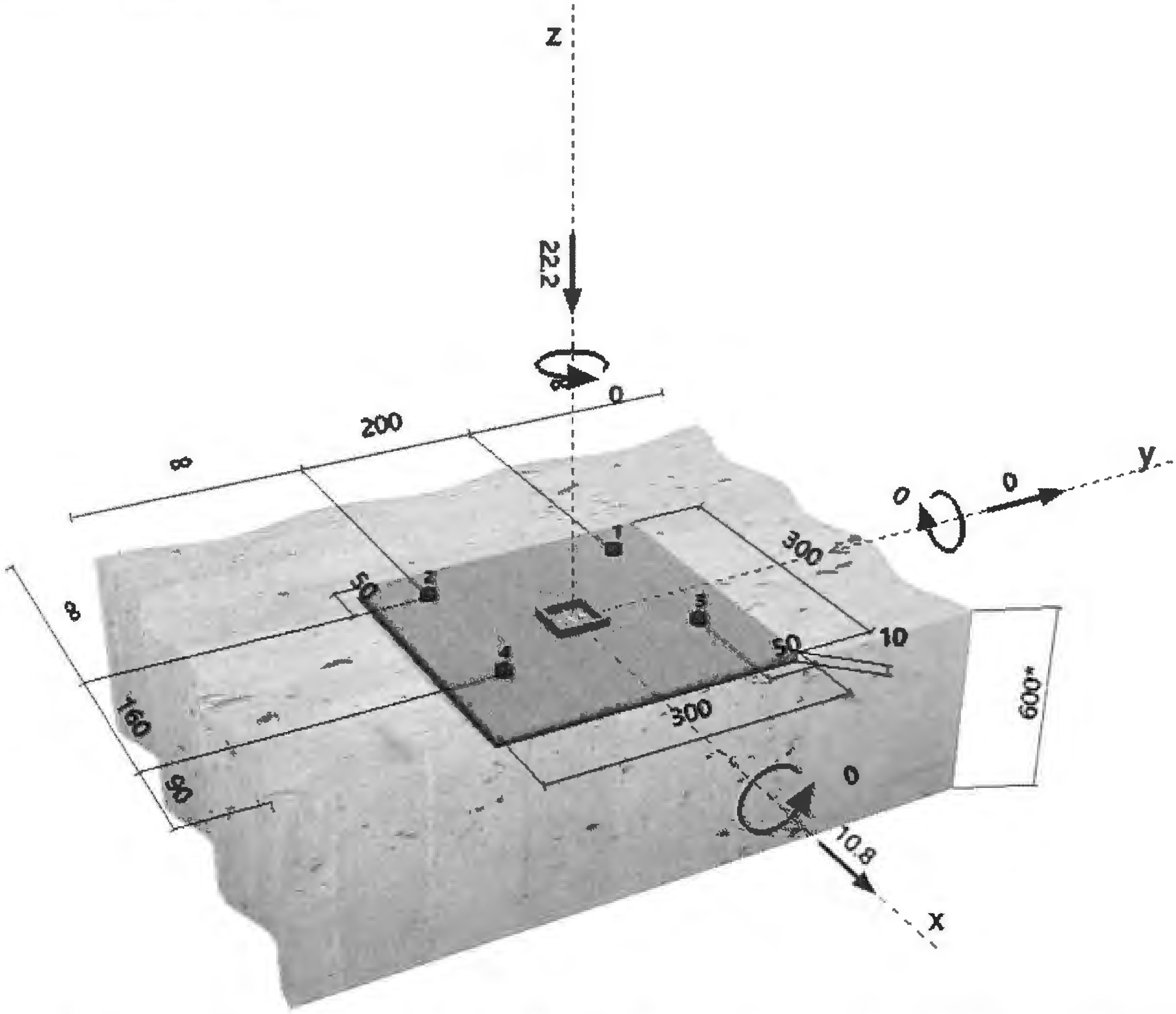
Specifier's comments:

1 Input data



Anchor type and diameter:	**HIT-HY 200-A + HIT-V (5.8) M16**
Effective embedment depth:	$h_{ef,act}$ = 175 mm ($h_{ef,limit}$ = - mm)
Material:	5.8
Evaluation Service Report:	ETA 11/0493
Issued I Valid:	28/07/2017 | -
Proof:	Design method ETAG BOND (EOTA TR 029)
Stand-off installation:	e_b = 0 mm (no stand-off); t = 10 mm
Anchor plate:	l_x x l_y x t = 300 mm x 300 mm x 10 mm; (Recommended plate thickness: not calculated
Profile:	Square hollow; (L x W x T) = 50 mm x 50 mm x 5 mm
Base material:	uncracked concrete, C30/37, $f_{c,cube}$ = 37.00 N/mm^2; h = 600 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

Geometry [mm] & Loading [kN, kNm]



Company:		Page:	2
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	18/10/2017
E-Mail:			

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [kN] Load	Capacity	Utilization β_N / β_V [%]	Status
Tension	-	-	-	- / -	-
Shear	Concrete edge failure in direction x+	10.800	30.691	- / 36	OK

Loading	β_N	β_V	α	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 42	Date OCT '17

MBA CONSULTING

STUD WALL TIMBERS:
SPAN < 4.0m

LOADINGS.

WIND LOAD = $0.93 \times 0.4 = 0.39$ kN/m

FLAT ROOF - DL = $[0.90 \times \cos(10) \times 2.6/2] \times 0.4 = 0.46$

IL = $[0.6 \times 2.6/2] \times 0.4 = 0.32$ kN

MOMENT = $\frac{0.39 \times 4^2}{8} + 0.2 \times [0.46 + 0.32] = 0.95$ kNm

SHEAR = $\frac{0.39 \times 4}{2} = 0.78$ kN.

AXIAL LOAD = 0.78 kN.

DISTANCE BETWEEN RESTRAINTS = 4000 mm

PROVIDE 2 No 140 x 38. C24 STUDS @ 400 mm c/c.

MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly			Job no.: 16240	
	Calcs for: Stud Wall Timbers - Span <4m			Start page no./Revision: 43	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER MEMBER DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00

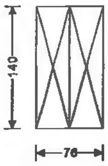
Analysis results

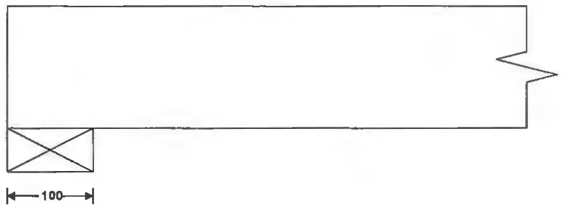
Design moment in major axis M_x = **0.950** kNm

Design shear F = **0.780** kN

Maximum reaction R = **0.710** kN

Design axial compression P = **0.780** kN





Timber section details

Breadth of section b = **38** mm | Depth of section h = **140** mm

Number of sections N = **2** | Breadth of beam b_b = **76** mm

Timber strength class **C24**

Member details

Service class of timber **1** | Load duration **Short term**

Length of bearing L_b = **100** mm

Unbraced length in x-axis L_x = **4000** mm | Unbraced length in y-axis L_y = **4000** mm Effective

length factor in x-axis K_x = **1** | Effective length factor in y-axis K_y = **1**

Effective length in x-axis L_{ex} = **4000** mm | Effective length in y-axis L_{ey} = **4000** mm

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

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** | Actual depth-to-breadth ratio **1.84**

PASS - Lateral support is adequate

Slenderness ratio - cl.2.11.4

Slenderness ratio λ = **182.321** | Permissible slenderness ratio λ_{max} = **180**

FAIL - Slenderness ratio exceeds permissible slenderness ratio

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.960** N/mm^2 | Applied bearing stress σ_{c_a} = **0.093** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **13.457** N/mm^2 | Applied bending stress σ_{m_a} = **3.827** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Compression parallel to grain

Permissible comp.stress σ_{c_adm} = **1.396** N/mm^2 | Applied compressive stress σ_{c_a} = **0.073** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress

Members subject to axial compression and bending - cl.2.11.6

Comb.comp.and bending **0.338 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Shear parallel to grain

Permissible shear stress τ_{adm} = **1.172** N/mm^2 | Applied shear stress τ_a = **0.110** N/mm^2

Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 44	Date Nov '17

MBA CONSULTING

STUD WALL TIMBERS -
SPAN < 3.5m.

LOADINGS

WIND LOAD = $0.93 \times 0.4 = 0.39 kN/m^2$

FLAT ROOF = $DL = \left[0.9 \times \cos(10) \times \frac{2.6}{2}\right] \times 0.4 = 0.46kN$

$IL = \left[0.6 \times \frac{2.6}{2}\right] \times 0.4 = 0.32kN.$

MOMENT $= \frac{0.39 \times 3.5^2}{8} + 0.2 \times [0.39 + 0.32] = 0.76kNm$

SHEAR $= \frac{0.39 \times 3.5}{2} = 0.69$

AXIAL LOAD = 0.78 kN.

PROVIDE
140 x 38 C24
TIMBER STUDS
@ 400mm c/c.

MBA CONSULTING Tel: Fax:	Project Carn Gwavel, Isles of Scilly				Job no. 16240	
	Calcs for Stud Wall Timbers - Span <3.5m				Start page no./Revision 45	
	Calcs by EP	Calcs date 13/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER MEMBER DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00

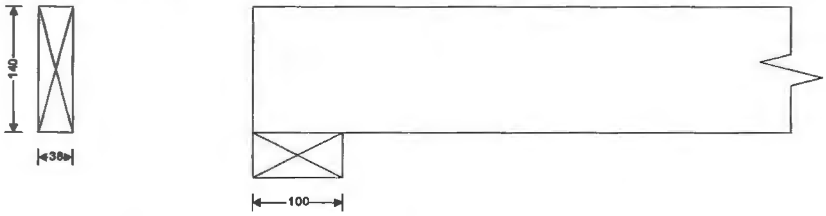
Analysis results

Design moment in major axis M_x = **0.950** kNm

Design shear F = **0.780** kN

Maximum reaction R = **0.710** kN

Design axial compression P = **0.780** kN



Timber section details

Breadth of section	b = **38** mm	Depth of section	h = **140** mm
Number of sections	N = **1**	Breadth of beam	b_b = **38** mm
Timber strength class	**C24**		

Member details

Service class of timber	**1**	Load duration	**Short term**
Length of bearing	L_b = **100** mm		
Unbraced length in x-axis	L_x = **3500** mm	Unbraced length in y-axis	L_y = **3500** mm Effective
length factor in x-axis	K_x = **1**	Effective length factor in y-axis	K_y = **0.7**
Effective length in x-axis	L_{ex} = **3500** mm	Effective length in y-axis	L_{ey} = **2450** mm

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

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **3.68**

PASS - Lateral support is adequate

Slenderness ratio - cl.2.11.4

Slenderness ratio λ = **223.343** Permissible slenderness ratio λ_{max} = **250**

PASS - Slenderness ratio is less than permissible slenderness ratio

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.960** N/mm^2 Applied bearing stress σ_{c_a} = **0.187** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **13.457** N/mm^2 Applied bending stress σ_{m_a} = **7.653** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Compression parallel to grain

Permissible comp.stress σ_{c_adm} = **0.953** N/mm^2 Applied compressive stress σ_{c_a} = **0.147** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress

Members subject to axial compression and bending - cl.2.11.6

Comb.comp.and bending **0.729 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Shear parallel to grain

Permissible shear stress τ_{adm} = **1.172** N/mm^2 Applied shear stress τ_a = **0.220** N/mm^2

Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 46	Date Nov '17

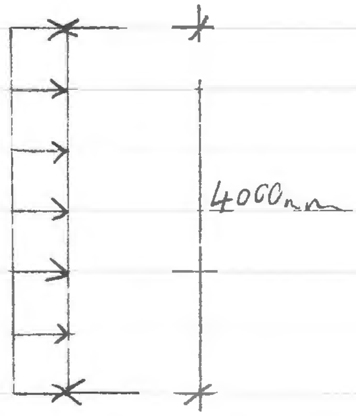
MBA CONSULTING

STUD WALL - TIMBERS ADJASENT TO OPENINGS.

LOADINGS

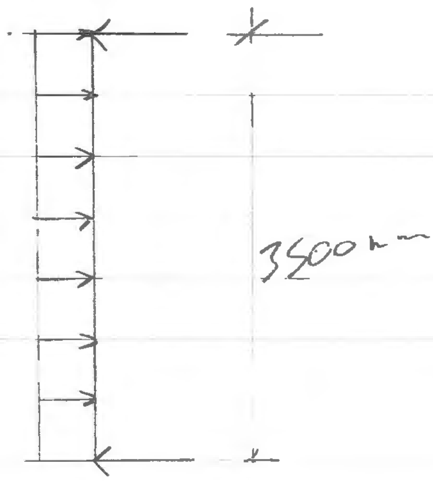
$$\text{WIND LOAD} = 0.93 \times \frac{0.4 + 1.0}{2} = 0.651$$

<4m SPAN



PROVIDE 3No 140 x 38 C24 TIMBER STUDS EACH SIDE OF OPENINGS.

< 3.5m SPAN

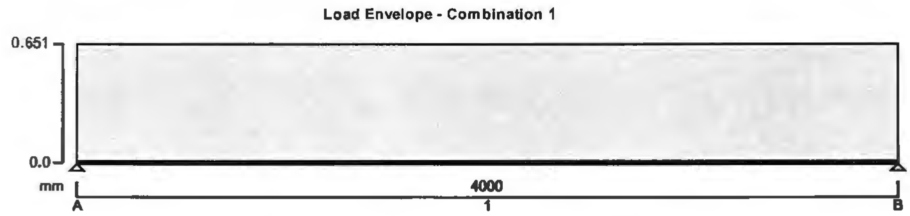


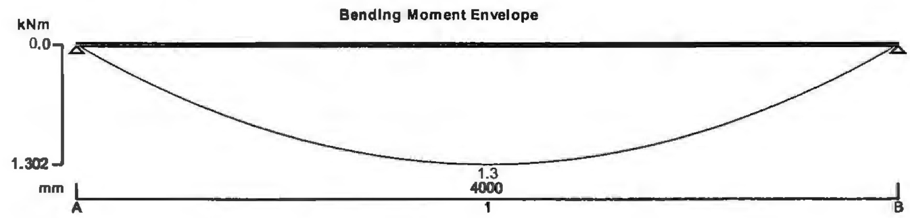
PROVIDE 2No 140 x 38 C24 TIMBER STUDS EACH SIDE OF OPENING

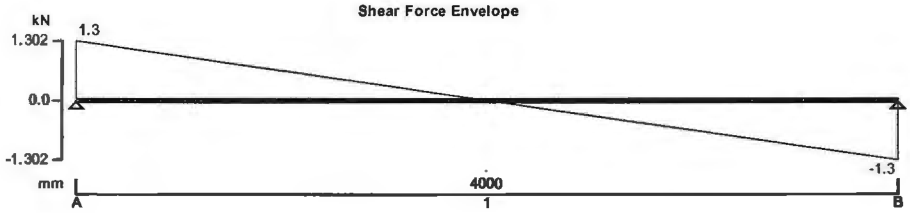
MBA CONSULTING Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN Tel: 01872 260962 Fax: 01872 260963	Project Carn Gwavel, Isles of Scilly			Job no. 16240	
	Calcs for Stud Wall Timbers (Adjasent to openings)			Start page no./Revision 47	
	Calcs by EP	Calcs date 10/10/2017	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Imposed full UDL 0.651 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment M = **1.302** kNm — Design shear F = **1.302** kN

Total load on beam W_{tot} = **2.604** kN

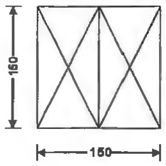
Reactions at support A R_{A_max} = **1.302** kN — R_{A_min} = **1.302** kN

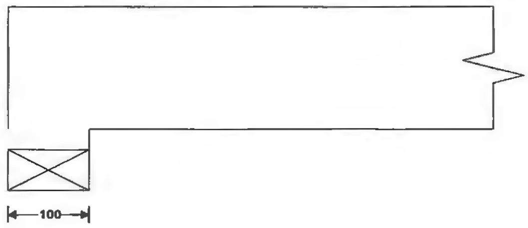
Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **1.302** kN

Reactions at support B R_{B_max} = **1.302** kN — R_{B_min} = **1.302** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **1.302** kN

MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly			Job no.: 16240	
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN. Tel: 01872 260962 Fax: 01872 260963	Calcs for: Stud Wall Timbers (Adjasent to openings)			Start page no./Revision: 48	
	Calcs by: EP	Calcs date: 10/10/2017	Checked by	Checked date	Approved by	Approved date





Timber section details

Breadth of section	b = **75** mm	Depth of section	h = **150** mm
Number of sections	N = **2**	Breadth of beam	b_b = **150** mm
Timber strength class	**C16**		

Member details

Service class of timber	**1**	Load duration	**Medium term**
Length of bearing	L_b = **100** mm		

Underside of beam notched at all supports

Beam depth at notch h_e = **175** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **5.00** Actual depth-to-breadth ratio **1.00**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.025** N/mm^2 Applied bearing stress σ_{c_a} = **0.087** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **7.865** N/mm^2 Applied bending stress σ_{m_a} = **2.315** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain at notched support

Permissible shear stress τ_{adm} = **1.075** N/mm^2 Applied shear stress τ_a = **0.074** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

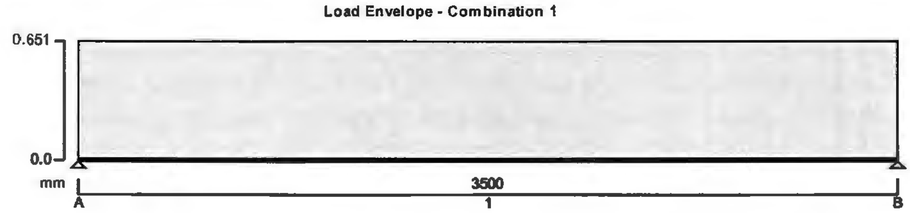
Permissible deflection δ_{adm} = **12.000** mm Total deflection δ_a = **7.947** mm

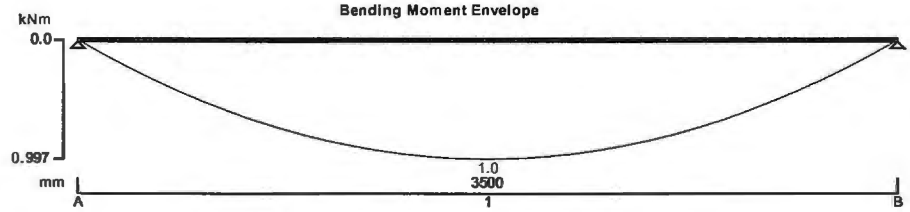
PASS - Total deflection is less than permissible deflection

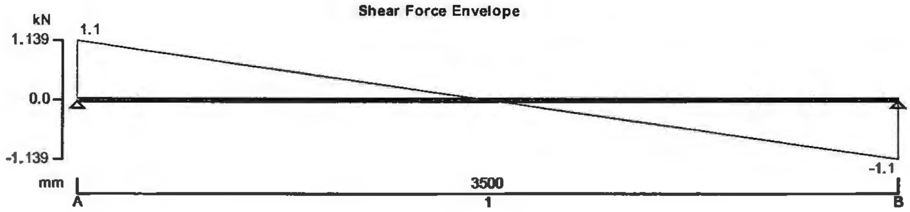
Boscawen House, Chapel Hill Tel: 01872 260962
Truro, Cornwall TR1 3BN Fax: 01872 260963

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Imposed full UDL 0.651 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment M = **0.997** kNm Design shear F = **1.139** kN

Total load on beam W_{tot} = **2.279** kN

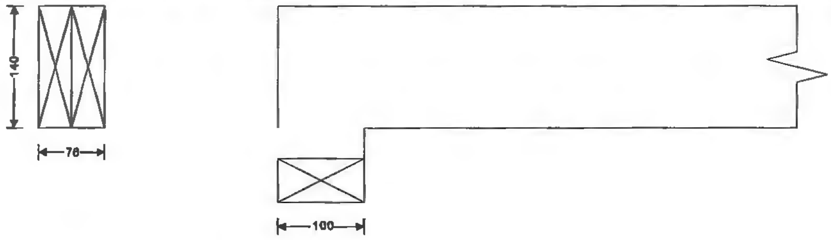
Reactions at support A R_{A_max} = **1.139** kN R_{A_min} = **1.139** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **1.139** kN

Reactions at support B R_{B_max} = **1.139** kN R_{B_min} = **1.139** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **1.139** kN

MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly				Job no.: 16240	
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN. Tel: 01872 260962, Fax: 01872 260963	Calcs for: Stud Wall Timbers (Adjasent To Openings)				Start page no./Revision: 50	
	Calcs by: EP	Calcs date: 03/11/2017	Checked by	Checked date	Approved by	Approved date



Timber section details

Breadth of section	b = **38** mm	Depth of section	h = **140** mm
Number of sections	N = **2**	Breadth of beam	b_b = **76** mm
Timber strength class	**C24**		

Member details

Service class of timber	**1**	Load duration	**Medium term**
Length of bearing	L_b = **100** mm		

Underside of beam notched at all supports

Beam depth at notch h_e = **175** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **5.00** — Actual depth-to-breadth ratio **1.84**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.000** N/mm^2 — Applied bearing stress σ_{c_a} = **0.150** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **10.195** N/mm^2 — Applied bending stress σ_{m_a} = **4.015** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain at notched support

Permissible shear stress τ_{adm} = **1.109** N/mm^2 — Applied shear stress τ_a = **0.128** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **10.500** mm — Total deflection δ_a = **10.416** mm

PASS - Total deflection is less than permissible deflection

Made by	Job Title	Carn Gwavel, Isles Of Scilly		
Checked by	Job No. 6240	Sheet 51	Date Oct '17	

MBA CONSULTING

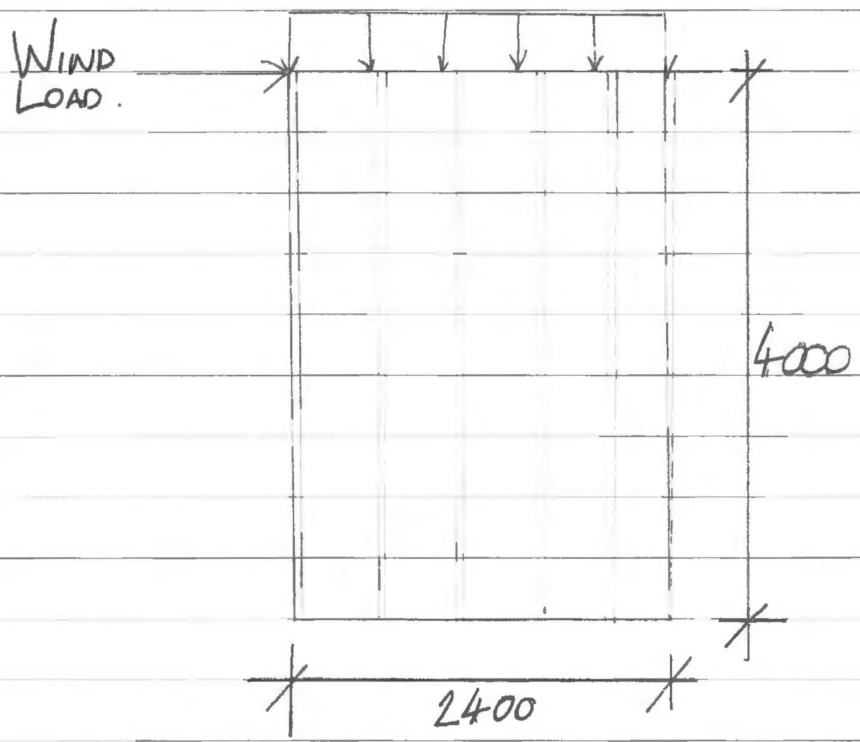
External West Wall – Racking

Loadings

Wind Load $= 0.93 \times \frac{3.5 \times 4}{4} = 3.26\ kN$

Flat Roof $- DL = \frac{0.90}{\cos 10} \times \frac{3.0}{2} = 1.38\ kN/m$

Wind Uplift $- WL = {}^{-}1.05 \times \frac{3.0}{2} = {}^{-}1.575\ kN/m$



MBA CONSULTING	Project Carn Gwavel, Isles of Scilly			Job no. 16240	
	Calcs for External West Wall Detail			Start page no./Revision 52	
	Calcs by EP	Calcs date 13/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER FRAME RACKING PANEL DESIGN

In accordance with EN1995-1-1:2004 + A1:2008 incorporating corrigendum June 2006 and the UK National Annex incorporating Corrrigendum No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.0.04

Compression force of the leeward end of the racking wall shall be checked independently in combination with the stud buckling design and the top rail bearing design in accordance with Eurocode 5. For a wall diaphragm with more than two studs within the 0.1 L of its leeward end in a dwelling of less than three storeys, the compressive force at the leeward end may be disregarded (clause 21.5.2.10 of PD6693-1).

Single storey racking wall without openings

Wall panel geometry

Wall height	H = **4000** mm	Wall length	L = **2400** mm
Width of stud	b_s = **38** mm	Depth of stud	h_s = **140** mm
Stud spacing	s_s = **400** mm	Sheathing layers	**1**

Timber frame material

Material	Solid timber	Strength class	C16
Characteristic density	ρ_k = **310** kg/m^3		

Sheathing materials

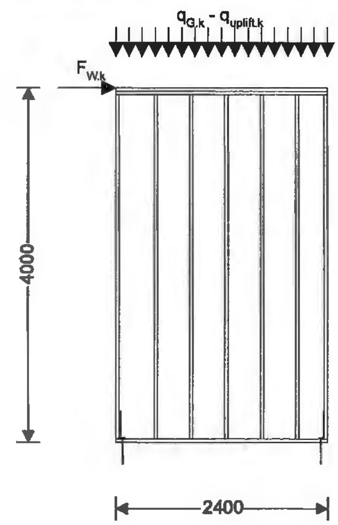
Sheathing material	Plywood	t_{s1} = **9** mm	ρ_{k1} = **384** kg/m^3
Fastener	d_{n1} = **3.1** mm	l_{n1} = **50** mm	s_1 = **150** mm

Connection to substrate

Sole plate detail	Open panel sole plate detail	Holding down restrain	Strap dimensions 50 mm ×
610 mm and no.4 nails			
		Characteristic restrain capacity	$F_{hd.k}$ = **5.40** kN

Actions on the wall

Self weight	$q_{sw.k}$ = **0.20** kN/m^2	Permanent load	$q_{G.k}$ = **1.38** kN/m
Uplift wind load	$q_{uplift.k}$ = **1.58** kN/m	Horizontal racking load	$F_{W.k}$ = **3.26** kN



Sole plate fixing detail

Number of shear planes	N_{sp} = **2**		
Sub-connection 1 - Bottom rail to sole plate, C1		BRT ring shanked nail 3.1 x 75 mm	

Charact withdrawal capacity	$F_{sp.w.k1}$ = **0.19** kN	Charact shear capacity	$F_{sp.v.k1}$ = **0.83** kN
Number of parallel fasteners	n_{sp1} = **1**	Spacing of the fasteners	s_{sp1} = **150** mm
Sub-connection 2 - Sole plate to foundations, C2		Shot-fired smooth round nail 3.5 x 70 mm	
Charact withdrawal capacity	$F_{sp.w.k2}$ = **0.05** kN	Charact shear capacity	$F_{sp.v.k2}$ = **1.50** kN
Number of parallel fasteners	n_{sp2} = **1**	Spacing of the fasteners	s_{sp2} = **300** mm

Partial factors

Unfactored calculation. All partial safety and material factors set to value 1.0 excluding unfavourable variable loading with value partial factor of 0.

Determination of design fastener capacities

Sole plate fixing fasteners

Shear plane 1			
Design withdrawal capacity	$f_{sp.w.d1}$ = **1.27** kN/m	Design lateral load capacity	$f_{sp.v.d1}$ = **5.55** kN/m
Shear plane 2			
Design withdrawal capacity	$f_{sp.w.d2}$ = **0.16** kN/m	Design lateral load capacity	$f_{sp.v.d2}$ = **5.01** kN/m
Minimum withdrawal capacity	$f_{sp.w.d}$ = **0.16** kN/m	Minimum lateral load capacity	$f_{sp.v.d}$ = **5.01** kN/m

Primary sheathing to frame connection

Fastener spacing	s_1 = **150** mm	Design lateral load capacity	$F_{v.Rd1}$ = **0.63** kN
Design shear by unit length	$f_{p.d1}$ = **5.47** kN/m		

Design loads acting on shear wall

Design permanent load	$q_{G.d}$ = **1.38** kN/m	Design wind uplift load	$q_{uplift.d}$ = **1.58** kN/m
Self-weight of the wall panel	$q_{sw.d}$ = **0.80** kN/m		
Design horizontal wind load	$F_{W.d}$ = **3.26** kN	Design holding down tension	$F_{hd.d}$ = **5.40** kN

Design destabilising moments

Distance to top sheathing	$h_{dst.top}$ = **38** mm	Distance to base sheathing	$h_{dst.base}$ = **4038** mm
Destabilising moment at top	$M_{d.dst.top}$ = **0.12** kNm	Destabilising moment at base	$M_{d.dst.base}$ = **13.16** kNm

Design stabilising moments

Design stabilising vertical load	$f_{stb.d}$ = **0.61** kN/m	Total vertical load	$F_{stb.d}$ = **1.45** kN
Stabilising moment load	$M_{d.stb.f}$ = **1.74** kNm	Stabilising moment h-down	$M_{d.stb.hd}$ = **12.96** kNm
Total stabilising moment	$M_{d.stb}$ = **14.70** kNm		

Design for overturning stability

Net stabilising moment - top	$M_{n.top}$ = **14.58** kNm	Factor of Utilisation	$M_{d.dst.top}$ / $M_{d.stb}$ = **0.008**

PASS - Design stabilising moment exceeds design destabilising moment at top of shear wall

Net stabilising moment - base	$M_{n.base}$ = **1.54** kNm	Factor of Utilisation	$M_{d.dst.base}$ / $M_{d.stb}$ = **0.895**

PASS - Design stabilising moment exceeds design destabilising moment at base of the shear wall

Design for sliding stability

Coefficient of friction	μ_{fr} = 0.4	Frictional resistance	$F_{friction}$ = **0.58** kN
Sole plate shear resistance	$F_{sp.v.d}$ = **12.02** kN	Sliding resistance	$F_{sliding}$ =**12.60** kN
Design horizontal wind load	$F_{W.d}$ = **3.26** kN		$F_{W.d}$ / $F_{sliding}$ = **0.259**

PASS - Design sliding resistance exceeds horizontal design wind load

Opening factor – cl 21.26

Percentage of opening area	p = **0.00**	Opening factor	$K_{opening}$ = **1.00**

Panel shape factor – cl 21.5.2.5

Ratio of shear capacities	μ = **0.03**	Shape factor	K_w = **0.29**

Design racking strength – cl 21.5.2

Shear strength of sole plate	$F_{sp.v.d}$ = **12.02** kN	Shear strength of panel	$F_{wall.v.d}$ = **3.75** kN

Approved date:

Design racking strength $F_{v.d}$ = **3.75** kN $F_{w.d}$ / $F_{v.d}$ = **0.869**

PASS - Design racking strength exceeds racking load due to wind

Serviceability load limit $F_{SLS.lim}$ = **4.80** kN/m Modified serviceability load $F_{SLS.mod}$ = **1.56** kN/m

$F_{SLS.mod}$ / $F_{SLS.lim}$ = **0.326**

PASS - The condition 21.5.2.3 of PD 6693-1 is met

Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 55	Date OCT '17

MBA CONSULTING

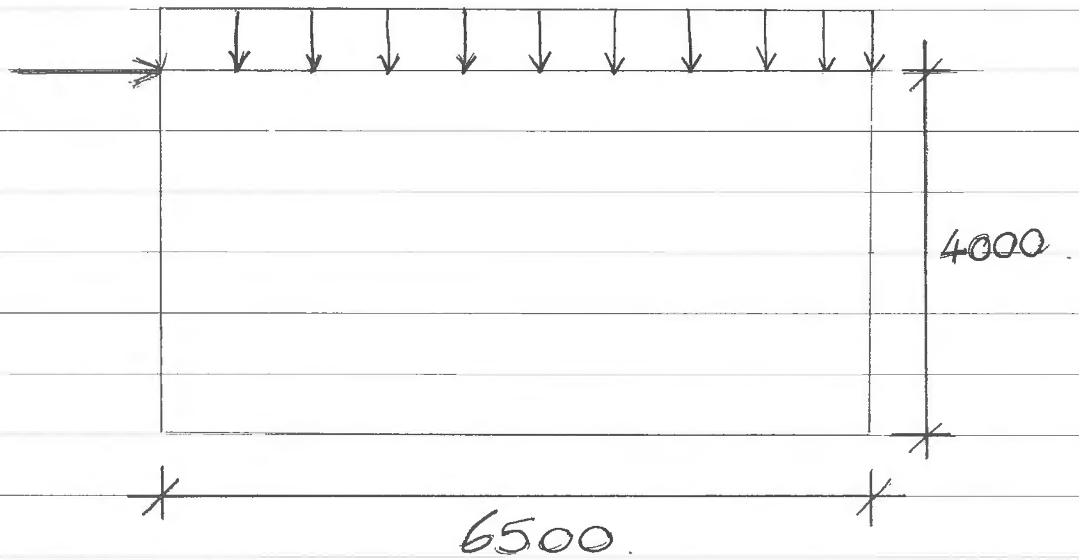
EXTERNAL NORTH WALL
- RACKING

LOADINGS.

WIND LOADING $- 0.93 \times \frac{4.5 \times 4}{4} = 4.19\,kN$

FLAT ROOF DL $= \frac{0.90}{\cos(10)} \times \frac{1.0}{2} = 0.46\,kN/m$

WIND UPLIFT $= -1.05 \times \frac{1.0}{2} = -0.53\,kN/m$



Project Carn Gwavel, Isles of Scilly				Job no. 16240	
Calcs for External North Wall Detail				Start page no./Revision 56	
Calcs by EP	Calcs date 13/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER FRAME RACKING PANEL DESIGN

In accordance with EN1995-1-1:2004 + A1:2008 incorporating corrigendum June 2006 and the UK National Annex incorporating Corrrigendum No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.0.04

Compression force of the leeward end of the racking wall shall be checked independently in combination with the stud buckling design and the top rail bearing design in accordance with Eurocode 5. For a wall diaphragm with more than two studs within the 0.1 L of its leeward end in a dwelling of less than three storeys, the compressive force at the leeward end may be disregarded (clause 21.5.2.10 of PD6693-1).

Single storey racking wall without openings

Wall panel geometry

Wall height	H = **4000** mm	Wall length	L = **6500** mm
Width of stud	b_s = **38** mm	Depth of stud	h_s = **140** mm
Stud spacing	s_s = **400** mm	Sheathing layers	**1**

Timber frame material

Material	Solid timber	Strength class	C16
Characteristic density	ρ_k = **310** kg/m^3		

Sheathing materials

Sheathing material	Plywood	t_{s1} = **9** mm	ρ_{k1} = **384** kg/m^3
Fastener	d_{n1} = **3.1** mm	l_{n1} = **50** mm	s_1 = **150** mm

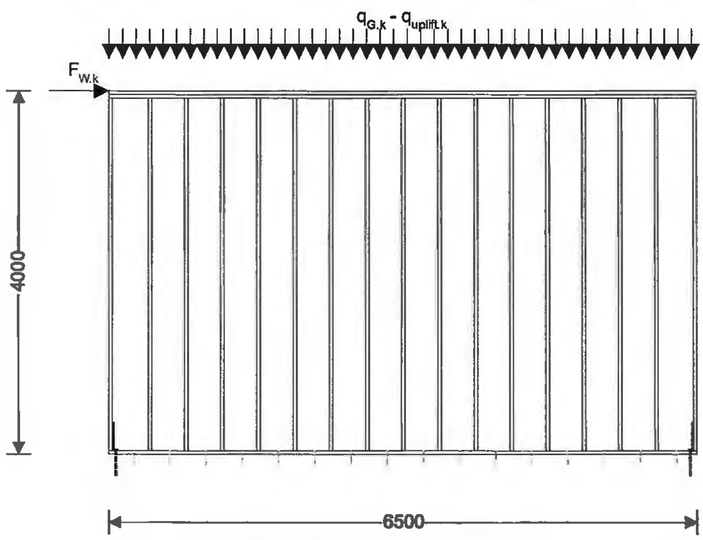
Connection to substrate

Sole plate detail Open panel sole plate detail Holding down restrain Strap dimensions 50 mm × 610 mm and no.4 nails

Characteristic restrain capacity $F_{hd.k}$ = **5.40** kN

Actions on the wall

Self weight	$q_{sw.k}$ = **0.20** kN/m^2	Permanent load	$q_{G.k}$ = **0.46** kN/m
Uplift wind load	$q_{uplift.k}$ = **0.53** kN/m	Horizontal racking load	$F_{W.k}$ = **4.19** kN



Sole plate fixing detail

Number of shear planes N_{sp} = **2**

Sub-connection 1 - Bottom rail to sole plate, C1 BRT ring shanked nail 3.1 x 75 mm

Charact withdrawal capacity	$F_{sp.w.k1}$ = **0.19** kN	Charact shear capacity	$F_{sp.v.k1}$ = **0.83** kN
Number of parallel fasteners	n_{sp1} = **1**	Spacing of the fasteners	s_{sp1} = **150** mm
Sub-connection 2 - Sole plate to foundations, C2		Shot-fired smooth round nail 3.5 x 70 mm	
Charact withdrawal capacity	$F_{sp.w.k2}$ = **0.05** kN	Charact shear capacity	$F_{sp.v.k2}$ = **1.50** kN
Number of parallel fasteners	n_{sp2} = **1**	Spacing of the fasteners	s_{sp2} = **300** mm

Partial factors

Unfactored calculation. All partial safety and material factors set to value 1.0 excluding unfavourable variable loading with value partial factor of 0.

Determination of design fastener capacities

Sole plate fixing fasteners

Shear plane 1

Design withdrawal capacity	$f_{sp.w.d1}$ = **1.27** kN/m	Design lateral load capacity	$f_{sp.v.d1}$ = **5.55** kN/m

Shear plane 2

Design withdrawal capacity	$f_{sp.w.d2}$ = **0.16** kN/m	Design lateral load capacity	$f_{sp.v.d2}$ = **5.01** kN/m
Minimum withdrawal capacity	$f_{sp.w.d}$ = **0.16** kN/m	Minimum lateral load capacity	$f_{sp.v.d}$ = **5.01** kN/m

Primary sheathing to frame connection

Fastener spacing	s_1 = **150** mm	Design lateral load capacity	$F_{v.Rd1}$ = **0.63** kN
Design shear by unit length	$f_{p.d1}$ = **5.47** kN/m		

Design loads acting on shear wall

Design permanent load	$q_{G.d}$ = **0.46** kN/m	Design wind uplift load	$q_{uplift.d}$ = **0.53** kN/m
Self-weight of the wall panel	$q_{sw.d}$ = **0.80** kN/m		
Design horizontal wind load	$F_{W.d}$ = **4.19** kN	Design holding down tension	$F_{hd.d}$ = **5.40** kN

Design destabilising moments

Distance to top sheathing	$h_{dst.top}$ = **38** mm	Distance to base sheathing	$h_{dst.base}$ = **4038** mm
Destabilising moment at top	$M_{d.dst.top}$ = **0.16** kNm	Destabilising moment at base	$M_{d.dst.base}$ = **16.92** kNm

Design stabilising moments

Design stabilising vertical load	$f_{stb.d}$ = **0.73** kN/m	Total vertical load	$F_{stb.d}$ = **4.75** kN
Stabilising moment load	$M_{d.stb.f}$ = **15.42** kNm	Stabilising moment h-down	$M_{d.stb.hd}$ = **35.10** kNm
Total stabilising moment	$M_{d.stb}$ = **50.52** kNm		

Design for overturning stability

Net stabilising moment - top	$M_{n.top}$ = **50.36** kNm	Factor of Utilisation	$M_{d.dst.top}$ / $M_{d.stb}$ = **0.003**

PASS - Design stabilising moment exceeds design destabilising moment at top of shear wall

Net stabilising moment - base	$M_{n.base}$ = **33.60** kNm	Factor of Utilisation	$M_{d.dst.base}$ / $M_{d.stb}$ = **0.335**

PASS - Design stabilising moment exceeds design destabilising moment at base of the shear wall

Design for sliding stability

Coefficient of friction	μ_{fr} = 0.4	Frictional resistance	$F_{friction}$ = **1.90** kN
Sole plate shear resistance	$F_{sp.v.d}$ = **32.57** kN	Sliding resistance	$F_{sliding}$ =**34.46** kN
Design horizontal wind load	$F_{W.d}$ = **4.19** kN		$F_{W.d}$ / $F_{sliding}$ = **0.122**

PASS - Design sliding resistance exceeds horizontal design wind load

Opening factor – cl 21.26

Percentage of opening area	p = **0.00**	Opening factor	$K_{opening}$ = **1.00**

Panel shape factor – cl 21.5.2.5

Ratio of shear capacities	μ = **0.03**	Shape factor	K_w = **0.37**

Design racking strength – cl 21.5.2

Shear strength of sole plate	$F_{sp.v.d}$ = **32.57** kN	Shear strength of panel	$F_{wall.v.d}$ = **13.32** kN

Design racking strength $F_{v.d}$ = **13.32 kN** $F_{w.d}$ / $F_{v.d}$ = **0.315**

PASS - Design racking strength exceeds racking load due to wind

Serviceability load limit $F_{SLS.lim}$ = **13.00** kN/m Modified serviceability load $F_{SLS.mod}$ = **2.05** kN/m

$F_{SLS.mod}$ / $F_{SLS.lim}$ = **0.158**

PASS - The condition 21.5.2.3 of PD 6693-1 is met

Made by EP	Job Title: Carn Gwavel, Isles Of Scilly		
Checked by	Job No. 16240	Sheet 59	Date Oct '17

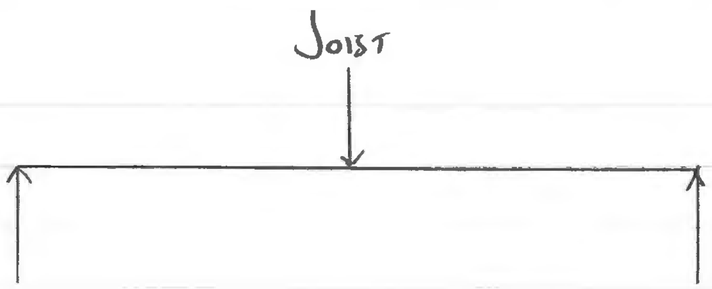
MBA CONSULTING

Stud Wall Bearing Timber Fixings

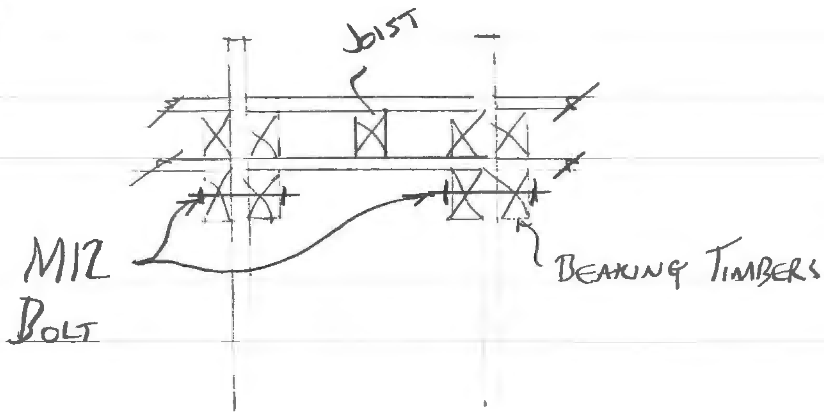
Loadings

Reaction From Joists $= DL = \frac{0.6}{\cos 10} \times 0.4 \times 3/2 \times 0.5 = 0.19$

$IL = 0.6 \times 0.4 \times 3/2 \times 0.5 = 0.18$



Vertical Resistance Req Per Bolt = 0.37 kN.



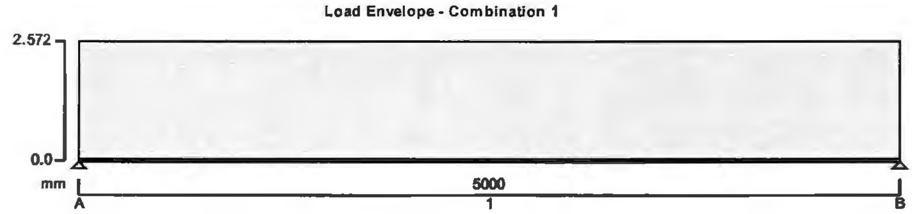
Provide 1 No. M10 Bolt Per Bearing Timber

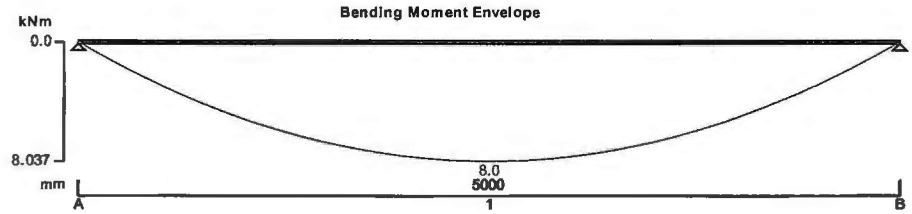
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN
Tel: 01872 260962
Fax 01872 260963

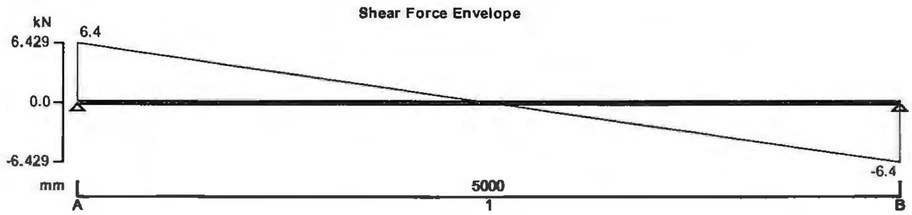
Project				Job no.	
Carn Gwavel, Isles Of Scilly				16240	
Calcs for				Start page no./Revision	
Purlin At Lower Level				60	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	17/10/2017				

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.230 kN/m
Imposed full UDL 1.110 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **8.037** kNm	Design shear	F = **6.429** kN
Total load on beam	W_{tot} = **12.858** kN		
Reactions at support A	R_{A_max} = **6.429** kN	R_{A_min} = **6.429** kN	

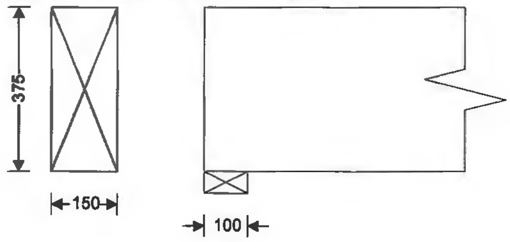
Unfactored dead load reaction at support A R_{A_Dead} = **3.654** kN
Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **2.775** kN

Boscawen House, Chapel Hill Truro, Cornwall TR1 3BN Tel: 01872 260962 Fax: 01872 260963

Reactions at support B R_{B_max} = **6.429** kN R_{B_min} = **6.429** kN

Unfactored dead load reaction at support B R_{B_Dead} = **3.654** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.775** kN



Timber section details

Breadth of section	b = **150** mm	Depth of section	h = **375** mm
Number of sections	N = **1**	Breadth of beam	b_b = **150** mm
Timber strength class	**C24**		

Member details

Service class of timber	**1**	Load duration	**Short term**
Length of bearing	L_b = **100** mm		

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **2.50**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.600** N/mm^2 Applied bearing stress σ_{c_a} = **0.429** N/mm^2

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

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **10.751** N/mm^2 Applied bending stress σ_{m_a} = **2.286** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **1.065** N/mm^2 Applied shear stress τ_a = **0.171** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **13.995** mm Total deflection δ_a = **4.791** mm

PASS - Total deflection is less than permissible deflection

Made by EP.	Job Title Carn Gwavel, Isles Of Scilly.		
Checked by	Job No. 16240	Sheet 62	Date Oct. 17

MBA CONSULTING

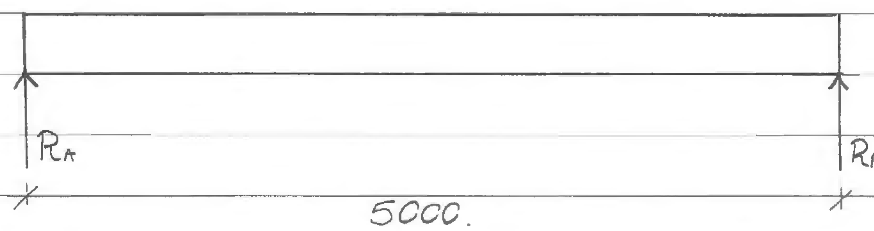
Purlins At Lower Level

Loadings

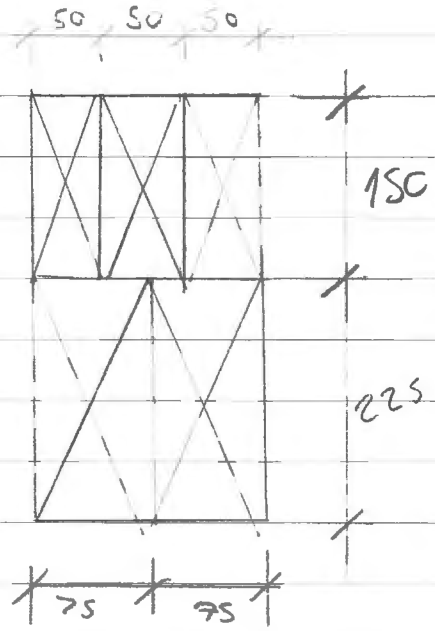
kN/m

Flat Roof – DL $= \left[\dfrac{0.75}{\cos(10)} \times \dfrac{2.5}{2}\right] + \left[0.75 \times \dfrac{1.2}{2}\right] = 1.41$

$IL = 0.6 \times \dfrac{2.5 + 1.2}{2} = 1.11$



Cross Section



Provide 3 N° 50 x 150 with 2 N° 75 x 225 C24 Timber Beams (see Diagram)

$R_A = R_B = [(4.6 \times 0.15 \times 0.375) + 1.41 + 1.11] \times 5.0/2$

$= 6.95$ kN

Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 63	Date OCT '17

MBA CONSULTING

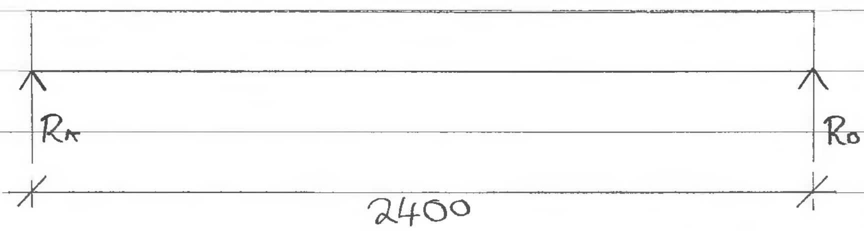
LOWER LEVEL PURLIN - MOST NORTHERLY

LOADINGS

kN/m

FLAT ROOF - $DL = \left[\frac{0.75}{\cos(10)} \times \frac{2.5}{2}\right] + \left[0.75 \times \frac{0.7}{2}\right] = 1.22$

$IL = 0.6 \times \frac{2.5 + 0.7}{2} = 0.96$



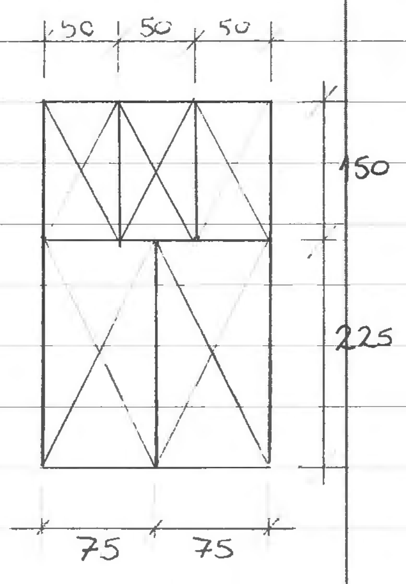
PROVIDE
3№ 50 x 150
WITH 2№ 75 x 225 C24
TIMBER BEAM

REACTIONS

$R_A = R_B = \left[[4.6 \times 0.375 \times 0.15] + 1.22 + 0.96 \right] \times \frac{2.4}{2}$

TIMBER S/W

$R_A = R_B = 2.93$ kN



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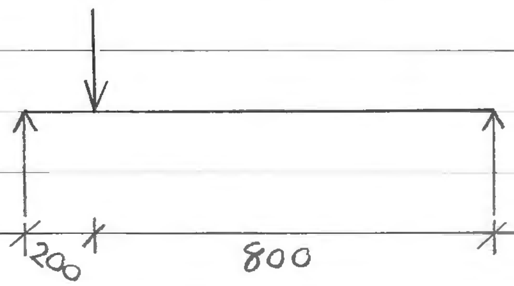
Checked by	Job No. 16240	Sheet 64	Date Oct '17

MBA CONSULTING

Timber Lintel Sup. Purlin

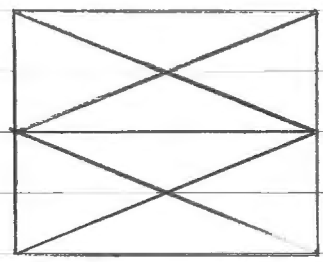
Loadings.

Purlin Reaction = 2.93 kN



Tedds Output

R_A = kN
R_B = kN



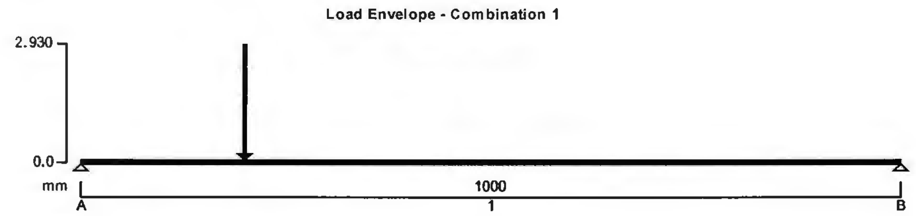
Section Of Beam.

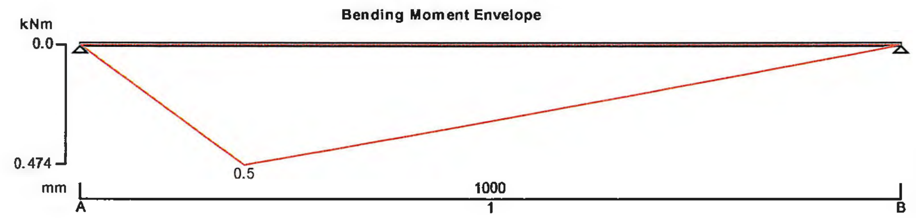
Provide 2 No. 150 x 50 dp C24 Timber Beams Above Opening

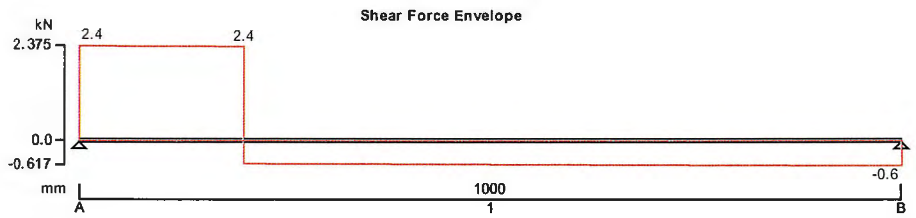
MBA CONSULTING	Project Carn Gwavel, Isles Of Scilly				Job no. 16240	
	Calcs for Timber Lintel Supporting Purlin				Start page no./Revision 65	
	Calcs by EP	Calcs date 13/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Dead self weight of beam × 1

Dead point load 2.930 kN at 200 mm

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

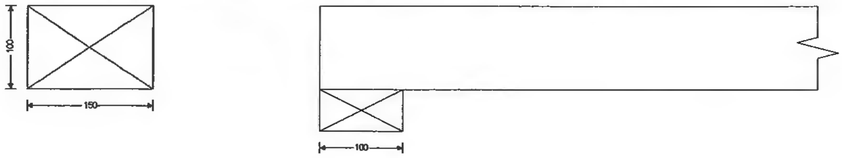
Analysis results

Maximum moment	M_{max} = **0.474** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.474** kNm	
Maximum shear	F_{max} = **2.375** kN	F_{min} = **-0.617** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.375** kN	
Total load on beam	W_{tot} = **2.992** kN	
Reactions at support A	R_{A_max} = **2.375** kN	R_{A_min} = **2.375** kN

Unfactored dead load reaction at support A R_{A_Dead} = **2.375** kN

Reactions at support B R_{B_max} = **0.617** kN R_{B_min} = **0.617** kN

Unfactored dead load reaction at support B R_{B_Dead} = **0.617** kN



Timber section details

Breadth of sections b = **150** mm

Depth of sections h = **100** mm

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

Overall breadth of member $b_b = N \times b$ = **150** mm

Timber strength class **C24**

Member details

Service class of timber **2**

Load duration **Short term**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **15000** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **250000** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **375000** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **12500000** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **28125000** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **28.9** mm

$i_y = \sqrt{(I_y / A)}$ = **43.3** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.50**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300\text{ mm} / h)^{0.11}$ = **1.13**

Load sharing - cl.2.9 K_8 = **1.00**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19 **2.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **0.67**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.600** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.158** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.044**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **12.695** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **1.895** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.149**

MBA CONSULTING	Project: Carn Gwavel, Isles Of Scilly			Job no.: 16240	
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PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **1.065** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.237** N/mm^2

$\tau_a / \tau_{adm} =$ **0.223**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} =$ **7200** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **3.000** mm

Bending deflection $\delta_{b\_s1} =$ **0.402** mm

Shear deflection $\delta_{v\_s1} =$ **0.084** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.486** mm

$\delta_a / \delta_{adm} =$ **0.162**

PASS - Total deflection is less than permissible deflection

Made by EP	Job Title Carn Gwavel, Isles Of Scilly		
Checked by	Job No. 16240	Sheet 68	Date Oct '17

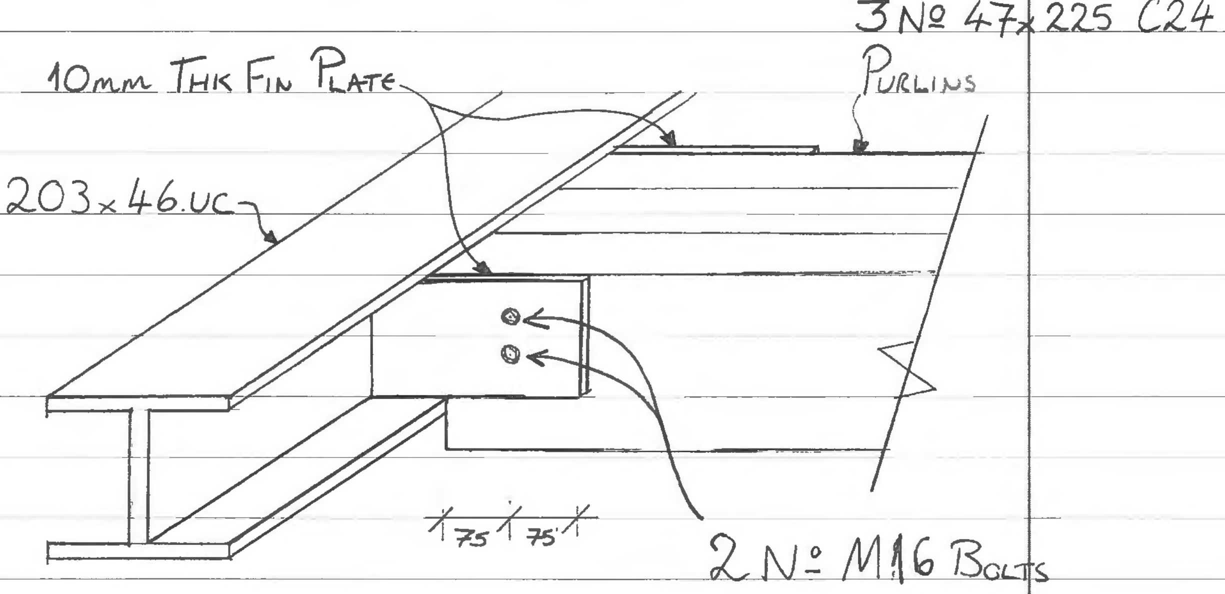
MBA CONSULTING

Purlin Connection Detail

Loadings

Timber Purlin Reaction – DL = 4.75 kN

IL = 3.34 kN



$F_{ADM} = F \times K_{56} \times K_{57}$

$F_{ADM} = 8.09 \times 0.97 = 7.85$ kN

$K_{56} = 1.0$

$K_{57} = 1 - \frac{3(n-1)}{100}$ $n = 2$

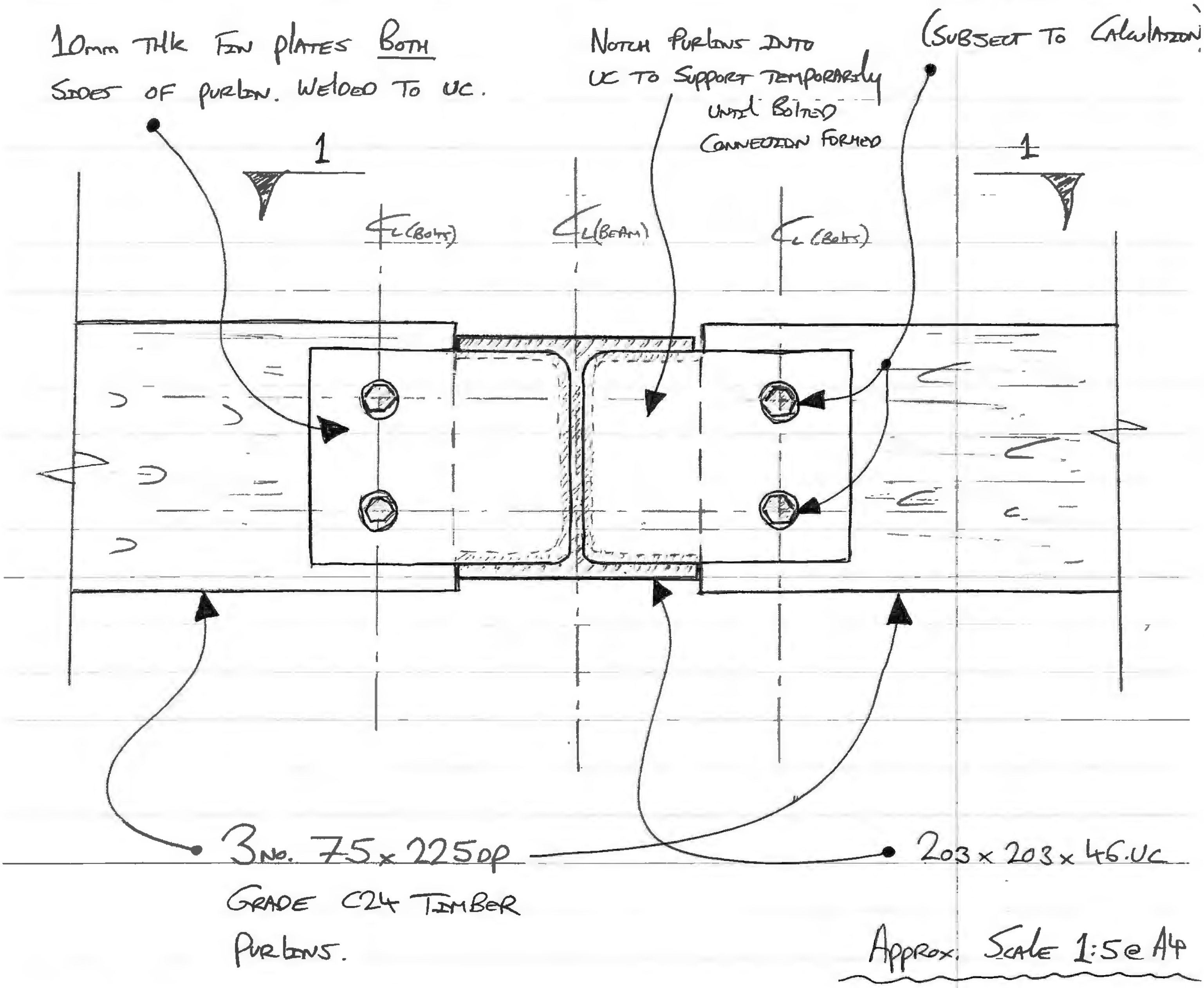
$K_{57} = 0.97$

Shear Perp. To Grain Per Bolt = 3.93 kN

Provide 2 Nº M16 Bolts (Shear Capacity Per Bolt · 4.32kN)

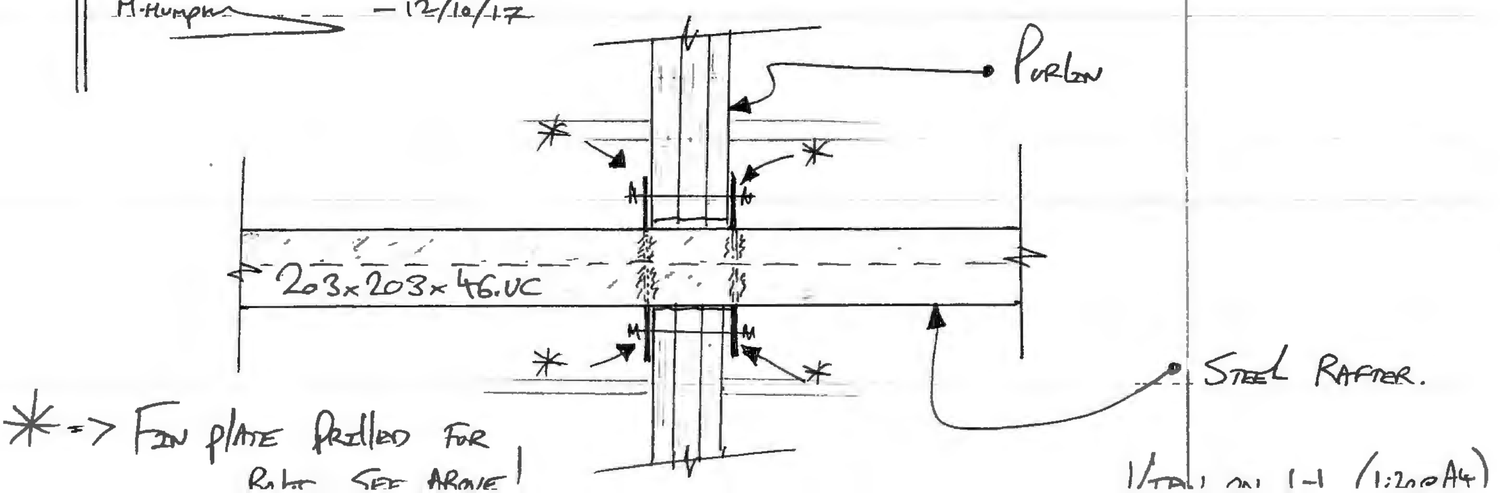
Made by	Job Title		
MJH	CARN GWAVEL, IOS.		
Checked by	**Job No.**	**Sheet**	**Date**
CO	16240	SK PC001	OCT '17

MBA CONSULTING



SECTION THRO' STEEL FRAME SHOWING PURLIN CONNECTION.

M.Humphr... – 12/10/17



Made by EP	Job Title CARN GWAVEL - ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 69	Date OCT. '17

MBA CONSULTING

DOOR HEAD BEAM

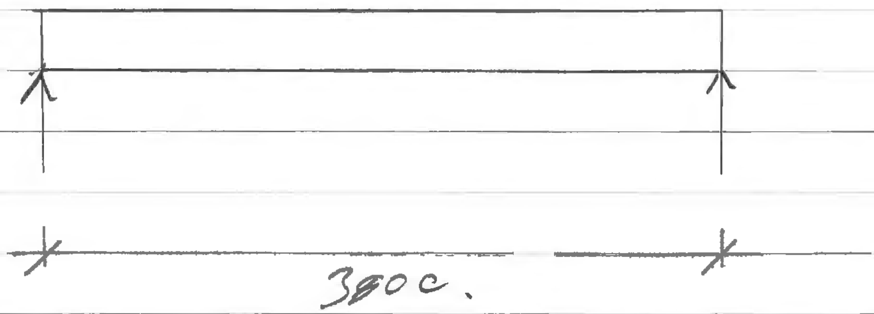
LOADINGS. kN/m

SLATE ROOF - DL = 2.15 × 7.5/2 = 8.07 (11.3)

IL = 0.6 × 7.5/2 = 1.50 (2.4)

FLAT ROOF - DL = 0.75 × 1.2/2 = 0.45 (0.63)

IL = 0.6 × 1.2/2 = 0.36 (0.58)



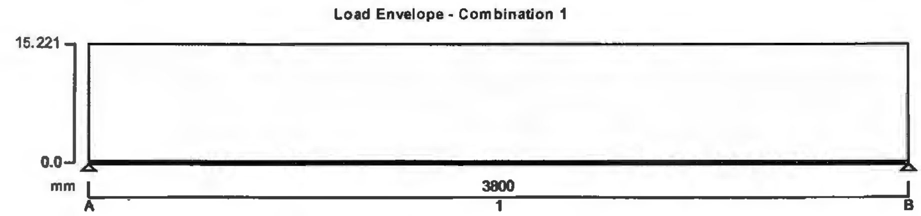
PROVIDE 203×102×23 UB IN S355 UNDER BOTH LEAVES OF MASONRY

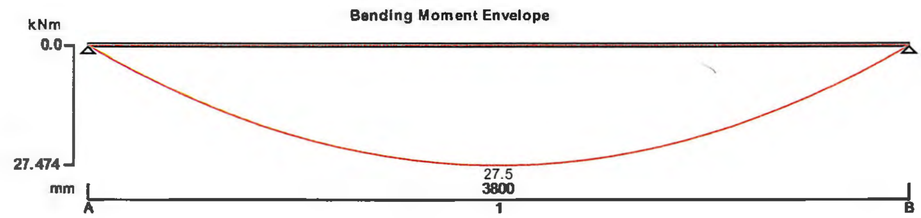
MBA CONSULTING	Project Carn Gwavel, Isles Of Scilly				Job no. 16240	
	Calcs for Door Head Beam				Start page no./Revision 70	
	Calcs by EP	Calcs date 13/11/2017	Checked by	Checked date	Approved by	Approved date

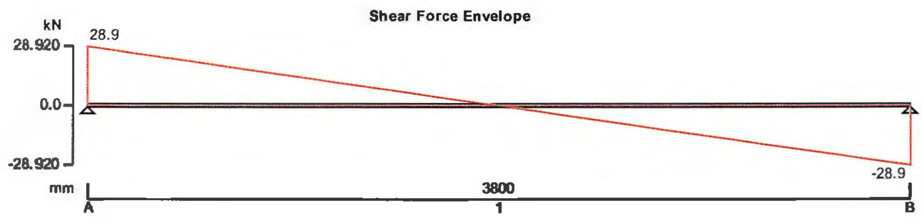
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







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 8.07 kN/m
	Imposed full UDL 1.5 kN/m
	Dead full UDL 0.45 kN/m
	Imposed full UDL 0.36 kN/m

Load combinations

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

MBA CONSULTING	Project: Carn Gwavel, Isles Of Scilly				Job no.: 16240	
	Calcs for: Door Head Beam				Start page no./Revision: 71	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

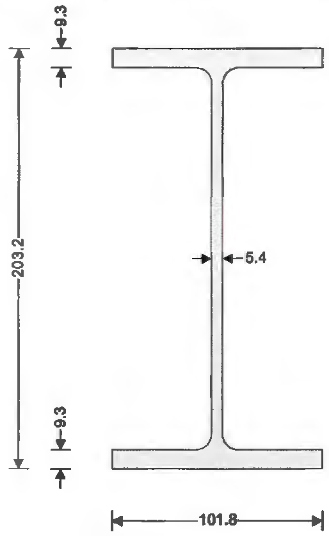
Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **27.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **28.9** kN	V_{min} = **-28.9** kN
Deflection	δ_{max} = **1.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **28.9** kN	R_{A_min} = **28.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **16.6** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **3.5** kN	
Maximum reaction at support B	R_{B_max} = **28.9** kN	R_{B_min} = **28.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **16.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **3.5** kN	

Section details

Section type **UKB 203x102x23 (Tata Steel Advance)** Steel grade **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **28.9** kN Design shear resistance P_v = **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **27.5** kNm Moment capacity low shear M_c = **83.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **10.556** mm Maximum deflection δ = **1.17** mm

PASS - Maximum deflection does not exceed deflection limit

Made by E.P	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 72	Date OCT 17

MBA CONSULTING

INTERNAL LINTELS.

LOADINGS

kN/m.

SLATE ROOF - DL = 2.15 × 7.5/2 = 8.07 (11.3)
IL = 0.6 × 7.5/2 = 2.25 (3.6)

ULTIMATE LOAD = 14.9 kN/m.

PROVIDE 100×110 HIGH STRENGTH PRE STRESSED CONCRETE LINTEL BY STRESSLINE LTD.

Made by	Job Title	Carn Gwavel, Isles Of Scilly	
EP			
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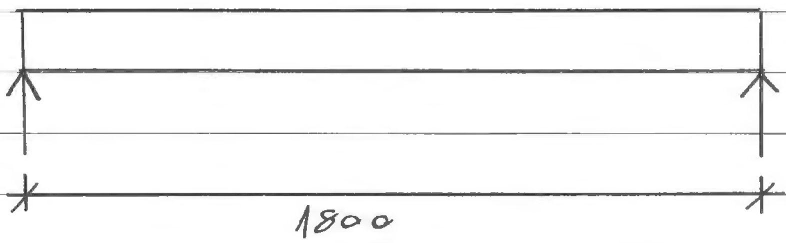
MBA CONSULTING

Internal Opening Extension

Loadings — kN/m

Slate Roof − DL $= 2.15 \times 7.5/2 = 8.07\ (11.3)$

IL $= 0.6 \times 7.5/2 = 2.25\ (3.6)$



Provide 203 × 102 × 23 UB Section In S355 Within Each Leaf

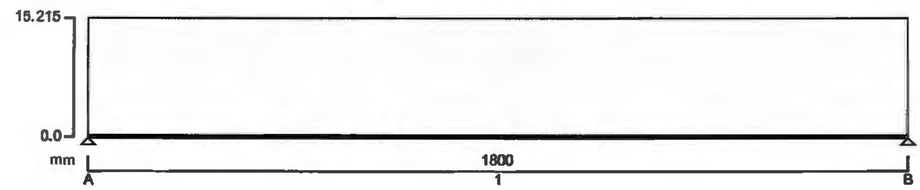
MBA CONSULTING	Project: Carn Gwavel, Isles Of Scilly			Job no.: 16240	
	Calcs for: Internal Opening Extension			Start page no./Revision: 74	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

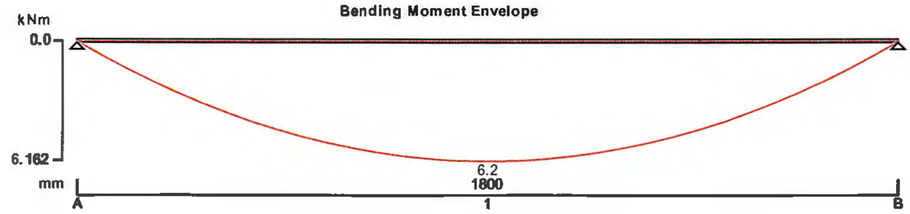
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05

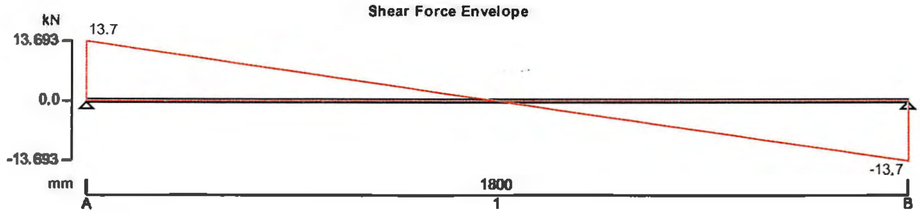
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



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 8.07 kN/m	
	Imposed full UDL 2.25 kN/m	

Load combinations

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

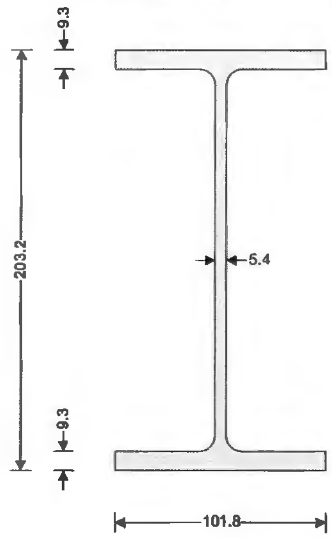
MBA CONSULTING	Project: Carn Gwavel, Isles Of Scilly			Job no.: 16240	
	Calcs for: Internal Opening Extension			Start page no./Revision: 75	
	Calcs by: EP	Calcs date: 13/11/2017	Checked by	Checked date	Approved by	Approved date

Analysis results

Maximum moment	M_{max} = **6.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **13.7** kN	V_{min} = **-13.7** kN
Deflection	δ_{max} = **0.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **13.7** kN	R_{A_min} = **13.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2** kN	
Maximum reaction at support B	R_{B_max} = **13.7** kN	R_{B_min} = **13.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2** kN	

Section details

Section type **UKB 203x102x23 (Tata Steel Advance)** Steel grade **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **13.7** kN Design shear resistance P_v = **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **6.2** kNm Moment capacity low shear M_c = **83.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **5** mm Maximum deflection δ = **0.071** mm

PASS - Maximum deflection does not exceed deflection limit

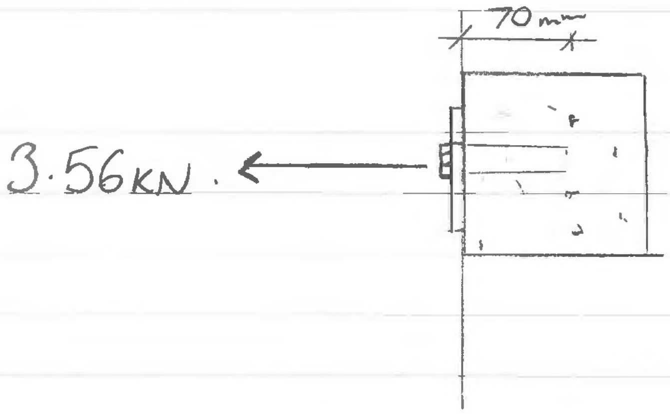
Made by	Job Title		
EP	CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 76	Date OCT 17

MBA CONSULTING

CANOPY COLUMN TO MASONRY CONNECTION

LOADING

WIND - $0.93 \times 3.0 \tan(20) \times \frac{5}{2} = 2.54\,kN$ (3.56 kN)



PROVIDE : HILTI HIT-V ANCHOR M12 ROD
SET IN HILTI HIT-HY200A INJ MORTAR
WITH 70 mm EMBED. TO C35 CONCRETE
PADSTONE .

Made by	Job Title: Carn Gwavel, Isles Of Scilly		
Checked by	Job No. 16240	Sheet 77	Date Oct. 17

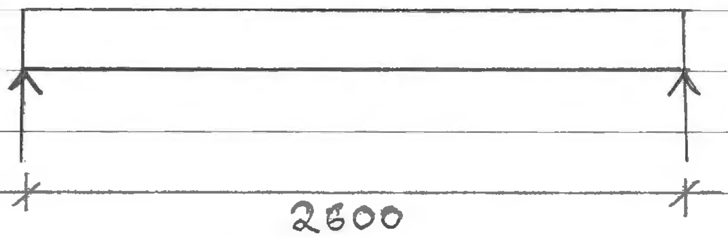
MBA CONSULTING

Canopy Area - Joists.

Loadings — kN/m

Flat Roof — $DL = \frac{0.60}{\cos(20)} \times 0.4 = 0.26$

$IL = 0.6 \times 0.4 = 0.24$



Provide 150x50 C16 Timber Joists @ 400mm c/c

MBA CONSULTING
Boscawen House, Chapel Hill Tel: 01872 260962
Truro, Cornwall TR1 3BN Fax: 01872 260963

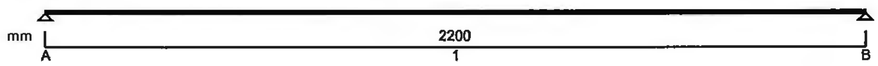
Project				Job no.	
Carn Gwavel, Isles of Scilly				16240	
Calcs for				Start page no./Revision	
Canopy Roof Joists				78	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	10/10/2017				

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

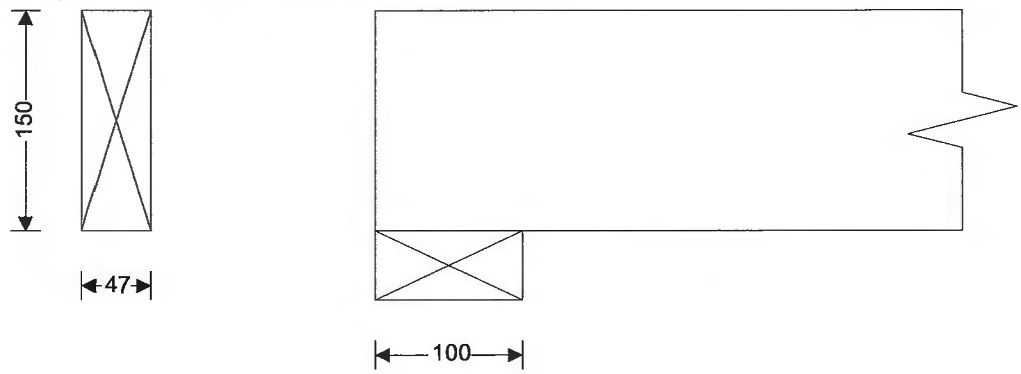
Joist details

Joist breadth	b = **47** mm	Joist depth	h = **150** mm
Joist spacing	s = **400** mm	Service class of timber	**1**
Timber strength class	**C16**		



Span details

Number of spans	N_{span} = **1**	Length of bearing	L_b = **100** mm
Clear length of span	L_{s1} = **2200** mm		



Section properties

Second moment of area	I = **13218750** mm^4	Section modulus	Z = **176250** mm^3

Loading details

Joist self weight	F_{swt} = **0.02** kN/m	Dead load	F_{d_udl} = **0.62** kN/m^2
Imposed UDL(Medium term)	F_{i_udl} = **0.60** kN/m^2		
Imposed point load (Short)	F_{i_pt} = **0.90** kN		

Consider medium term loads

Design bending moment	M = **0.308** kNm	Design shear force	V = **0.560** kN
Design support reaction	R = **0.560** kN	Design deflection	δ = **1.431** mm

Check bending stress

Permissible bending stress	σ_{m_adm} = **7.865** N/mm^2	Applied bending stress	σ_{m_max} = **1.749** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress	τ_{adm} = **0.921** N/mm^2	Applied shear stress	τ_{max} = **0.119** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.025** N/mm^2 — Applied bearing stress σ_{c_max} = **0.119** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **6.600** mm — Actual deflection δ = **1.431** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Design bending moment M = **0.658** kNm — Design shear force V = **1.196** kN

Design support reaction R = **1.196** kN — Design deflection δ = **2.626** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **9.438** N/mm^2 — Applied bending stress σ_{m_max} = **3.733** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.106** N/mm^2 — Applied shear stress τ_{max} = **0.255** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.630** N/mm^2 — Applied bearing stress σ_{c_max} = **0.255** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **6.600** mm — Actual deflection δ = **2.626** mm

PASS - Actual deflection within permissible limits

Made by EP	Job Title CARN GWAVEL, ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 80	Date OCT. '17

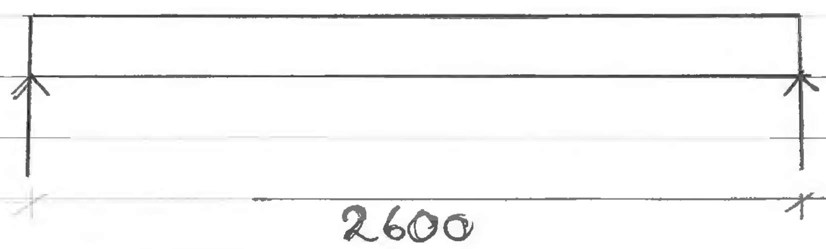
MBA CONSULTING

CANOPY AREA – TIMBER BEAMS

LOADINGS – KN/m

FLAT ROOF – $DL = \frac{0.70}{\cos(20)} \times 3/2 = 1.20$

$IL = 0.6 \times 3/2 = 0.9$



PROVIDE 2 No 150 x 50 C24 TIMBER BEAMS.

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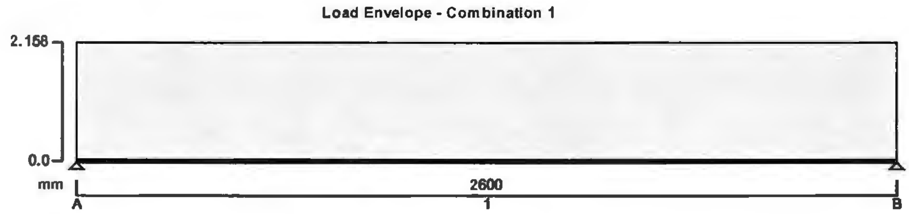
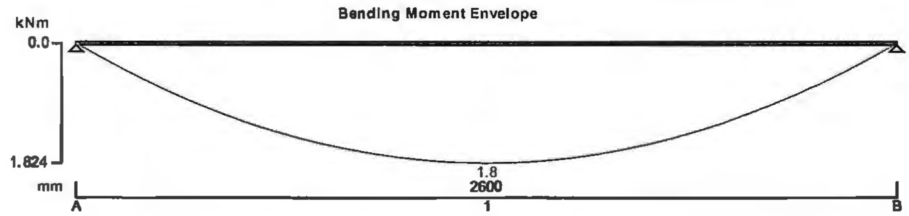
Tel: 01872 260962
Fax: 01872 260963

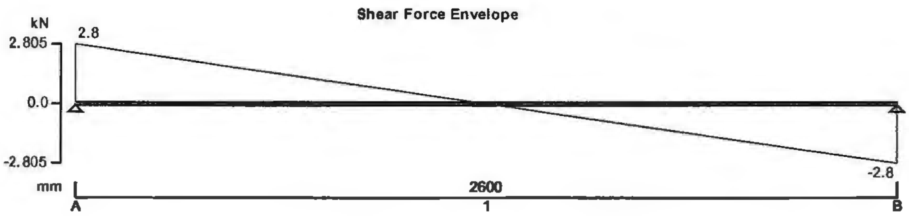
Project				Job no.	
Carn Gwavel, Isles of Scilly				16240	
Calcs for				Start page no./Revision	
Canopy Area - Beams				81	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	10/10/2017				

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.200 kN/m
Imposed full UDL 0.900 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **1.824** kNm	M_{min} = **0.000** kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **1.824** kNm	
Maximum shear	F_{max} = **2.805** kN	F_{min} = **-2.805** kN
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **2.805** kN	
Total load on beam	W_{tot} = **5.611** kN	

Reactions at support A R_{A_max} = **2.805** kN R_{A_min} = **2.805** kN

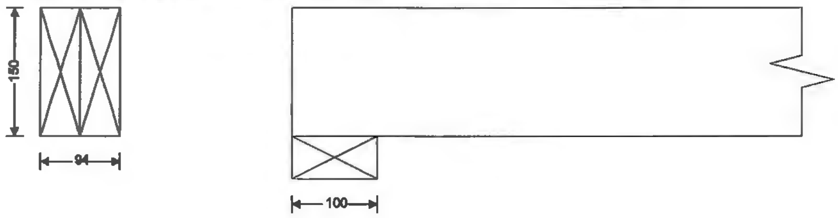
Unfactored dead load reaction at support A R_{A_Dead} = **1.635** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **1.170** kN

Reactions at support B R_{B_max} = **2.805** kN R_{B_min} = **2.805** kN

Unfactored dead load reaction at support B R_{B_Dead} = **1.635** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **1.170** kN



Timber section details

Breadth of sections b = **47** mm

Depth of sections h = **150** mm

Number of sections in member N = **2**

Overall breadth of member $b_b = N \times b$ = **94** mm

Timber strength class **C24**

Member details

Service class of timber **2**

Load duration **Short term**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **14100** mm²

Section modulus $Z_x = N \times b \times h^2 / 6$ = **352500** mm³

$Z_y = h \times (N \times b)^2 / 6$ = **220900** mm³

Second moment of area $I_x = N \times b \times h^3 / 12$ = **26437500** mm⁴

$I_y = h \times (N \times b)^3 / 12$ = **10382300** mm⁴

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **43.3** mm

$i_y = \sqrt{(I_y / A)}$ = **27.1** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.50**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.08**

Load sharing - cl.2.9 K_8 = **1.00**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19 **2.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.60**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.600** N/mm²

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.298** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.083**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **12.141** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **5.173** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.426**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **1.065** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.298** N/mm^2

τ_a / τ_{adm} = **0.280**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **7200** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1})$ = **7.800** mm

Bending deflection δ_{b_s1} = **6.746** mm

Shear deflection δ_{v_s1} = **0.345** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **7.091** mm

δ_a / δ_{adm} = **0.909**

PASS - Total deflection is less than permissible deflection

Made by EP	Job Title CARN GWAVEL - ISLES OF SCILLY		
Checked by	Job No. 16240	Sheet 84	Date OCT '17

MBA CONSULTING

CANOPY AREA - TIMBER COLUMN

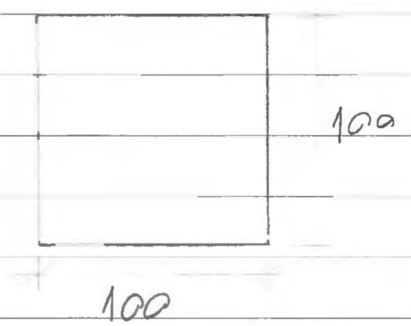
LOADINGS kN.

FLAT ROOF - DL = $\frac{0.75}{\cos 20} \times \frac{2 \times 2.7}{4} = 1.08$

IL = $0.6 \times \frac{2 \times 2.7}{4} = 0.81$

Σ 1.89

DISTANCE BETWEEN RESTRAINTS = 3700mm MAX.



AS ARCHITECT REQUESTS CIRCULAR COLUMN:

$I = \frac{bd^3}{12} = 8.3 \times 10^6 \ mm^4$

$\Rightarrow 8.3 \times 10^6 = \frac{\pi r^4}{4} \Rightarrow r > 57 mm$

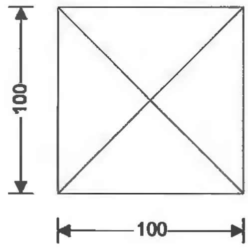
PROVIDE 150 Ø CIRCULAR COLUMN IN C24 TIMBER

TIMBER MEMBER DESIGN TO BS5268-2:2002

TEDDS calculation version 1.6.00

Analysis results

Design axial compression P = **1.890** kN



Timber section details

Breadth of section b = **100** mm
Depth of section h = **100** mm
Number of sections N = **1**
Breadth of beam b_b = **100** mm
Timber strength class **C24**

Member details

Service class of timber **1**
Load duration **Medium term**
Unbraced length in x-axis L_x = **3700** mm
Unbraced length in y-axis L_y = **3700** mm
Effective length factor in x-axis K_x = **1**
Effective length factor in y-axis K_y = **1**
Effective length in x-axis L_{ex} = **3700** mm
Effective length in y-axis L_{ey} = **3700** mm

Slenderness ratio - cl.2.11.4

Slenderness ratio λ = **128.172**
Permissible slenderness ratio λ_{max} = **180**

PASS - Slenderness ratio is less than permissible slenderness ratio

Compression parallel to grain

Permissible comp.stress σ_{c_adm} = **2.317** N/mm^2
Applied compressive stress σ_{c_a} = **0.189** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress

CANOPY AREA - FOUNDATIONS.

LOADINGS

KN

FLAT ROOF - DL $= \frac{0.75}{\cos 20} \times \frac{2 \times 2.7}{4} = 1.08$

IL $= 0.6 \times \frac{2 \times 2.7}{4} = 0.81$

TIMBER COL. S/W - DL $= 4.2 \times 0.075^2 \pi \times 3.7 = 0.28$

FOUNDATION S/W - DL $= 0.6 \times 0.6 \times 0.6 \times 24 = 5.19$

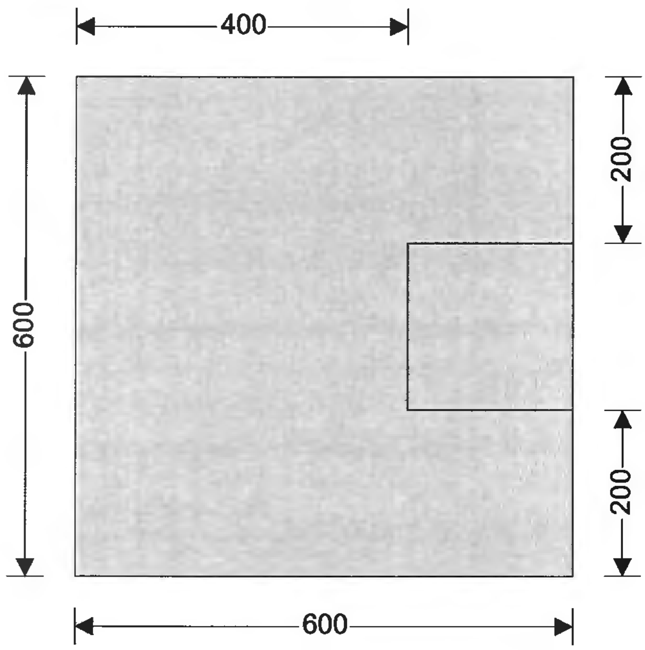
Σ 7.36 kN

PROVIDE 600x600 x450 MASS CONCRETE PAD FOUNDATION IN C20

MBA CONSULTING	Project			Job no.	
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN Tel: 01872 260962 Fax: 01872 260963	Carn Gwavel, Isles of Scilly			16240	
	Calcs for			Start page no./Revision	
	Canopy Pad Foundations			87	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	EP	10/10/2017				

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

TEDDS calculation version 2.0.07



Pad footing details

Length of pad footing $L = \mathbf{600}$ mm — Width of pad footing $B = \mathbf{600}$ mm

Depth of pad footing $h = \mathbf{450}$ mm — Depth of soil over pad footing $h_{soil} = \mathbf{200}$ mm

Density of concrete $\rho_{conc} = \mathbf{23.6}$ kN/m^3

Column details

Column base length $l_A = \mathbf{200}$ mm — Column base width $b_A = \mathbf{200}$ mm

Column eccentricity in x $e_{PxA} = \mathbf{200}$ mm — Column eccentricity in y $e_{PyA} = \mathbf{0}$ mm

Soil details

Depth of soil over pad footing $h_{soil} = \mathbf{200}$ mm — Density of soil $\rho_{soil} = \mathbf{20.0}$ kN/m^3

Allowable bearing pressure $P_{bearing} = \mathbf{150}$ kN/m^2

Axial loading on column

Dead axial load $P_{GA} = \mathbf{1.4}$ kN — Imposed axial load $P_{QA} = \mathbf{0.8}$ kN

Wind axial load $P_{WA} = \mathbf{0.0}$ kN — Total axial load $P_A = \mathbf{2.2}$ kN

Foundation loads

Dead surcharge load $F_{Gsur} = \mathbf{0.000}$ kN/m^2 — Imposed surcharge load $F_{Qsur} = \mathbf{0.000}$ kN/m^2

Pad footing self weight $F_{swt} = \mathbf{10.620}$ kN/m^2

Soil self weight $F_{soil} = \mathbf{4.000}$ kN/m^2 — Total foundation load $F = \mathbf{5.3}$ kN

Calculate pad base reaction

Total base reaction $T = \mathbf{7.4}$ kN

Base reaction eccentricity in x $e_{Tx} = \mathbf{58}$ mm — Base reaction eccentricity in y $e_{Ty} = \mathbf{0}$ mm

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = \mathbf{8.592}$ kN/m^2 — $q_2 = \mathbf{8.592}$ kN/m^2 — $q_3 = \mathbf{32.703}$ kN/m^2 — $q_4 = \mathbf{32.703}$ kN/m^2

Minimum base pressure $q_{min} = \mathbf{8.592}$ kN/m^2 — Maximum base pressure $q_{max} = \mathbf{32.703}$ kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

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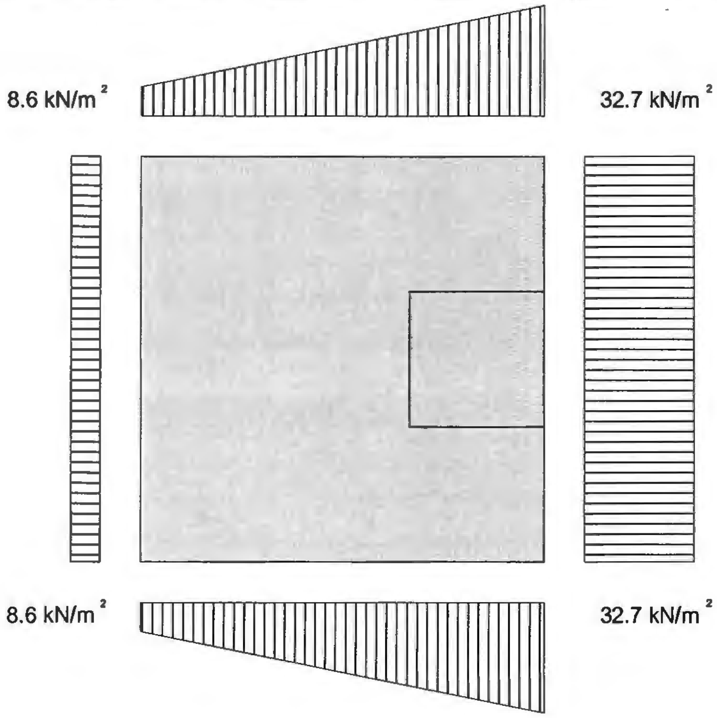
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN
Tel: 01872 260962
Fax: 01872 260963

Project				Job no.	
Carn Gwavel, Isles of Scilly				16240	
Calcs for				Start page no./Revision	
Canopy Pad Foundations				88	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EP	10/10/2017				



Material details

Char.strength of concrete f_{cu} = **20** N/mm²

Calculate minimum depth of unreinforced pad footing

Ave.pressure to left of footing q_L = **16.629** kN/m²
Ave.pressure to right of footing q_R = **32.703** kN/m²
Ave.pressure to top of footing q_T = **20.648** kN/m²
Ave.pressure to btm of footing q_B = **20.648** kN/m²
Min.depth unreinforced footing h_{min} = **400** mm

Min.depth to left of footing h_{Lmin} = **400** mm
Min.depth to right of footing h_{Rmin} = **0** mm
Min.depth to top of footing h_{Tmin} = **200** mm
Min.depth to btm of footing h_{Bmin} = **200** mm

PASS - Unreinforced pad footing depth is greater than minimum

Made by EP	Job Title Carn Gwavel, Isles Of Scilly		
Checked by	Job No. 16240	Sheet 89	Date Nov '17

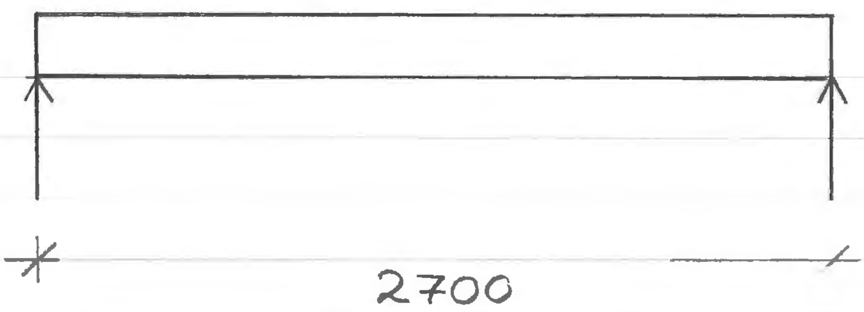
MBA CONSULTING

Extenal Shelter Roof Joists

Loadings.

KN/m.

Flat Roof - DL = 0.7 x 0.4 = 0.28
IL = 0.6 x 0.4 = 0.24.



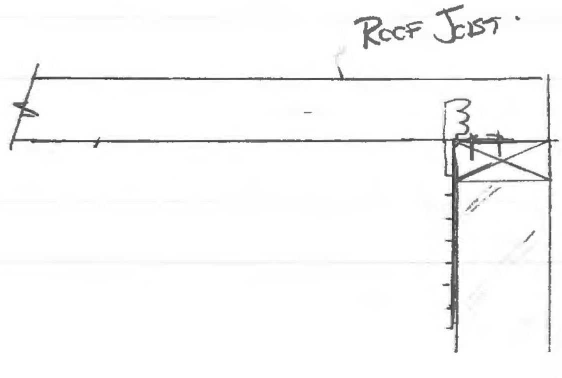
Wind Uplift To Each Joist = 0.61 x 0.4 x 2.7/2 = 0.33 kN.

Provide 150 x 50 C24 Timber Joists @ 400 mm c/c.

Fix Roof Joists Via Simpson Strong Tie L10B10 Common Bent LL Straps With Fixings To Manufacturers Req.

Strap span 4 block.

Resis. Of Masonry From Uplift = 18 x 0.44 x 0.215 x 0.1 x 4 = 0.68 kN > 0.33 kN ∴ OK.



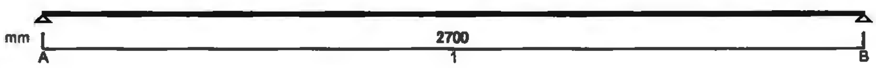
MBA CONSULTING	Project: Carn Gwavel, Isles of Scilly				Job no.: 16240	
Boscawen House, Chapel Hill, Truro, Cornwall TR1 3BN. Tel: 01872 260962 Fax: 01872 260963	Calcs for: External Shelter				Start page no./Revision: 90	
	Calcs by: EP	Calcs date: 02/11/2017	Checked by	Checked date	Approved by	Approved date

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

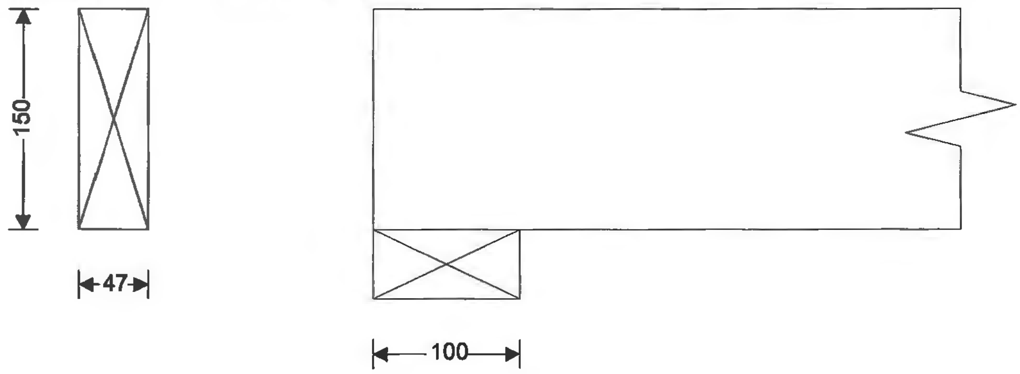
Joist details

Joist breadth	b = **47** mm	Joist depth	h = **150** mm
Joist spacing	s = **400** mm	Service class of timber	**1**
Timber strength class	**C24**		



Span details

Number of spans	N_{span} = **1**	Length of bearing	L_b = **100** mm
Clear length of span	L_{s1} = **2700** mm		



Section properties

Second moment of area	I = **13218750** mm^4	Section modulus	Z = **176250** mm^3

Loading details

Joist self weight	F_{swt} = **0.02** kN/m	Dead load	F_{d_udl} = **0.70** kN/m^2
Imposed UDL(Medium term)	F_{i_udl} = **0.60** kN/m^2		
Imposed point load (Short)	F_{i_pt} = **0.90** kN		

Consider medium term loads

Design bending moment	M = **0.496** kNm	Design shear force	V = **0.735** kN
Design support reaction	R = **0.735** kN	Design deflection	δ = **2.763** mm

Check bending stress

Permissible bending stress	σ_{m_adm} = **11.130** N/mm^2	Applied bending stress	σ_{m_max} = **2.814** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress	τ_{adm} = **0.976** N/mm^2	Applied shear stress	τ_{max} = **0.156** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.300** N/mm^2 Applied bearing stress σ_{c_max} = **0.156** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **8.100** mm Actual deflection δ = **2.763** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Design bending moment M = **0.885** kNm Design shear force V = **1.311** kN

Design support reaction R = **1.311** kN Design deflection δ = **4.283** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **13.355** N/mm^2 Applied bending stress σ_{m_max} = **5.020** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.172** N/mm^2 Applied shear stress τ_{max} = **0.279** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.960** N/mm^2 Applied bearing stress σ_{c_max} = **0.279** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **8.100** mm Actual deflection δ = **4.283** mm

PASS - Actual deflection within permissible limits