| Finite engineering<br>Consulting Civil and Structural Engineers<br>17 Marchfield Place, Dumfries, DG1 1GQ<br>Director: A Morrow CEng MICE MIStructE MCIHT<br>[redacted] | Whithorn Roundhouse | | | | Job Ref. 160209 | |
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| | Status | Building Warrant Application | | | Sheet no./rev.<br>1 | |
| | Calc. by<br>A.M | Date<br>10 April 2016 | Chk'd by | Date | App'd by | |

## Calculations for:

# Structural Calculations for New Timber Roundhouse, Whithorn

## Calculations by:

Finite engineering

CONSULTING CIVIL & STRUCTURAL ENGINEERS

[redacted]

10th April 2016

Building Standards Approved Certifier

The Scottish Government

# CONTENTS

| finite engineering<br>*Consulting Civil and Structural Engineers*<br>*17 Marchfield Place, Dumfries, DG1 1GQ*<br>*Director: A Morrow CEng MICE MIStructE MCIHT*<br>[redacted] | Whithorn Roundhouse | | | | Job Ref. 160209 | |
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| | Status | Building Warrant Application | | | Sheet no./rev. 4 | |
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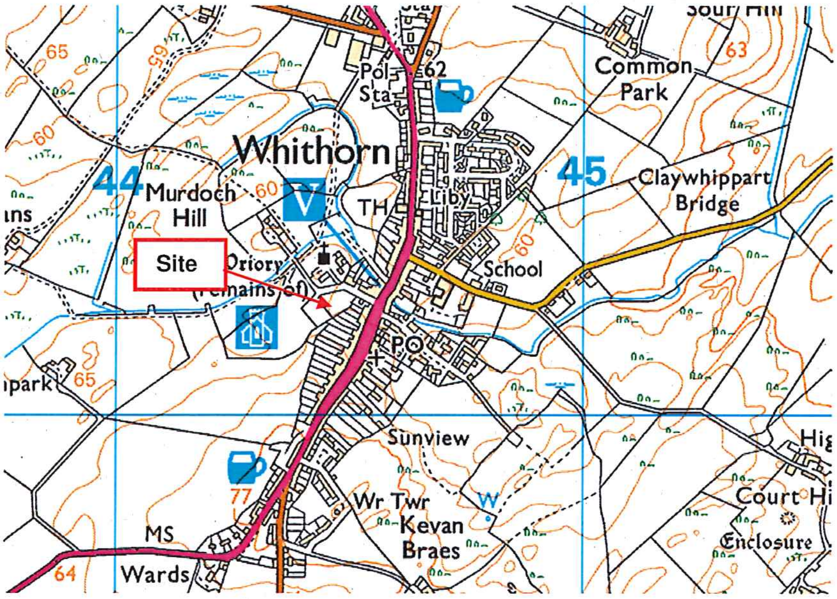
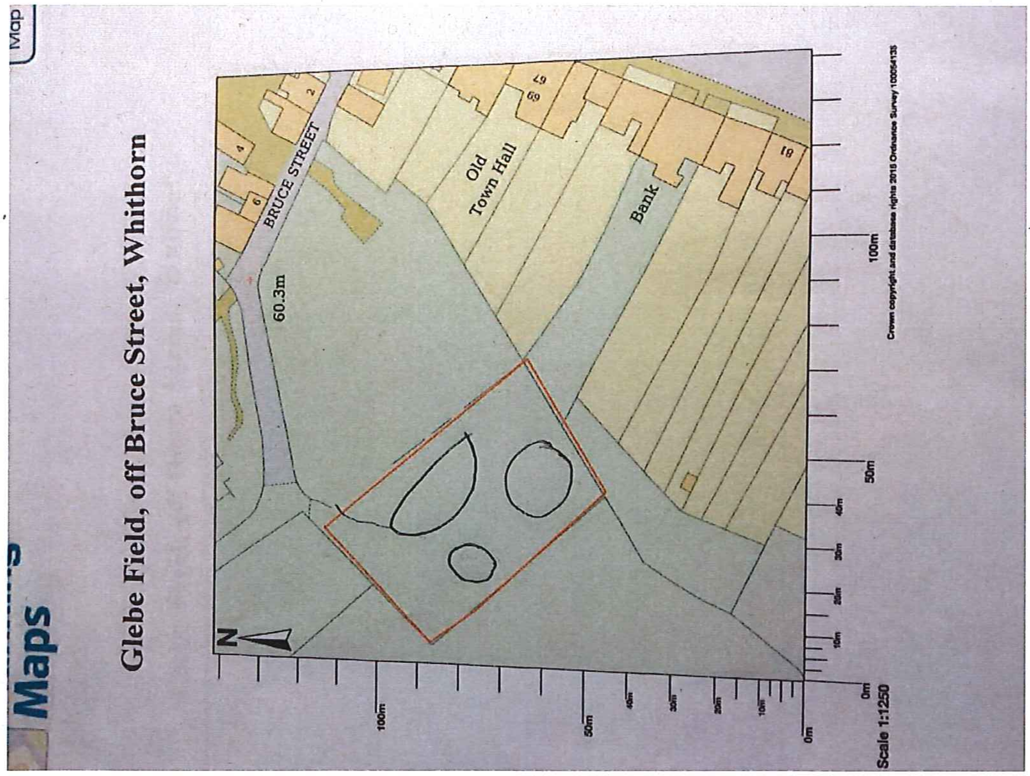
## SITE LOCATION




Site 61m AOD and 4km from sea

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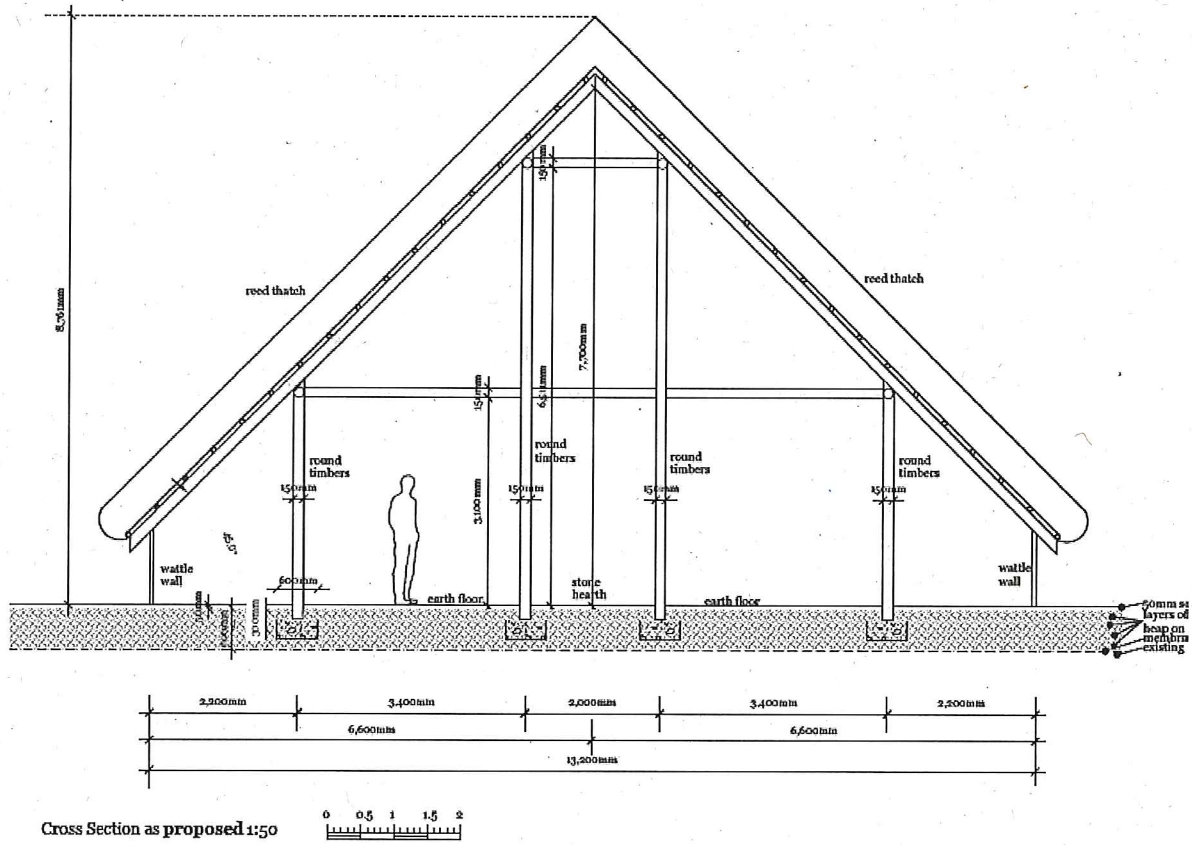
# ARCHITECTURAL DETAILS




Cross Section as proposed 1:50

| | | | | | |
|---|---|---|---|---|---|
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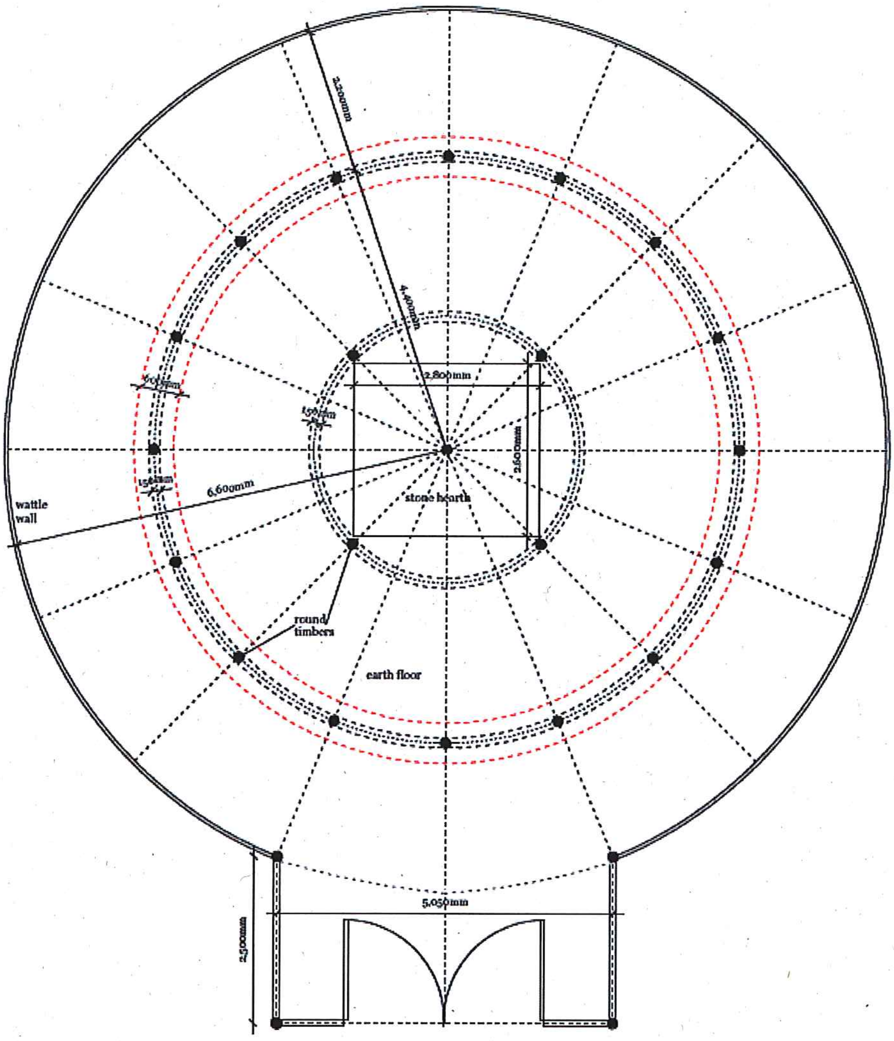


**PROPOSED ROUNDHOUSE MODEL**

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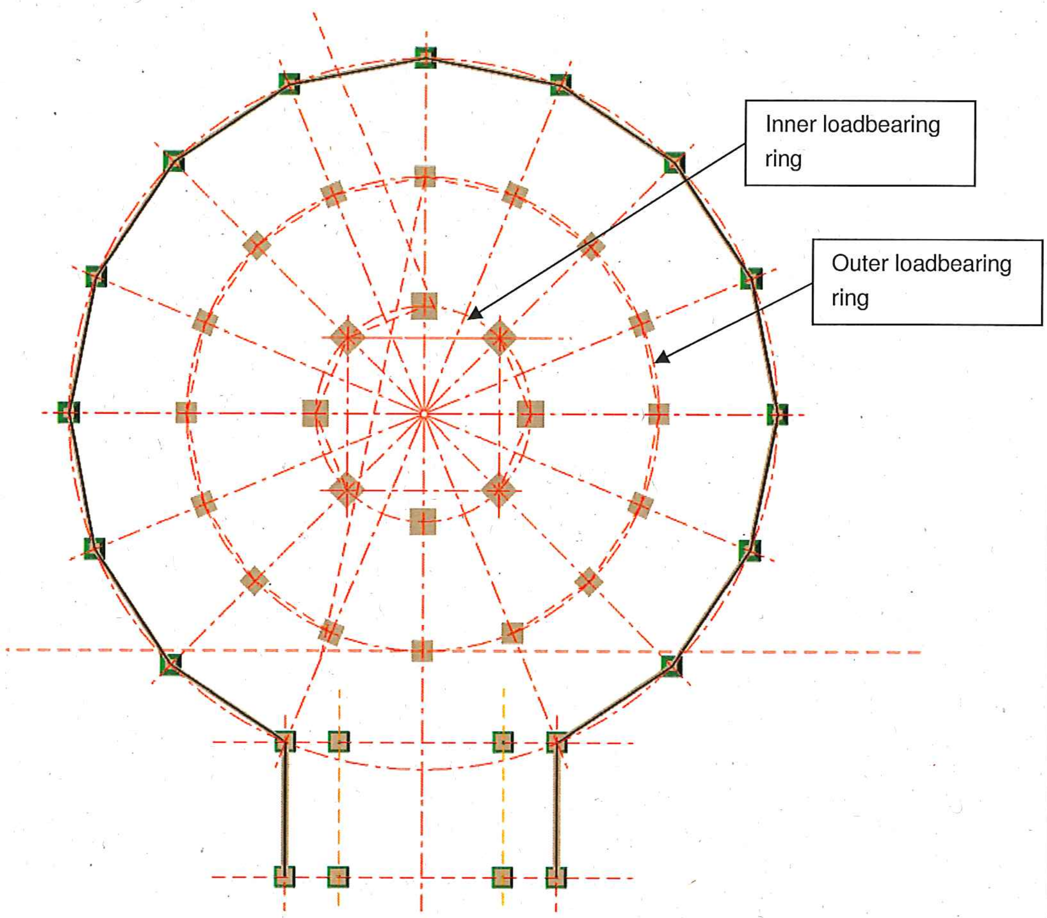
# PROPOSED DESIGN MODEL

## BASE LAYOUT



| | | | | | | |
|---|---|---|---|---|---|---|
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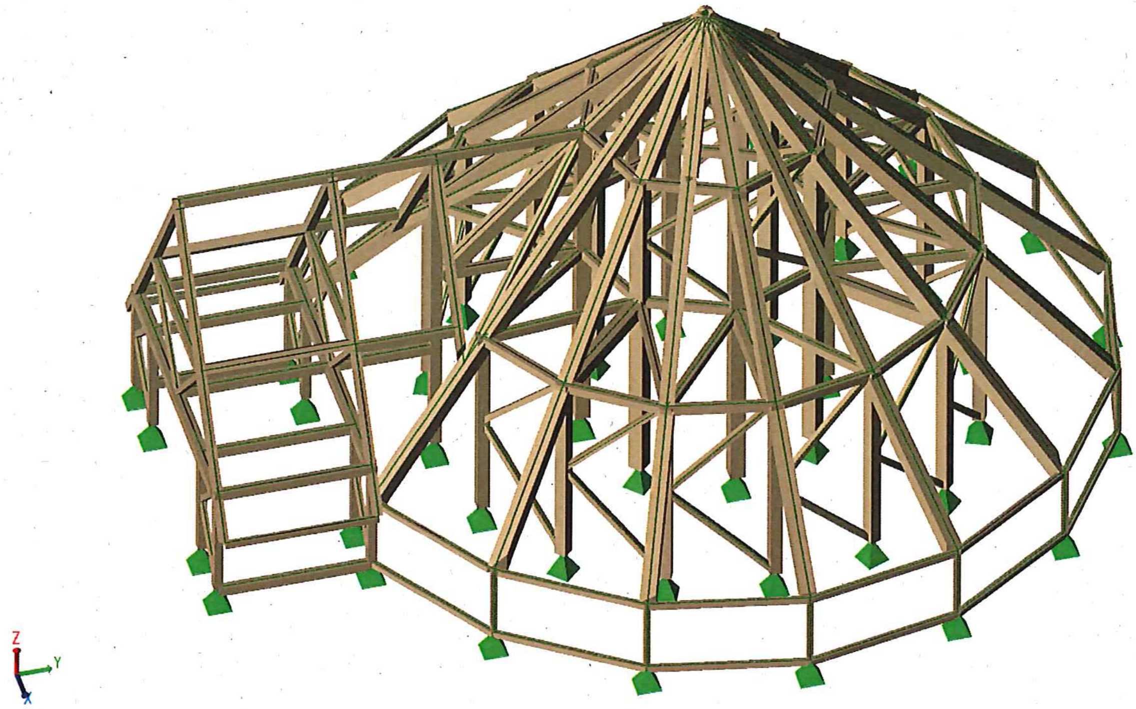
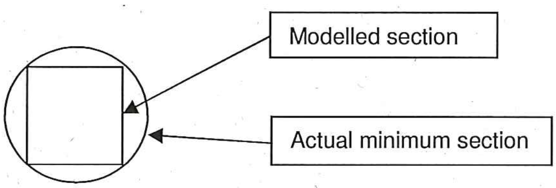



For analysis timbers modelled as square sections although understood actual timbers will generally be circular in section. Assumed that circular timbers used will be larger than the section modelled therefore analysis will be slightly conservative in this respect.



<u>Main Member Sizes</u>

All timber within model taken as low grade C14 timber.

| | |
|---|---|
| Rafters: | 250mm x 250mm timbers |
| Inner ring columns: | 300mm x 300mm timbers |
| Outer ring columns: | 250mm x 250mm timbers |
| Outer wall rafter supports: | 100mm x 100mm timbers |
| Bracing to walls | 100mm x 100mm timbers |

| | | | | | |
|---|---|---|---|---|---|
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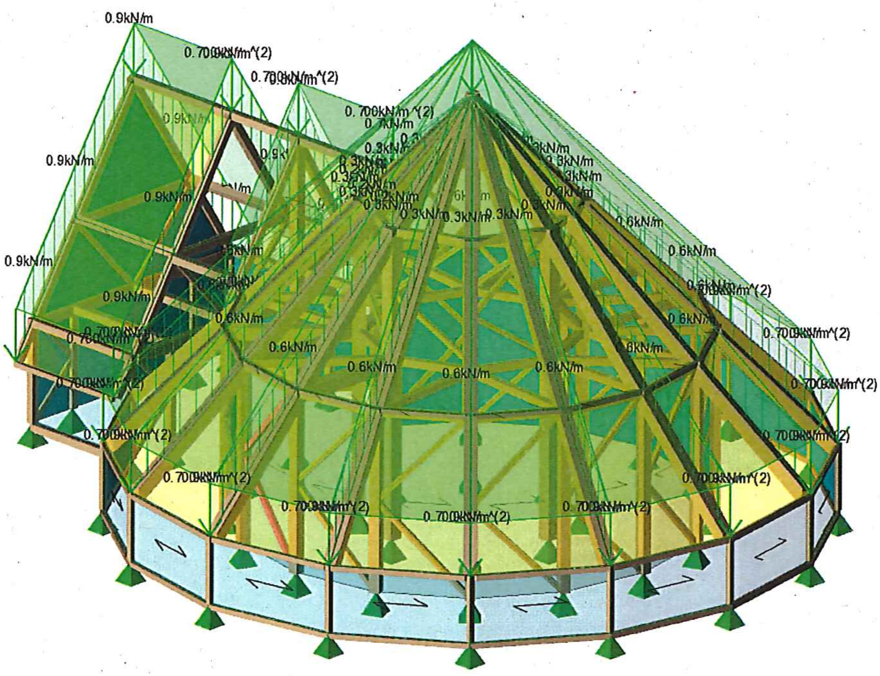
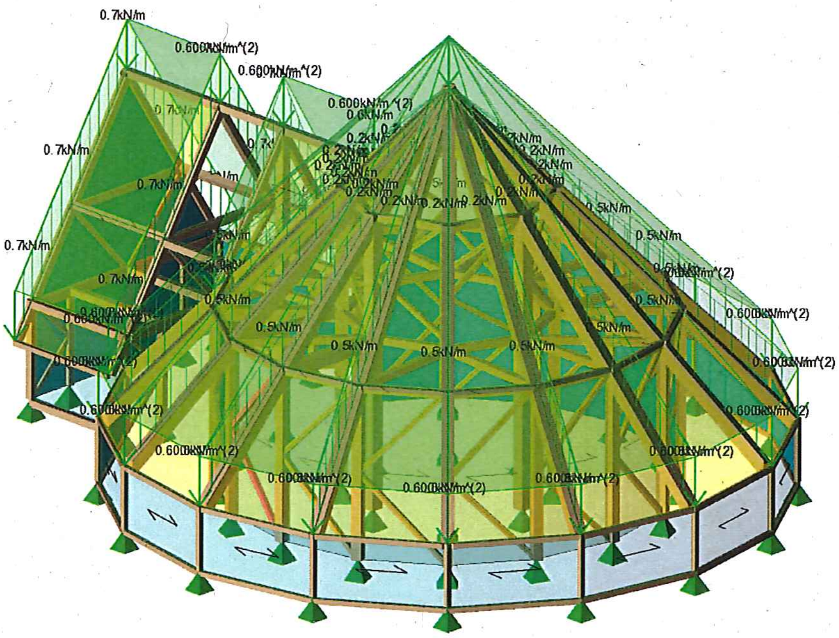
# APPLIED LOADINGS




Permanent Thatch Loading **0.7** kN/m²




Variable Imposed Loading – **0.6** kN/m²

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*Director: A Morrow CEng MICE MIStructE MCIHT*
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| | | | | | |
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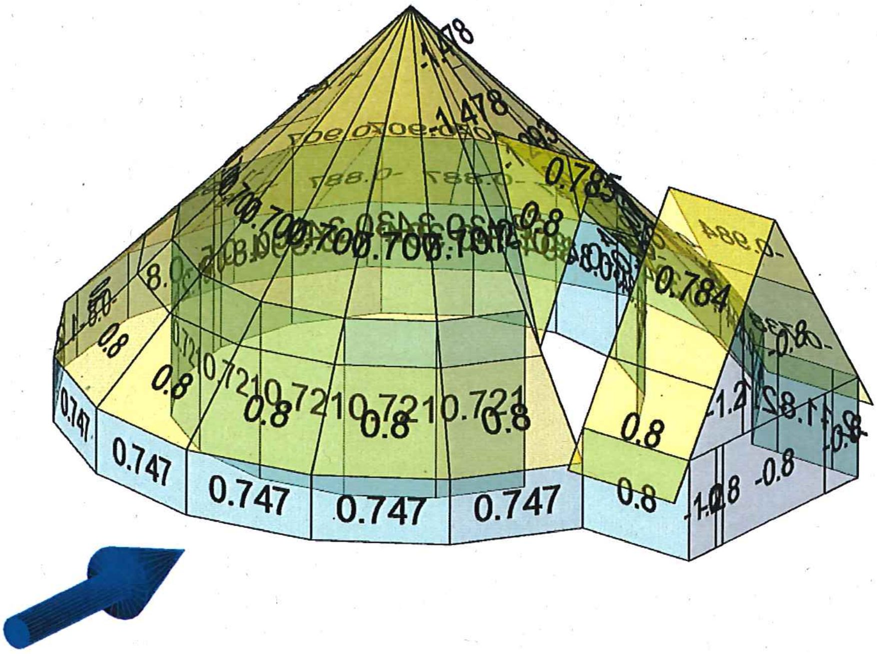
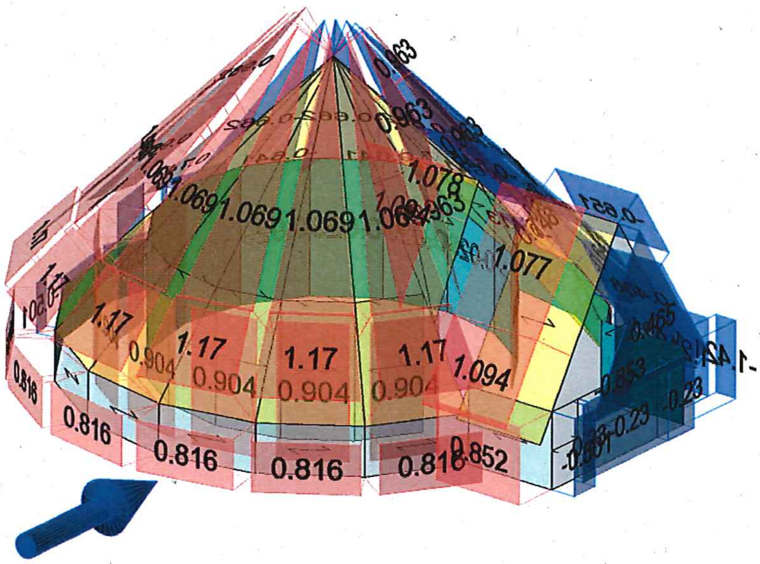



Wind zones from wind acting on side of structure

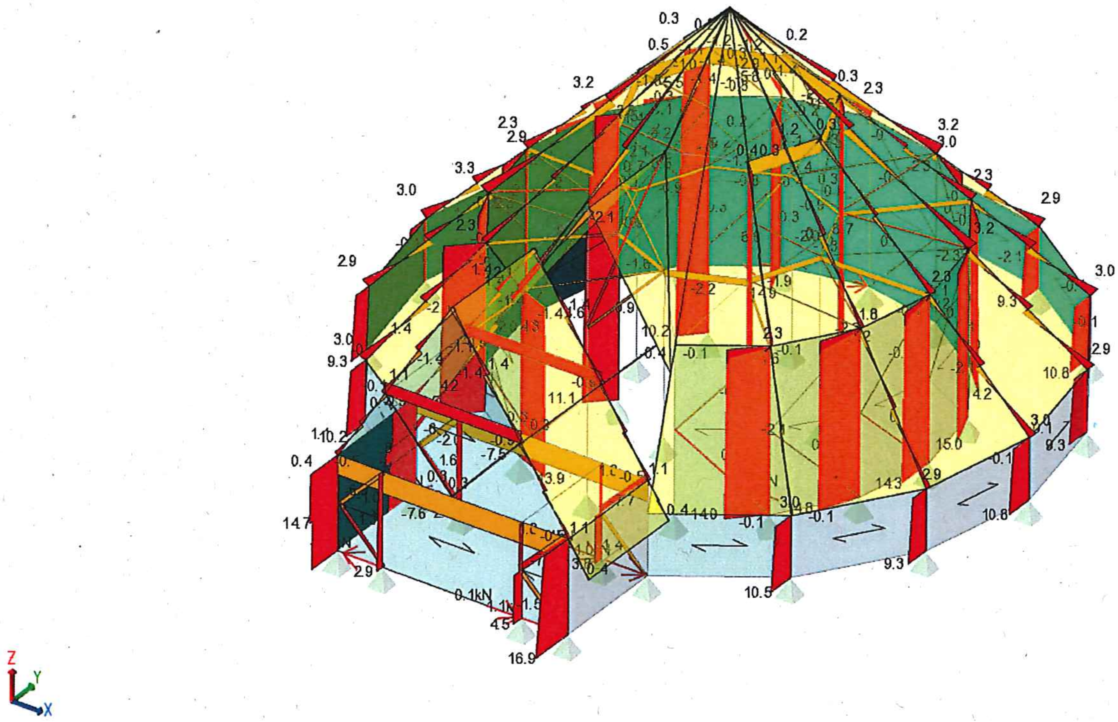



Appied wind loads from wind acting on side of structure

# ANALYSIS RESULTS FOR 'DEAD + IMPOSED' COMBINATION

## MEMBER AXIAL FORCES

AxialForce1D min/max=-7.7/16.9kN



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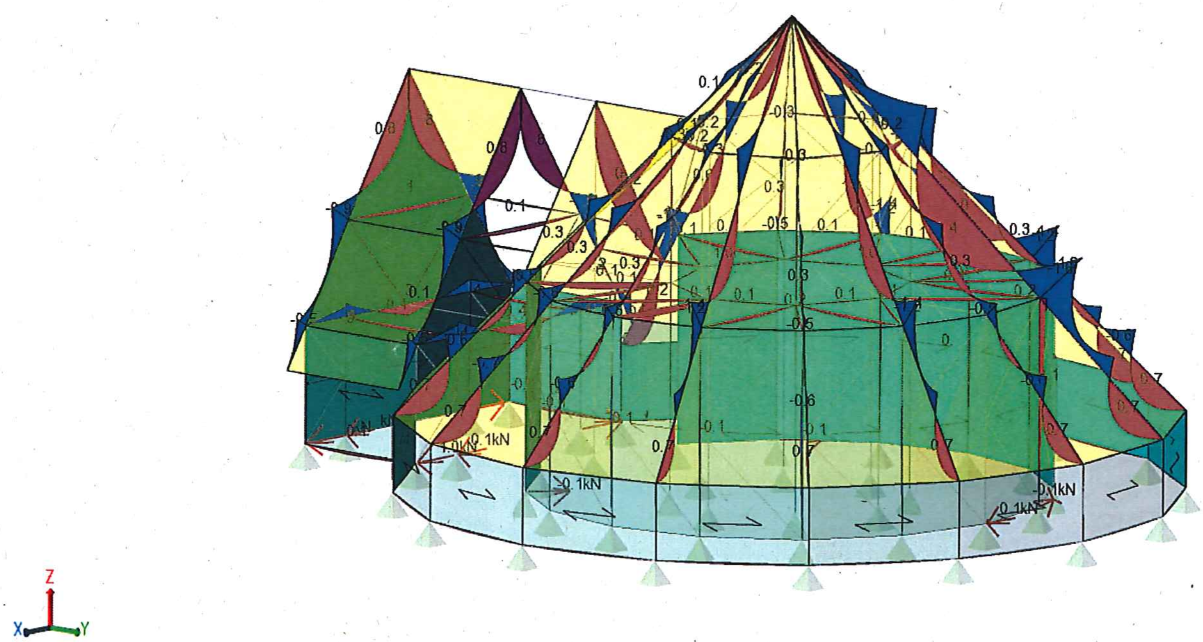
## MEMBER BENDING MOMENTS

MomentMajor1D min/max=-1.8/1.4kNm



| | | | | | | |
|---|---|---|---|---|---|---|
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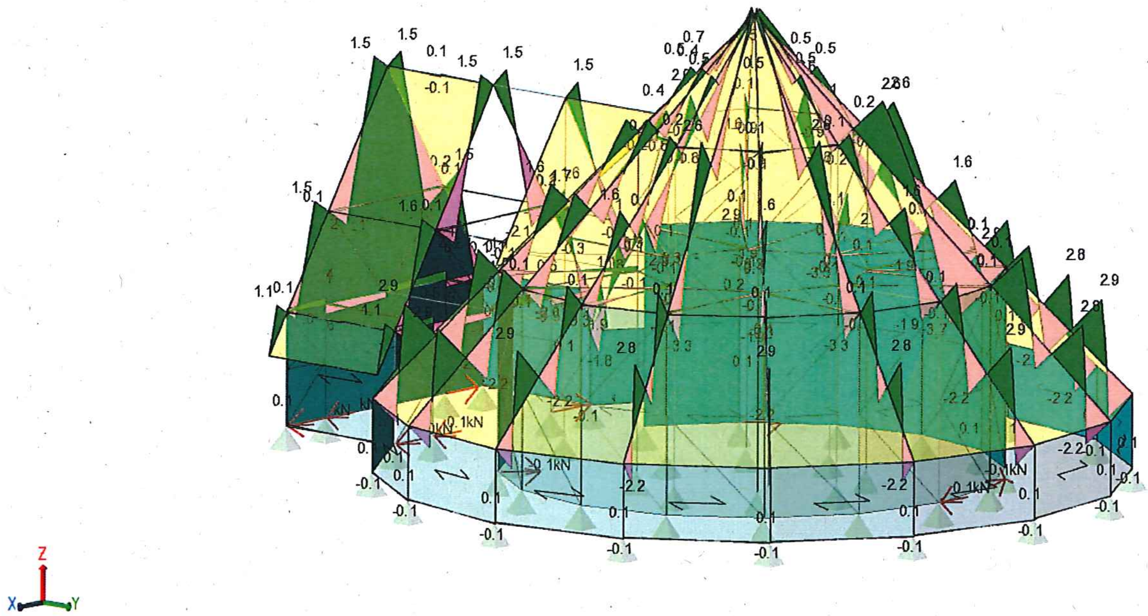
| | Calc. by<br>A.M | Date<br>10 April 2016 | Chk'd by | Date | App'd by | |

## MEMBER SHEAR FORCES

ShearMajor1D min/max=-3.7/2.9kN



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Director: A Morrow CEng MICE MIStructE MCIHT
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| Whithorn Roundhouse | | | | Job Ref. 160209 | |
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| Status | Building Warrant Application | | | Sheet no./rev. 15 | |
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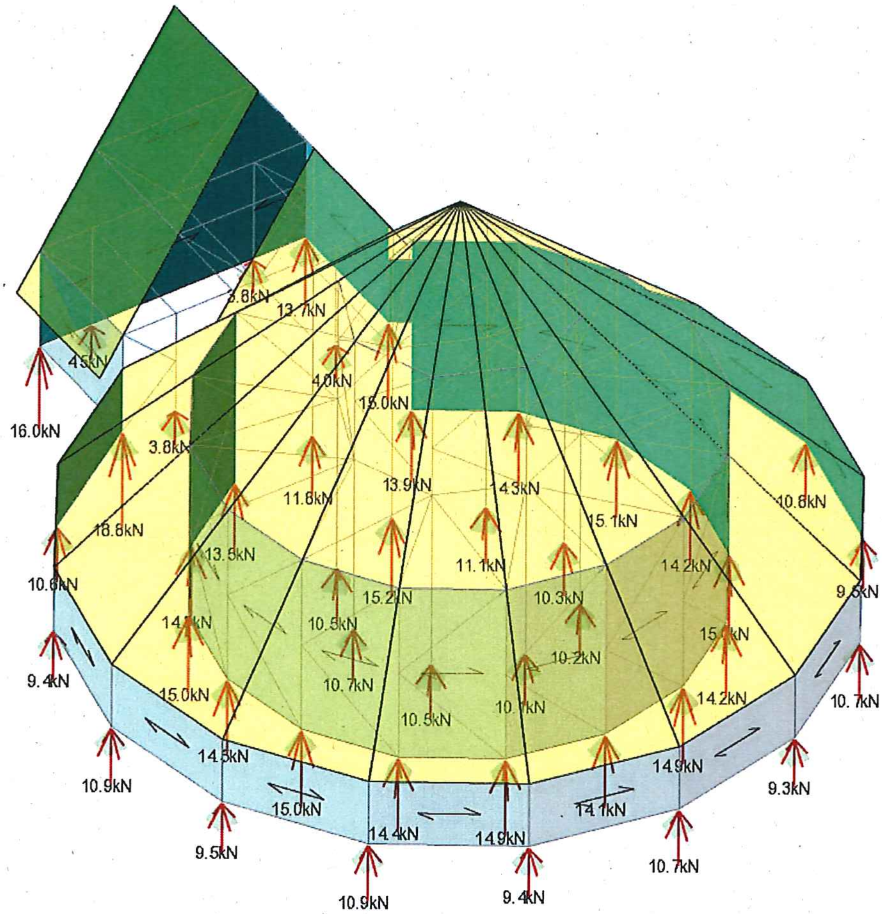
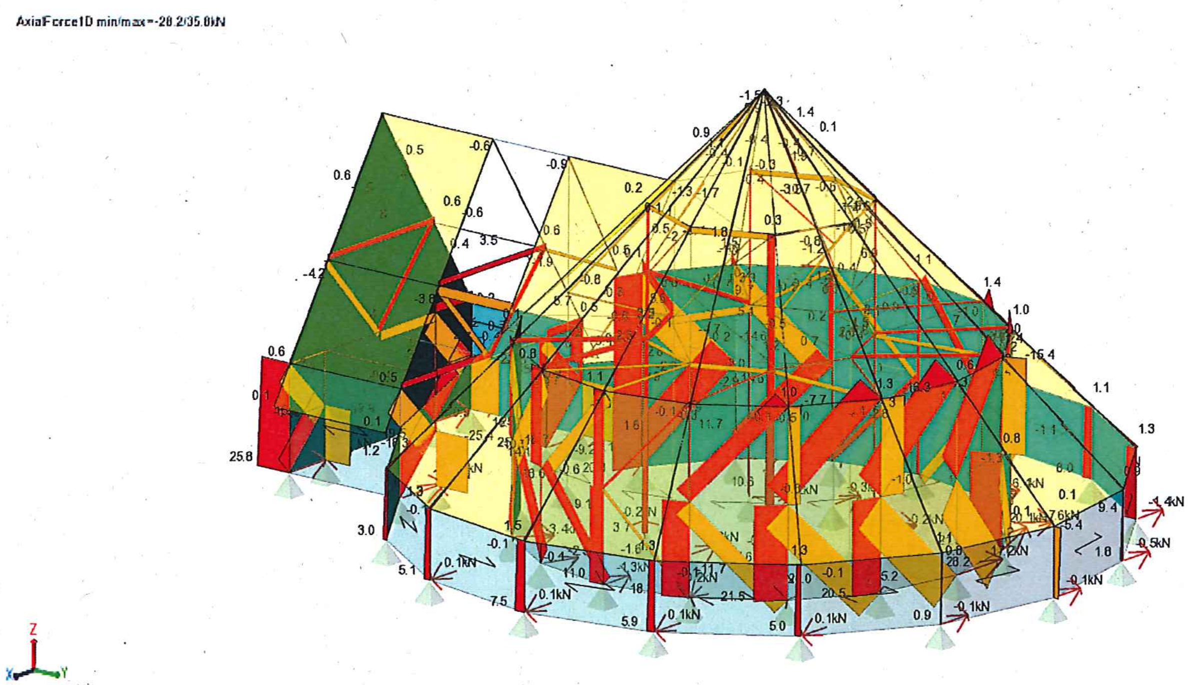
## BASE REACTIONS



# ANALYSIS RESULTS FOR 'DEAD + WIND' COMBINATION

Due to symmetry of roundhouse wind loads in any direction produce consistent results in terms of member loads and deflections.

## MEMBER AXIAL FORCES



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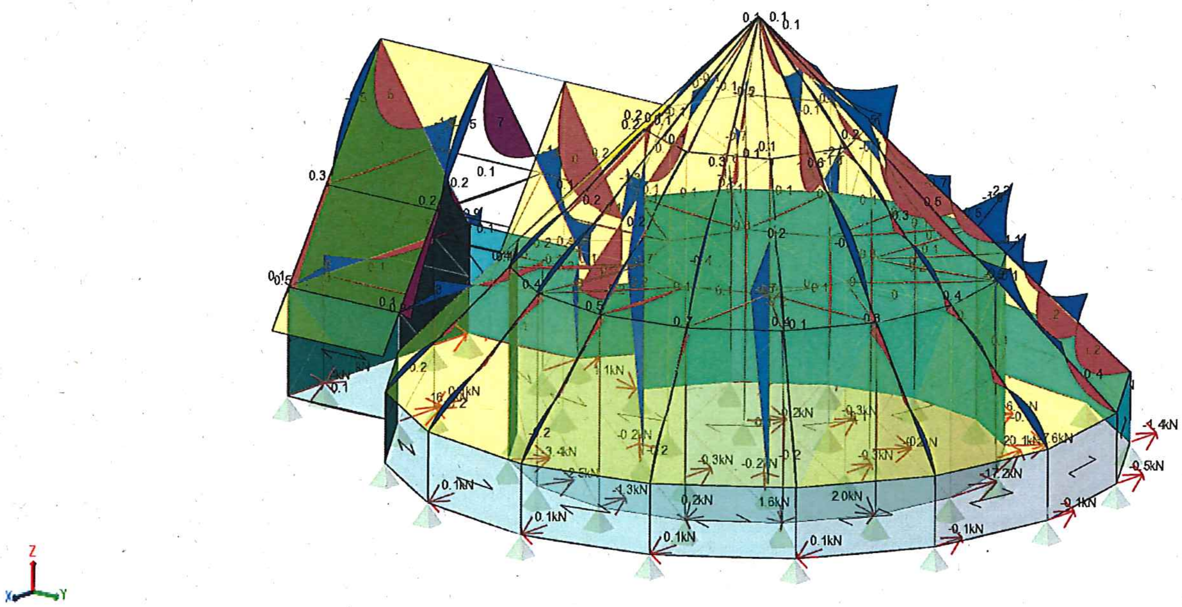
## MEMBER MOMENT FORCES

MomentMajor1D min/max=-2.2/2.0kNm



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|---|---|---|---|---|---|---|
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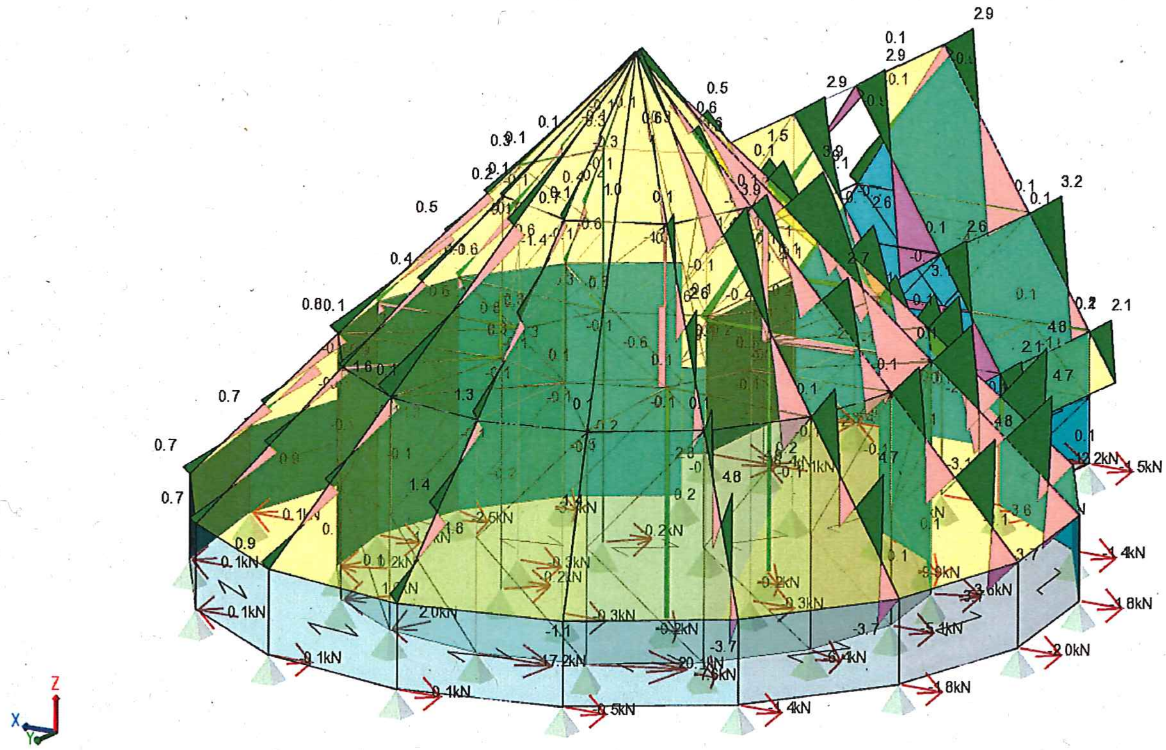
## MEMBER SHEAR FORCES

ShearMajor1D min/max=-4.9/4.8kN



| Finite engineering<br>Consulting Civil and Structural Engineers<br>17 Marchfield Place, Dumfries, DG1 1GQ<br>Director: A Morrow CEng MICE MIStructE MCIHT<br>[redacted] | Whithorn Roundhouse | | | | Job Ref. 160209 | |
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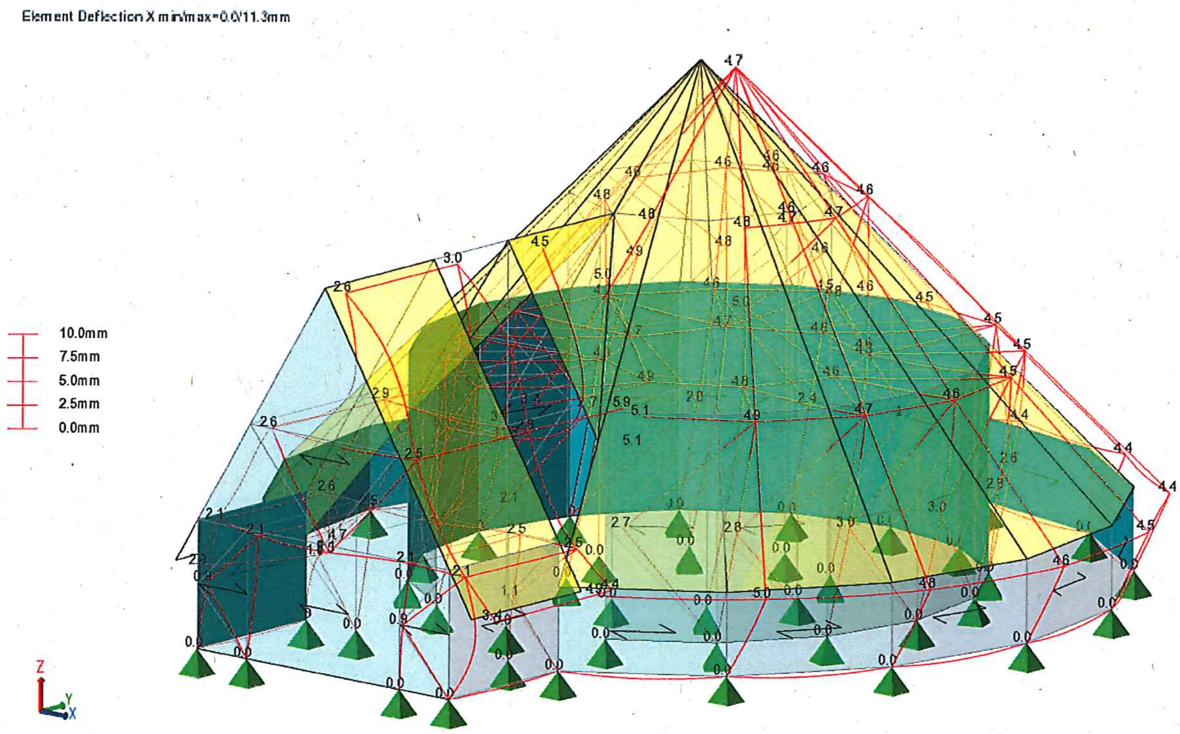
| | Status | Building Warrant Application | | | Sheet no./rev.<br>19 | |
| | Calc. by<br>A.M | Date<br>10 April 2016 | Chk'd by | Date | App'd by | |

## STRUCTURAL DEFLECTIONS UNDER WIND LOADING



| Finite engineering<br>Consulting Civil and Structural Engineers<br>17 Marchfield Place, Dumfries, DG1 1GQ<br>Director: A Morrow CEng MICE MIStructE MCIHT<br>[redacted] | Whithorn Roundhouse | | | | Job Ref. 160209 | |
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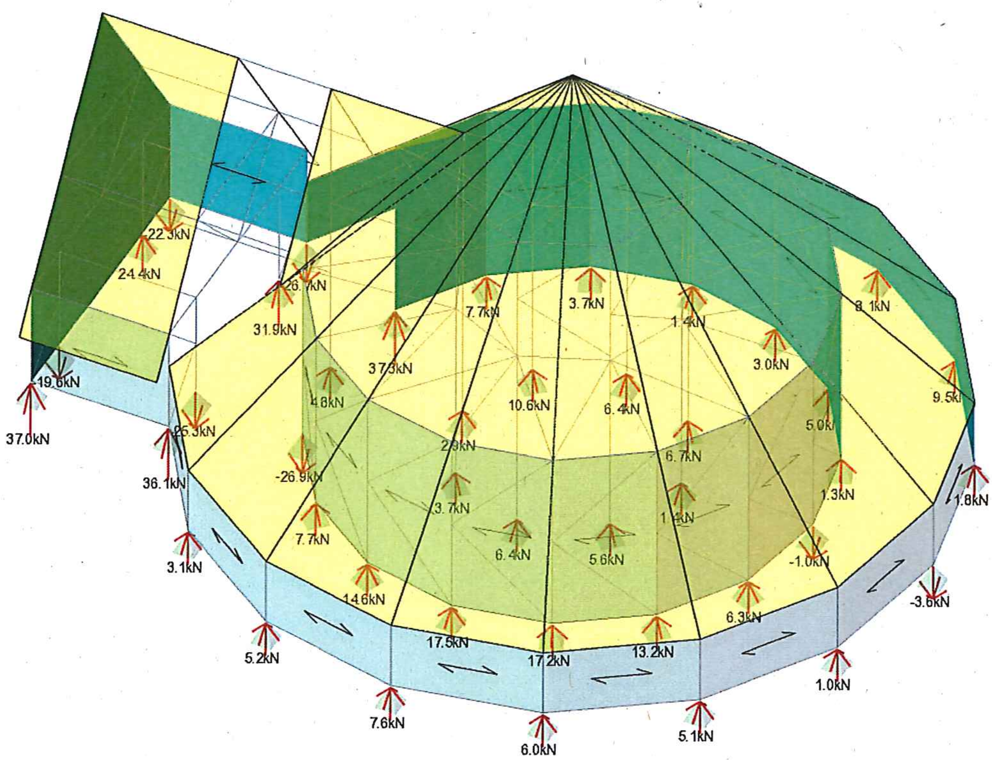
## BASE REACTIONS



## CHECK CENTRAL COLUMNS

Max ULS axial force 16 kN say
Moment - 1 kNm
Shear – 0.2 kN

Distance between restraints – 4m say (conservative)

## TIMBER MEMBER DESIGN (EN1995)

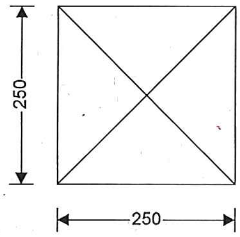
### TIMBER MEMBER DESIGN TO EN1995-1-1:2004

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

Tedds calculation version 1.5.11

**Analysis results**

| | |
|---|---|
| Design moment in major axis | $M_y$ = **1.000** kNm |
| Design shear | F = **0.500** kN |
| Design axial compression | P = **16.000** kN |




**Timber section details**

| | |
|---|---|
| Breadth of timber sections | b = **250** mm |
| Depth of timber sections | h = **250** mm |
| Number of timber sections in member | N = **1** |
| Overall breadth of timber member | $b_b$ = N × b = **250** mm |
| Timber strength class - EN 338:2009 Table 1 | **C14** |

**Member details**

| | |
|---|---|
| Load duration - cl.2.3.1.2 | **Medium-term** |
| Service class of timber - cl.2.3.1.3 | **2** |
| Unbraced length in major (y-y) axis | $L_y$ = **4000** mm |
| Effective length factor in major (y-y) axis | $K_y$ = **1** |
| Effective length in major (y-y) axis | $L_{ey} = L_y \times K_y$ = **4000** mm |
| Unbraced length in minor (z-z) axis | $L_z$ = **4000** mm |
| Effective length factor in minor (z-z) axis | $K_z$ = **1** |
| Effective length in minor (z-z) axis | $L_{ez} = L_z \times K_z$ = **4000** mm |

*In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.*

**Section properties**

Cross sectional area of member $A = N \times b \times h = \mathbf{62500}$ mm$^2$

Section modulus $W_y = N \times b \times h^2 / 6 = \mathbf{2604167}$ mm$^3$

$W_z = h \times (N \times b)^2 / 6 = \mathbf{2604167}$ mm$^3$

Second moment of area $I_y = N \times b \times h^3 / 12 = \mathbf{325520833}$ mm$^4$

$I_z = h \times (N \times b)^3 / 12 = \mathbf{325520833}$ mm$^4$

Radius of gyration $r_y = \sqrt{(I_y / A)} = \mathbf{72.2}$ mm

$r_z = \sqrt{(I_z / A)} = \mathbf{72.2}$ mm

**Partial factor for material properties and resistances**

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

**Modification factors**

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} = \mathbf{0.800}$

Deformation factor for service classes - Table 3.2 $k_{def} = \mathbf{0.800}$

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

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

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = \mathbf{0.700}$

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

Load configuration factor - exp.6.4 $k_{c.90} = \mathbf{1.000}$

System strength factor - cl.6.6 $k_{sys} = \mathbf{1.100}$

Effective length - Table 6.1 $L_{ef} = 1.0 \times L_s = \mathbf{4000}$ mm

Critical bending stress - exp.6.32 $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef}) = \mathbf{229.125}$ N/mm$^2$

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]} = \mathbf{0.247}$

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

**Bending - cl 6.1.6**

Design bending stress $\sigma_{m.d} = M_y / W_y = \mathbf{0.384}$ N/mm$^2$

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M = \mathbf{9.477}$ N/mm$^2$

$\sigma_{m.d} / f_{m.d} = \mathbf{0.041}$

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

**Compression parallel to the grain - cl.6.1.4**

Design compressive stress $\sigma_{c.0.d} = P / A = \mathbf{0.256}$ N/mm$^2$

Design compressive strength $f_{c.0.d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M = \mathbf{10.831}$ N/mm$^2$

$\sigma_{c.0.d} / f_{c.0.d} = \mathbf{0.024}$

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

**Columns subjected to either compression or combined compression and bending - cl.6.3.2**

Slenderness ratio in major (y-y) axis $\lambda_y = L_{ey} / r_y = \mathbf{55.426}$

Relative slenderness ratio - eq.6.21 $\lambda_{rel.y} = \lambda_y \times \sqrt{(f_{c.0.k} / E_{0.05})} / \pi = \mathbf{1.029}$

Slenderness ratio in minor (z-z) axis $\lambda_z = L_{ez} / r_z = \mathbf{55.426}$

Relative slenderness ratio - eq.6.22 $\lambda_{rel.z} = \lambda_z \times \sqrt{(f_{c.0.k} / E_{0.05})} / \pi = \mathbf{1.029}$

Instability factors - eq.6.25 to eq.6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}^2) = \mathbf{1.103}$

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^2) = \mathbf{1.103}$

$k_{c.y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel.y}^2)}) = \mathbf{0.667}$

$k_{c.z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel.z}^2)}) = \mathbf{0.667}$

Column stability checks - eq.6.23 & eq.6.24 $\sigma_{c.0.d} / (k_{c.y} \times f_{c.0.d}) + \sigma_{m.d} / f_{m.d} = \mathbf{0.076}$

$\sigma_{c.0.d} / (k_{c.z} \times f_{c.0.d}) + k_m \times \sigma_{m.d} / f_{m.d} =$ **0.064**

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

**Combined bending and axial compression - cl.6.2.4**

Combined loading checks - eq.6.19 & 6.20 — $(\sigma_{c.0.d} / f_{c.0.d})^2 + \sigma_{m.d} / f_{m.d} =$ **0.041**

$(\sigma_{c.0.d} / f_{c.0.d})^2 + k_m \times \sigma_{m.d} / f_{m.d} =$ **0.029**

***PASS - Member design meets combined bending and axial compression criteria***

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

Lateral torsional stability check - eq.6.35 — $(\sigma_{m.d} / (k_{crit} \times f_{m.d}))^2 + \sigma_{c.0.d} / (k_{c.z} \times f_{c.0.d}) =$ **0.037**

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

**Shear - cl.6.1.7**

Applied shear stress — $\tau_d = 3 \times F / (2 \times k_{cr} \times A) =$ **0.018** N/mm²

Permissible shear stress — $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M =$ **2.031** N/mm²

$\tau_d / f_{v.d} =$ **0.009**

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

## CHECK TIMBER BRACING

From wind combination ULS axial force = 30 kN

Proposed timbers 100mm x 100mm    length 2.4m

## TIMBER MEMBER DESIGN (EN1995)

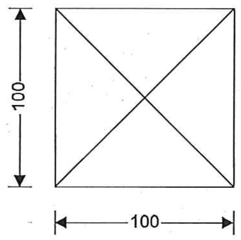
### TIMBER MEMBER DESIGN TO EN1995-1-1:2004

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

Tedds calculation version 1.5.11

**Analysis results**

Design axial compression    P = **30.000** kN




**Timber section details**

Breadth of timber sections    b = **100** mm

Depth of timber sections    h = **100** mm

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

Overall breadth of timber member    $b_b = N \times b =$ **100** mm

Timber strength class - EN 338:2009 Table 1    **C14**

**Member details**

Load duration - cl.2.3.1.2    **Short-term**

Service class of timber - cl.2.3.1.3    **2**

Unbraced length in major (y-y) axis    $L_y =$ **2500** mm

Effective length factor in major (y-y) axis    $K_y =$ **1**

Effective length in major (y-y) axis    $L_{ey} = L_y \times K_y =$ **2500** mm

Unbraced length in minor (z-z) axis    $L_z =$ **2500** mm

Effective length factor in minor (z-z) axis    $K_z =$ **1**

Effective length in minor (z-z) axis    $L_{ez} = L_z \times K_z =$ **2500** mm

*In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.*

**Section properties**

Cross sectional area of member    $A = N \times b \times h =$ **10000** mm$^2$

Section modulus    $W_y = N \times b \times h^2 / 6 =$ **166667** mm$^3$

$W_z = h \times (N \times b)^2 / 6 =$ **166667** mm$^3$

Second moment of area    $I_y = N \times b \times h^3 / 12 =$ **8333333** mm$^4$

$I_z = h \times (N \times b)^3 / 12 =$ **8333333** mm$^4$

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

$r_z = \sqrt{(I_z / A)}$ = **28.9** mm

**Partial factor for material properties and resistances**

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

**Modification factors**

Modification factor for load duration and moisture content - Table 3.1

$k_{mod}$ = **0.900**

Deformation factor for service classes - Table 3.2 $k_{def}$ = **0.800**

Depth factor for bending - exp.3.1 $k_{h.m} = \min((150 \text{ mm} / h)^{0.2}, 1.3)$ = **1.084**

Depth factor for tension - exp.3.1 $k_{h.t} = \min((150 \text{ mm} / \max(b, h))^{0.2}, 1.3)$ = **1.084**

Bending stress re-distribution factor - cl.6.1.6(2) $k_m$ = **0.700**

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

Load configuration factor - exp.6.4 $k_{c.90}$ = **1.000**

System strength factor - cl.6.6 $k_{sys}$ = **1.100**

Effective length - Table 6.1 $L_{ef} = 1.0 \times L_s$ = **2500** mm

Critical bending stress - exp.6.32 $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef})$ = **146.640** N/mm$^2$

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]}$ = **0.309**

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

**Compression parallel to the grain - cl.6.1.4**

Design compressive stress $\sigma_{c.0.d} = P / A$ = **3.000** N/mm$^2$

Design compressive strength $f_{c.0.d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **12.185** N/mm$^2$

$\sigma_{c.0.d} / f_{c.0.d}$ = **0.246**

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

**Columns subjected to either compression or combined compression and bending - cl.6.3.2**

Slenderness ratio in major (y-y) axis $\lambda_y = L_{ey} / r_y$ = **86.603**

Relative slenderness ratio - eq.6.21 $\lambda_{rel.y} = \lambda_y \times \sqrt{(f_{c.0.k} / E_{0.05})} / \pi$ = **1.608**

Slenderness ratio in minor (z-z) axis $\lambda_z = L_{ez} / r_z$ = **86.603**

Relative slenderness ratio - eq.6.22 $\lambda_{rel.z} = \lambda_z \times \sqrt{(f_{c.0.k} / E_{0.05})} / \pi$ = **1.608**

Instability factors - eq.6.25 to eq.6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}^2)$ = **1.924**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^2)$ = **1.924**

$k_{c.y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel.y}^2)})$ = **0.335**

$k_{c.z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel.z}^2)})$ = **0.335**

Column stability checks - eq.6.23 & eq.6.24 $\sigma_{c.0.d} / (k_{c.y} \times f_{c.0.d})$ = **0.734**

$\sigma_{c.0.d} / (k_{c.z} \times f_{c.0.d})$ = **0.734**

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

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# SECONDARY CROSS TIMBERS BETWEEN RAFTERS WHERE SPAN > 1.5M

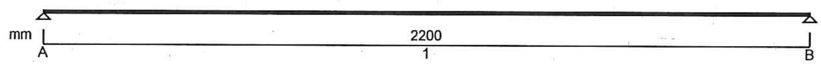
# TIMBER JOIST DESIGN (BS5268)

## TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.03

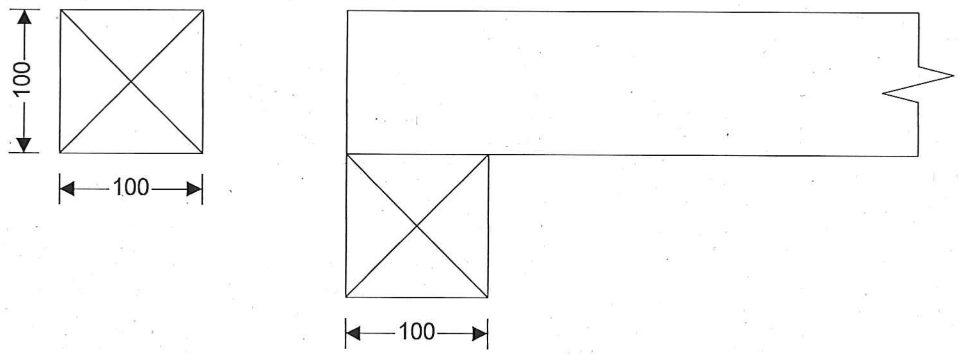
**Joist details**

| | |
|---|---|
| Joist breadth | $b = \mathbf{100}$ mm |
| Joist depth | $h = \mathbf{100}$ mm |
| Joist spacing | $s = \mathbf{300}$ mm |
| Timber strength class | **C14** |
| Service class of timber | **1** |




**Span details**

| | |
|---|---|
| Number of spans | $N_{span} = \mathbf{1}$ |
| Length of bearing | $L_b = \mathbf{100}$ mm |
| Effective length of span | $L_{s1} = \mathbf{2200}$ mm |




**Section properties**

| | |
|---|---|
| Second moment of area | $I = b \times h^3 / 12 = \mathbf{8333333}$ mm$^4$ |
| Section modulus | $Z = b \times h^2 / 6 = \mathbf{166667}$ mm$^3$ |

**Loading details**

| | |
|---|---|
| Joist self weight | $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.03}$ kN/m |
| Dead load | $F_{d\_udl} = \mathbf{0.70}$ kN/m$^2$ |
| Imposed UDL(Medium term) | $F_{i\_udl} = \mathbf{0.60}$ kN/m$^2$ |
| Imposed point load (Short term) | $F_{i\_pt} = \mathbf{0.90}$ kN |

**Modification factors**

| | |
|---|---|
| Service class for bending parallel to grain | $K_{2m} = \mathbf{1.00}$ |

Service class for compression $K_{2c}$ = **1.00**

Service class for shear parallel to grain $K_{2s}$ = **1.00**

Service class for modulus of elasticity $K_{2e}$ = **1.00**

Section depth factor $K_7$ = **1.13**

Load sharing factor $K_8$ = **1.10**

**<u>Consider medium term loads</u>**

Load duration factor $K_3$ = **1.25**

Maximum bending moment M = **0.253** kNm

Maximum shear force V = **0.460** kN

Maximum support reaction R = **0.460** kN

Maximum deflection $\delta$ = **2.324** mm

**Check bending stress**

Bending stress $\sigma_m$ = **4.100** N/mm$^2$

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **6.362** N/mm$^2$

Applied bending stress $\sigma_{m\_max} = M / Z$ = **1.519** N/mm$^2$

***PASS - Applied bending stress within permissible limits***

**Check shear stress**

Shear stress $\tau$ = **0.600** N/mm$^2$

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.825** N/mm$^2$

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.069** N/mm$^2$

***PASS - Applied shear stress within permissible limits***

**Check bearing stress**

Compression perpendicular to grain (no wane) $\sigma_{cp1}$ = **2.100** N/mm$^2$

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.888** N/mm$^2$

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.046** N/mm$^2$

***PASS - Applied bearing stress within permissible limits***

**Check deflection**

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ \text{mm})$ = **6.600** mm

Bending deflection (based on $E_{mean}$) $\delta_{bending}$ = **2.252** mm

Shear deflection $\delta_{shear}$ = **0.071** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **2.324** mm

***PASS - Actual deflection within permissible limits***

**<u>Consider short term loads</u>**

Load duration factor $K_3$ = **1.50**

Maximum bending moment M = **0.639** kNm

Maximum shear force V = **1.162** kN

Maximum support reaction R = **1.162** kN

Maximum deflection $\delta$ = **4.987** mm

**Check bending stress**

Bending stress $\sigma_m$ = **4.100** N/mm$^2$

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.634** N/mm$^2$

Applied bending stress $\sigma_{m\_max} = M / Z$ = **3.836** N/mm$^2$

***PASS - Applied bending stress within permissible limits***

**Check shear stress**

Shear stress $\tau = \mathbf{0.600}$ N/mm$^2$

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = \mathbf{0.990}$ N/mm$^2$

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = \mathbf{0.174}$ N/mm$^2$

***PASS - Applied shear stress within permissible limits***

**Check bearing stress**

Compression perpendicular to grain (no wane) $\sigma_{cp1} = \mathbf{2.100}$ N/mm$^2$

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{3.465}$ N/mm$^2$

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b) = \mathbf{0.116}$ N/mm$^2$

***PASS - Applied bearing stress within permissible limits***

**Check deflection**

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm}) = \mathbf{6.600}$ mm

Bending deflection (based on $E_{mean}$) $\delta_{bending} = \mathbf{4.807}$ mm

Shear deflection $\delta_{shear} = \mathbf{0.180}$ mm

Total deflection $\delta = \delta_{bending} + \delta_{shear} = \mathbf{4.987}$ mm

***PASS - Actual deflection within permissible limits***

| | | | | | |
|---|---|---|---|---|---|
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# SECONDARY TIMBERS BETWEEN RAFTERS WHERE SPAN UPTO 1.5M

## TIMBER JOIST DESIGN (BS5268)

### TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.03

**Joist details**

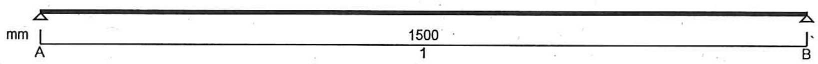
Joist breadth — $b = \mathbf{100}$ mm

Joist depth — $h = \mathbf{100}$ mm

Joist spacing — $s = \mathbf{600}$ mm

Timber strength class — **C14**

Service class of timber — **1**

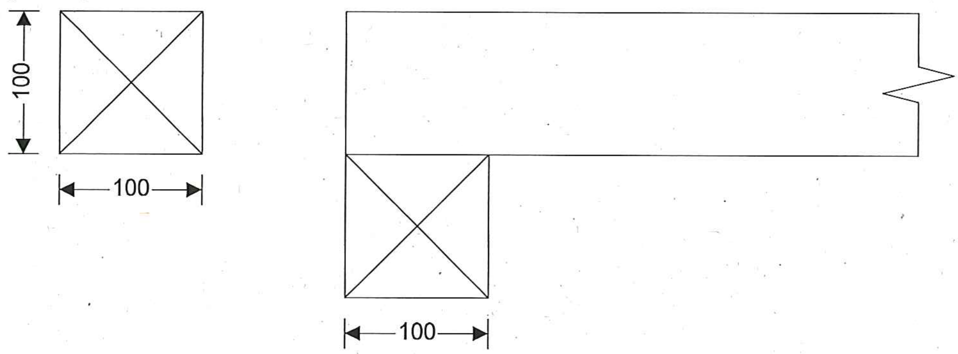



**Span details**

Number of spans — $N_{span} = \mathbf{1}$

Length of bearing — $L_b = \mathbf{100}$ mm

Effective length of span — $L_{s1} = \mathbf{1500}$ mm




**Section properties**

Second moment of area — $I = b \times h^3 / 12 = \mathbf{8333333}$ mm$^4$

Section modulus — $Z = b \times h^2 / 6 = \mathbf{166667}$ mm$^3$

**Loading details**

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

Dead load — $F_{d\_udl} = \mathbf{0.70}$ kN/m$^2$

Imposed UDL(Medium term) — $F_{i\_udl} = \mathbf{0.60}$ kN/m$^2$

Imposed point load (Short term) — $F_{i\_pt} = \mathbf{0.90}$ kN

**Modification factors**

Service class for bending parallel to grain $K_{2m}$ = **1.00**

Service class for compression $K_{2c}$ = **1.00**

Service class for shear parallel to grain $K_{2s}$ = **1.00**

Service class for modulus of elasticity $K_{2e}$ = **1.00**

Section depth factor $K_7$ = **1.13**

Load sharing factor $K_8$ = **1.10**

**Consider medium term loads**

Load duration factor $K_3$ = **1.25**

Maximum bending moment M = **0.227** kNm

Maximum shear force V = **0.606** kN

Maximum support reaction R = **0.606** kN

Maximum deflection $\delta$ = **1.005** mm

**Check bending stress**

Bending stress $\sigma_m$ = **4.100** N/mm$^2$

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **6.362** N/mm$^2$

Applied bending stress $\sigma_{m\_max}$ = M / Z = **1.364** N/mm$^2$

***PASS - Applied bending stress within permissible limits***

**Check shear stress**

Shear stress $\tau$ = **0.600** N/mm$^2$

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.825** N/mm$^2$

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.091** N/mm$^2$

***PASS - Applied shear stress within permissible limits***

**Check bearing stress**

Compression perpendicular to grain (no wane) $\sigma_{cp1}$ = **2.100** N/mm$^2$

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.888** N/mm$^2$

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.061** N/mm$^2$

***PASS - Applied bearing stress within permissible limits***

**Check deflection**

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **4.500** mm

Bending deflection (based on $E_{mean}$) $\delta_{bending}$ = **0.940** mm

Shear deflection $\delta_{shear}$ = **0.064** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **1.005** mm

***PASS - Actual deflection within permissible limits***

**Consider short term loads**

Load duration factor $K_3$ = **1.50**

Maximum bending moment M = **0.464** kNm

Maximum shear force V = **1.236** kN

Maximum support reaction R = **1.236** kN

Maximum deflection $\delta$ = **1.769** mm

**Check bending stress**

Bending stress $\sigma_m$ = **4.100** N/mm$^2$

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.634** N/mm$^2$

Applied bending stress $\sigma_{m\_max}$ = M / Z = **2.782** N/mm$^2$

***PASS - Applied bending stress within permissible limits***

**Check shear stress**

Shear stress $\tau = \mathbf{0.600}$ N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = \mathbf{0.990}$ N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = \mathbf{0.185}$ N/mm²

***PASS - Applied shear stress within permissible limits***

**Check bearing stress**

Compression perpendicular to grain (no wane) $\sigma_{cp1} = \mathbf{2.100}$ N/mm²

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{3.465}$ N/mm²

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b) = \mathbf{0.124}$ N/mm²

***PASS - Applied bearing stress within permissible limits***

**Check deflection**

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm}) = \mathbf{4.500}$ mm

Bending deflection (based on $E_{mean}$) $\delta_{bending} = \mathbf{1.638}$ mm

Shear deflection $\delta_{shear} = \mathbf{0.131}$ mm

Total deflection $\delta = \delta_{bending} + \delta_{shear} = \mathbf{1.769}$ mm

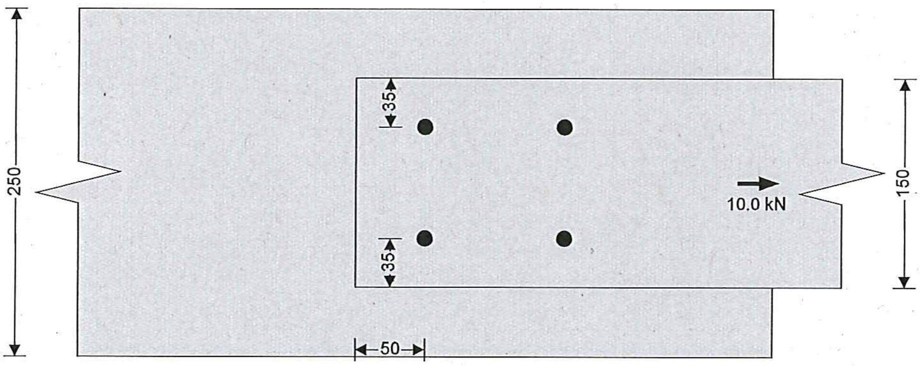
***PASS - Actual deflection within permissible limits***

## PROPOSED COLUMN TO RAFT CONNECTION

## TIMBER CONNECTION DESIGN (BS5268)

### BOLTED TIMBER TO STEEL CONNECTION DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.10




**Main timber member details**

Main timber member thickness $b_m$ = **125** mm

Main timber member depth $h_m$ = **250** mm

Number of timbers in main member $N_m$ = **2**

**Strength class C14 timber (Table 8 BS5268:Pt 2:2002)**

**Connected steel plate details**

Connected steel plate thickness $b_c$ = **10.0** mm

Connected steel plate depth $h_c$ = **150** mm

Number of steel plates connected $N_c$ = **2**

**Connection details**

Angle of connected member $\alpha$ = **0°**

Load applied to connected member F = **10.000** kN

Duration of Loading **Very short term**

Member service class **2**

**Bolting details**

Bolt diameter $\phi_b$ = **12** mm

Number of rows of bolts $N_{rows}$ = **2 (80 mm centres)**

Number of bolts per row $N_{conns}$ = **2 (100 mm centres)**

Total number of bolts $N_{total} = N_{conns} \times N_{rows}$ = **4**

**The bolts are aligned with the main member**

Number of interfaces $N_{int} = (N_m + N_c) - 1$ = **3**

**Check main member**

Effective width of timber $b_t$ = **125** mm

**From BS 5268 : Part 2 : 2002 ..... Table 75**

Basic bolt shear load parallel to loading $F_{basic}$ = **3.690** kN

**Modification factors**

Steel to timber $K_{46}$ = **1.25**

| | |
|---|---|
| Moisture content | $K_{56}$ = **1.00** |
| Bolts in line with grain | $K_{57}$ = **0.97** |

**Capacity of bolted connection**

| | |
|---|---|
| Capacity of connection | $F_m = N_{total} \times F_{basic} \times K_{46} \times K_{56} \times K_{57} \times N_{int}$ = **53.690** kN |

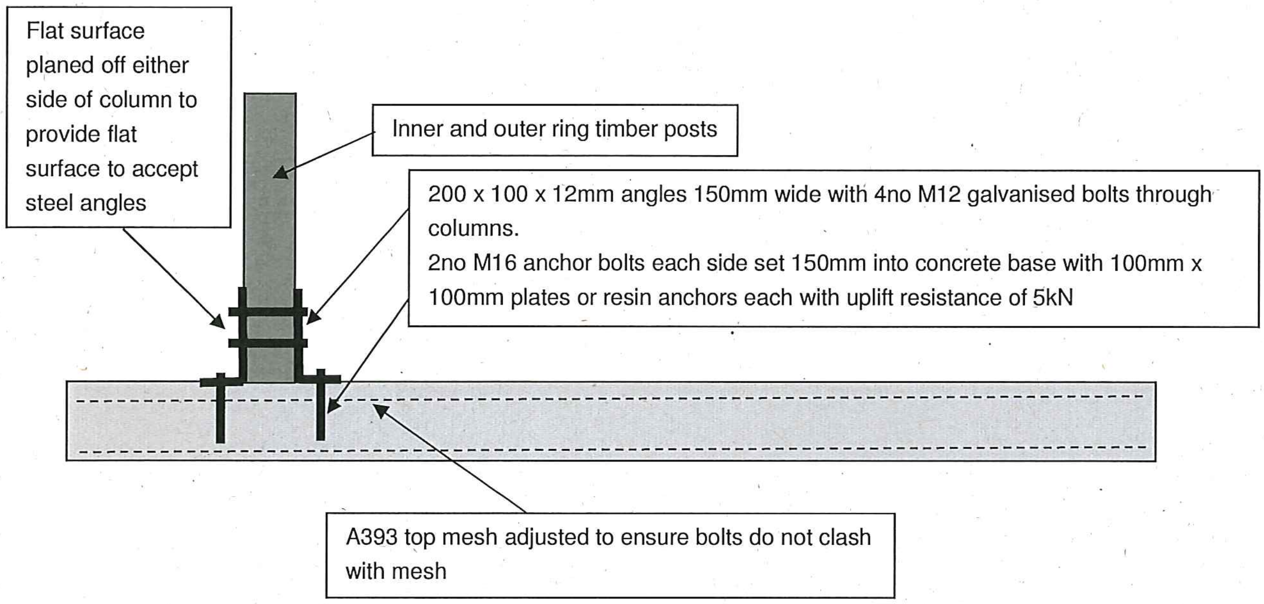
***PASS - Connection capacity exceeds applied load***

**Minimum bolt spacings**

| | |
|---|---|
| Loaded end spacing | $S_{end\_l} = 7 \times \phi_b$ = **84** mm |
| Unloaded end spacing | $S_{end\_u} = 4 \times \phi_b$ = **48** mm |
| Loaded edge spacing | $S_{edge\_l} = 4 \times \phi_b$ = **48** mm |
| Unloaded edge spacing | $S_{edge\_u} = 1.5 \times \phi_b$ = **18** mm |
| Minimum bolt spacing | $S_{bolt} = 4 \times \phi_b$ = **48** mm |

**Washer details**

| | |
|---|---|
| Minimum washer diameter | $\phi_w = 3 \times \phi_b$ = **36** mm |
| Minimum washer thickness | $t_w = 0.25 \times \phi_b$ = **3.0** mm |

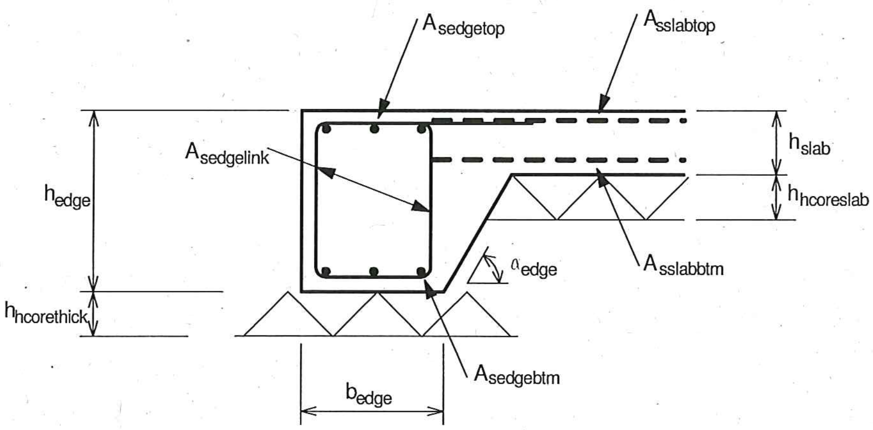


## CONCRETE RAFT DESIGN

# RC RAFT FOUNDATION (BS8110)

### RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.11




### Soil and raft definition

**Soil definition**

| | |
|---|---|
| Allowable bearing pressure | $q_{allow}$ = **25.0** kN/m² |
| Number of types of soil forming sub-soil | **One type only** |
| Soil density | **Firm to loose** |
| Depth of hardcore beneath slab | $h_{hcoreslab}$ = **150** mm (Dispersal allowed for bearing pressure check) |
| Depth of hardcore beneath thickenings | $h_{hcorethick}$ = **150** mm (Dispersal allowed for bearing pressure check) |
| Density of hardcore | $\gamma_{hcore}$ = **20.0** kN/m³ |
| Basic assumed diameter of local depression | $\phi_{depbasic}$ = **2000**mm |
| Diameter under slab modified for hardcore | $\phi_{depslab} = \phi_{depbasic} - h_{hcoreslab}$ = **1850** mm |
| Diameter under thickenings modified for hardcore | $\phi_{depthick} = \phi_{depbasic} - h_{hcorethick}$ = **1850** mm |

**Raft slab definition**

| | |
|---|---|
| Max dimension/max dimension between joints | $l_{max}$ = **15.000** m |
| Slab thickness | $h_{slab}$ = **200** mm |
| Concrete strength | $f_{cu}$ = **30** N/mm² |
| Poissons ratio of concrete | $\nu$ = **0.2** |
| Slab mesh reinforcement strength | $f_{yslab}$ = **500** N/mm² |
| Partial safety factor for steel reinforcement | $\gamma_s$ = **1.15** |

From C&CA document 'Concrete ground floors' Table 5

| | |
|---|---|
| Minimum mesh required in top for shrinkage | **A142** |
| Actual mesh provided in top | **A393 ($A_{sslabtop}$ = 393 mm²/m)** |
| Mesh provided in bottom | **A393 ($A_{sslabbtm}$ = 393 mm²/m)** |
| Top mesh bar diameter | $\phi_{slabtop}$ = **10** mm |
| Bottom mesh bar diameter | $\phi_{slabbtm}$ = **10** mm |
| Cover to top reinforcement | $c_{top}$ = **40** mm |
| Cover to bottom reinforcement | $c_{btm}$ = **40** mm |

| | | | | | | |
|---|---|---|---|---|---|---|
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Average effective depth of top reinforcement $d_{tslabav} = h_{slab} - c_{top} - \phi_{slabtop} =$ **150** mm

Average effective depth of bottom reinforcement $d_{bslabav} = h_{slab} - c_{btm} - \phi_{slabbtm} =$ **150** mm

Overall average effective depth $d_{slabav} = (d_{tslabav} + d_{bslabav})/2 =$ **150** mm

Minimum effective depth of top reinforcement $d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 =$ **145** mm

Minimum effective depth of bottom reinforcement $d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 =$ **145** mm

**Edge beam definition**

Overall depth $h_{edge} =$ **450** mm

Width $b_{edge} =$ **450** mm

Angle of chamfer to horizontal $\alpha_{edge} =$ **60** deg

Strength of main bar reinforcement $f_y =$ **500** N/mm$^2$

Strength of link reinforcement $f_{ys} =$ **500** N/mm$^2$

Reinforcement provided in top **5 H10 bars ($A_{sedgetop}$ = 393 mm$^2$)**

Reinforcement provided in bottom **5 H10 bars ($A_{sedgebtm}$ = 393 mm$^2$)**

Link reinforcement provided **2 H10 legs at 250 ctrs ($A_{sv}/s_v$ = 0.628 mm)**

Bottom cover to links $c_{beam} =$ **40** mm

Effective depth of top reinforcement $d_{edgetop} = h_{edge} - c_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 =$ **385** mm

Effective depth of bottom reinforcement $d_{edgebtm} = h_{edge} - c_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 =$ **395** mm

**<u>Internal slab design checks</u>**

**Basic loading**

Slab self weight $w_{slab} = 24$ kN/m$^3$ $\times h_{slab} =$ **4.8** kN/m$^2$

Hardcore $w_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} =$ **3.0** kN/m$^2$

**Applied loading**

Uniformly distributed dead load $w_{Dudl} =$ **1.0** kN/m$^2$

Uniformly distributed live load $w_{Ludl} =$ **5.0** kN/m$^2$

**Slab load number 1**

Load type **Point load**

Dead load $w_{D1} =$ **8.0** kN

Live load $w_{L1} =$ **20.0** kN

Ultimate load $w_{ult1} = 1.4 \times w_{D1} + 1.6 \times w_{L1} =$ **43.2** kN

Load dimension 1 $b_{11} =$ **250** mm

Load dimension 2 $b_{21} =$ **250** mm

**Internal slab bearing pressure check**

Total uniform load at formation level $w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl} =$ **13.8** kN/m$^2$

**Bearing pressure beneath load number 1**

Net bearing pressure available to resist point load $q_{net} = q_{allow} - w_{udl} =$ **11.2** kN/m$^2$

Net 'ultimate' bearing pressure available $q_{netult} = q_{net} \times w_{ult1}/(w_{D1} + w_{L1}) =$ **17.3** kN/m$^2$

Loaded area required at formation $A_{req1} = w_{ult1}/q_{netult} =$ **2.500** m$^2$

Length of cantilever projection at formation $p_1 = \max(0\text{ m}, [-(b_{11}+b_{21}) + \sqrt{((b_{11}+b_{21})^2 - 4\times(b_{11}\times b_{21} - A_{req1}))}]/4)$

$p_1 =$ **0.666** m

Length of cantilever projection at u/side slab $p_{eff1} = \max(0\text{ m}, p_1 - h_{hcoreslab} \times \tan(30)) =$ **0.579** m

Effective loaded area at u/side slab $A_{eff1} = (b_{11} + 2 \times p_{eff1}) \times (b_{21} + 2 \times p_{eff1}) =$ **1.982** m$^2$

Effective net ult bearing pressure at u/side slab $q_{netulteff} = q_{netult} \times A_{req1}/A_{eff1} =$ **21.8** kN/m$^2$

Cantilever bending moment $M_{cant1} = q_{netulteff} \times p_{eff1}^2/2 =$ **3.7** kNm/m

**Reinforcement required in bottom**

Maximum cantilever moment $M_{cantmax} =$ **3.7** kNm/m

K factor $K_{slabbp} = M_{cantmax}/(f_{cu} \times d_{bslabmin}^2) = \mathbf{0.006}$

Lever arm $z_{slabbp} = d_{bslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbp}/0.9)}) = \mathbf{137.8}$ mm

Area of steel required $A_{sslabbpreq} = M_{cantmax}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbp}) = \mathbf{61}$ mm²/m

***PASS - $A_{sslabbpreq} <= A_{sslabbtm}$ - Area of reinforcement provided to distribute the load is adequate***

***The allowable bearing pressure will not be exceeded***

**Internal slab bending and shear check**

**Applied bending moments**

Span of slab $l_{slab} = \phi_{depslab} + d_{tslabav} = \mathbf{2000}$ mm

Ultimate self weight udl $w_{swult} = 1.4 \times w_{slab} = \mathbf{6.7}$ kN/m²

Self weight moment at centre $M_{csw} = w_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{0.5}$ kNm/m

Self weight moment at edge $M_{esw} = w_{swult} \times l_{slab}^2 / 32 = \mathbf{0.8}$ kNm/m

Self weight shear force at edge $V_{sw} = w_{swult} \times l_{slab} / 4 = \mathbf{3.4}$ kN/m

**Moments due to applied uniformly distributed loads**

Ultimate applied udl $w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl} = \mathbf{9.4}$ kN/m²

Moment at centre $M_{cudl} = w_{udlult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{0.7}$ kNm/m

Moment at edge $M_{eudl} = w_{udlult} \times l_{slab}^2 / 32 = \mathbf{1.2}$ kNm/m

Shear force at edge $V_{udl} = w_{udlult} \times l_{slab} / 4 = \mathbf{4.7}$ kN/m

**Moment due to load number 1**

Moment at centre $M_{c1} = w_{ult1}/(4\times\pi) \times (1+\nu) \times \ln(l_{slab}/\min(b_{11}, b_{21})) = \mathbf{8.6}$ kNm/m

Moment at edge $M_{e1} = w_{ult1}/(4\times\pi) = \mathbf{3.4}$ kNm/m

Minimum dispersal width for shear $b_{v1} = \min(b_{11} + 2\times b_{21}, b_{21} + 2\times b_{11}) = \mathbf{750.0}$ mm

Approximate shear force $V_1 = w_{ult1} / b_{v1} = \mathbf{57.6}$ kN/m

**Resultant moments and shears**

Total moment at edge $M_{\Sigma e} = \mathbf{5.5}$ kNm/m

Total moment at centre $M_{\Sigma c} = \mathbf{9.8}$ kNm/m

Total shear force $V_{\Sigma} = \mathbf{65.7}$ kN/m

**Reinforcement required in top**

K factor $K_{slabtop} = M_{\Sigma e}/(f_{cu} \times d_{tslabav}^2) = \mathbf{0.008}$

Lever arm $z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = \mathbf{142.5}$ mm

Area of steel required for bending $A_{sslabtopbend} = M_{\Sigma e}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtop}) = \mathbf{88}$ mm²/m

Minimum area of steel required $A_{sslabmin} = 0.0013 \times h_{slab} = \mathbf{260}$ mm²/m

Area of steel required $A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = \mathbf{260}$ mm²/m

***PASS - $A_{sslabtopreq} <= A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate***

**Reinforcement required in bottom**

K factor $K_{slabbtm} = M_{\Sigma c}/(f_{cu} \times d_{bslabav}^2) = \mathbf{0.014}$

Lever arm $z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = \mathbf{142.5}$ mm

Area of steel required for bending $A_{sslabbtmbend} = M_{\Sigma c}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbtm}) = \mathbf{158}$ mm²/m

Area of steel required $A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = \mathbf{260}$ mm²/m

***PASS - $A_{sslabbtmreq} <= A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate***

**Shear check**

Applied shear stress $v = V_{\Sigma}/d_{tslabmin} = \mathbf{0.453}$ N/mm²

Tension steel ratio $\rho = 100 \times A_{sslabtop}/d_{tslabmin} = \mathbf{0.271}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_c = \mathbf{0.560}$ N/mm²

***PASS - $v <= v_c$ - Shear capacity of the slab is adequate***

**Internal slab deflection check**

Basic allowable span to depth ratio — $Ratio_{basic}$ = **26.0**

Moment factor — $M_{factor} = M_{\Sigma c}/d_{bslabav}^2$ = **0.435** N/mm²

Steel service stress — $f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend}/A_{sslabbtm}$ = **133.987** N/mm²

Modification factor — $MF_{slab} = \min(2.0, 0.55 + [(477\text{N/mm}^2 - f_s)/(120 \times (0.9\text{N/mm}^2 + M_{factor}))])$

$MF_{slab}$ = **2.000**

Modified allowable span to depth ratio — $Ratio_{allow} = Ratio_{basic} \times MF_{slab}$ = **52.000**

Actual span to depth ratio — $Ratio_{actual} = l_{slab}/ d_{bslabav}$ = **13.333**

***PASS - $Ratio_{actual} <= Ratio_{allow}$ - Slab span to depth ratio is adequate***

**<u>Edge beam design checks</u>**

**Basic loading**

Hardcore — $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick}$ = **3.0** kN/m²

Edge beam

Rectangular beam element — $w_{beam} = 24\text{ kN/m}^3 \times h_{edge} \times b_{edge}$ = **4.9** kN/m

Chamfer element — $w_{chamfer} = 24\text{ kN/m}^3 \times (h_{edge} - h_{slab})^2/(2 \times \tan(\alpha_{edge}))$ = **0.4** kN/m

Slab element — $w_{slabelmt} = 24\text{ kN/m}^3 \times h_{slab} \times (h_{edge} - h_{slab})/\tan(\alpha_{edge})$ = **0.7** kN/m

Edge beam self weight — $w_{edge} = w_{beam} + w_{chamfer} + w_{slabelmt}$ = **6.0** kN/m

**Edge load number 1**

Load type — **Longitudinal line load**

Dead load — $w_{Dedge1}$ = **5.0** kN/m

Live load — $w_{Ledge1}$ = **5.0** kN/m

Ultimate load — $w_{ultedge1} = 1.4 \times w_{Dedge1} + 1.6 \times w_{Ledge1}$ = **15.0** kN/m

Longitudinal line load width — $b_{edge1}$ = **100** mm

Centroid of load from outside face of raft — $x_{edge1}$ = **300** mm

**Edge beam bearing pressure check**

Effective bearing width of edge beam — $b_{bearing} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge})$ = **594** mm

Total uniform load at formation level — $w_{udledge} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick}$ = **19.1** kN/m²

**Centroid of longitudinal and equivalent line loads from outside face of raft**

Load x distance for edge load 1 — $Moment_1 = w_{ultedge1} \times x_{edge1}$ = **4.5** kN

Sum of ultimate longitud'l and equivalent line loads — $\Sigma UDL$ = **15.0** kN/m

Sum of load x distances — $\Sigma Moment$ = **4.5** kN

Centroid of loads — $x_{bar} = \Sigma Moment/\Sigma UDL$ = **300** mm

**Initially assume no moment transferred into slab due to load/reaction eccentricity**

Sum of unfactored longitud'l and eff'tive line loads — $\Sigma UDLsls$ = **10.0** kN/m

Allowable bearing width — $b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30)$ = **773** mm

Bearing pressure due to line/point loads — $q_{linepoint} = \Sigma UDLsls/ b_{allow}$ = **12.9** kN/m²

Total applied bearing pressure — $q_{edge} = q_{linepoint} + w_{udledge}$ = **32.0** kN/m²

***$q_{edge} > q_{allow}$ - The slab is required to resist a moment due to eccentricity***

**Now assume moment due to load/reaction eccentricity is resisted by slab**

Bearing width required — $b_{req} = \Sigma UDLsls/(q_{allow} - w_{udledge})$ = **1687** mm

Effective bearing width at u/s of slab — $b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30)$ = **1514** mm

Load/reaction eccentricity — $e = b_{reqeff}/2 - x_{bar}$ = **457** mm

Ultimate moment to be resisted by slab — $M_{ecc} = \Sigma UDL \times e$ = **6.9** kNm/m

From slab bending check

Moment due to depression under slab (hogging) $M_{\Sigma e}$ = **5.5** kNm/m

Total moment to be resisted by slab top steel $M_{slabtop} = M_{ecc} + M_{\Sigma e}$ = **12.3** kNm/m

K factor $K_{slab} = M_{slabtop}/(f_{cu} \times d_{tslabmin}^2)$ = **0.020**

Lever arm $z_{slab} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)})$ = **138** mm

Area of steel required $A_{sslabreq} = M_{slabtop}/((1.0/\gamma_s) \times 460\ N/mm^2 \times z_{slab})$ = **223** mm²/m

***PASS - $A_{sslabreq}$ <= $A_{sslabtop}$ - Area of reinforcement provided to transfer moment into slab is adequate***

***The allowable bearing pressure under the edge beam will not be exceeded***

**Edge beam bending check**

Divider for moments due to udl's $\beta_{udl}$ = **10.0**

**Applied bending moments**

Span of edge beam $l_{edge} = \phi_{depthick} + d_{edgetop}$ = **2235** mm

Ultimate self weight udl $w_{edgeult} = 1.4 \times w_{edge}$ = **8.4** kN/m

Ultimate slab udl (approx) $w_{edgeslab} = \max(0\ kN/m, 1.4 \times w_{slab} \times ((\phi_{depthick}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$

$w_{edgeslab}$ = **0.7** kN/m

Self weight and slab bending moment $M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2/\beta_{udl}$ = **4.5** kNm

Self weight shear force $V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}/2$ = **10.1** kN

**Moments due to applied uniformly distributed loads**

Ultimate udl (approx) $w_{edgeudl} = w_{udlult} \times \phi_{depthick}/2 \times 3/4$ = **6.5** kN/m

Bending moment $M_{edgeudl} = w_{edgeudl} \times l_{edge}^2/\beta_{udl}$ = **3.3** kNm

Shear force $V_{edgeudl} = w_{edgeudl} \times l_{edge}/2$ = **7.3** kN

**Moment and shear due to load number 1**

Bending moment $M_{edge1} = w_{ultedge1} \times l_{edge}^2/\beta_{udl}$ = **7.5** kNm

Shear force $V_{edge1} = w_{ultedge1} \times l_{edge}/2$ = **16.8** kN

**Resultant moments and shears**

Total moment (hogging and sagging) $M_{\Sigma edge}$ = **15.3** kNm

Maximum shear force $V_{\Sigma edge}$ = **34.2** kN

**Reinforcement required in top**

Width of section in compression zone $b_{edgetop} = b_{edge}$ = **450** mm

Average web width $b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2$ = **580** mm

K factor $K_{edgetop} = M_{\Sigma edge}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.008**

Lever arm $z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)})$ = **366** mm

Area of steel required for bending $A_{sedgetopbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgetop})$ = **96** mm²

Minimum area of steel required $A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **339** mm²

Area of steel required $A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin})$ = **339** mm²

***PASS - $A_{sedgetopreq}$ <= $A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate***

**Reinforcement required in bottom**

Width of section in compression zone $b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) + 0.1 \times l_{edge}$ = **818** mm

K factor $K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2)$ = **0.004**

Lever arm $z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)})$ = **375** mm

Area of steel required for bending $A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgebtm})$ = **94** mm²

Minimum area of steel required $A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **339** mm²

Area of steel required $A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin})$ = **339** mm²

***PASS - $A_{sedgebtmreq}$ <= $A_{sedgebtm}$ - Area of reinforcement provided in bottom of edge beams is adequate***

**Edge beam shear check**

Applied shear stress $v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop}) = \mathbf{0.153}$ N/mm$^2$

Tension steel ratio $\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.176}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_{cedge} = \mathbf{0.380}$ N/mm$^2$

***$v_{edge}$ <= $v_{cedge}$ + 0.4N/mm$^2$ - Therefore minimum links required***

Link area to spacing ratio required $A_{sv}$_upon_$s_{vreqedge} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.534}$ mm

Link area to spacing ratio provided $A_{sv}$_upon_$s_{vprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge}) = \mathbf{0.628}$ mm

***PASS - $A_{sv}$_upon_$s_{vreqedge}$ <= $A_{sv}$_upon_$s_{vprovedge}$ - Shear reinforcement provided in edge beams is adequate***

## Corner design checks

**Basic loading**

**Corner load number 1**

Load type **Line load in x direction**

Dead load $w_{Dcorner1} = \mathbf{5.0}$ kN/m

Live load $w_{Lcorner1} = \mathbf{5.0}$ kN/m

Ultimate load $w_{ultcorner1} = 1.4 \times w_{Dcorner1} + 1.6 \times w_{Lcorner1} = \mathbf{15.0}$ kN/m

Centroid of load from outside face of raft $y_{corner1} = \mathbf{300}$ mm

**Corner bearing pressure check**

Total uniform load at formation level $w_{udlcorner} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick} = \mathbf{19.1}$ kN/m$^2$

Net bearing press avail to resist line/point loads $q_{netcorner} = q_{allow} - w_{udlcorner} = \mathbf{5.9}$ kN/m$^2$

**Total line/point loads**

Total unfactored line load in x direction $w_{\Sigma linex} = \mathbf{10.0}$ kN/m

Total ultimate line load in x direction $w_{\Sigma ultlinex} = \mathbf{15.0}$ kN/m

Total unfactored line load in y direction $w_{\Sigma liney} = \mathbf{0.0}$ kN/m

Total ultimate line load in y direction $w_{\Sigma ultliney} = \mathbf{0.0}$ kN/m

Total unfactored point load $W_{\Sigma point} = \mathbf{0.0}$ kN

Total ultimate point load $W_{\Sigma ultpoint} = \mathbf{0.0}$ kN

Length of side of sq reqd to resist line/point loads $p_{corner} = [w_{\Sigma linex}+w_{\Sigma liney}+\sqrt{((w_{\Sigma linex}+w_{\Sigma liney})^2+4 \times q_{netcorner} \times W_{\Sigma point})}]/(2 \times q_{netcorner})$

$p_{corner} = \mathbf{1687}$ mm

**Bending moment about x-axis due to load/reaction eccentricity**

Moment due to load 1 (x line) $M_{x1} = \max(0 \text{ kNm}, w_{ultcorner1} \times p_{corner} \times (p_{corner}/2 - y_{corner1})) = \mathbf{13.7}$ kNm

Total moment about x axis $M_{\Sigma x} = \mathbf{13.7}$ kNm

**Bending moment about y-axis due to load/reaction eccentricity**

Total moment about y axis $M_{\Sigma y} = \mathbf{0.0}$ kNm

**Check top reinforcement in edge beams for load/reaction eccentric moment**

Max moment due to load/reaction eccentricity $M_\Sigma = \max(M_{\Sigma x}, M_{\Sigma y}) = \mathbf{13.7}$ kNm

Assume all of this moment is resisted by edge beam

From edge beam design checks away from corners

Moment due to edge beam spanning depression $M_{\Sigma edge} = \mathbf{15.3}$ kNm

Total moment to be resisted $M_{\Sigma cornerbp} = M_\Sigma + M_{\Sigma edge} = \mathbf{29.0}$ kNm

Width of section in compression zone $b_{edgetop} = b_{edge} = \mathbf{450}$ mm

K factor $K_{cornerbp} = M_{\Sigma cornerbp}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.015}$

Lever arm $z_{cornerbp} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{cornerbp}/0.9)}) = \mathbf{366}$ mm

Total area of top steel required $A_{scornerbp} = M_{\Sigma cornerbp}/((1.0/\gamma_s) \times f_y \times z_{cornerbp}) = \mathbf{182}$ mm$^2$

***PASS - $A_{scornerbp} <= A_{sedgetop}$ - Area of reinforcement provided to resist eccentric moment is adequate***

***The allowable bearing pressure at the corner will not be exceeded***

**Corner beam bending check**

Cantilever span of edge beam — $l_{corner} = \phi_{depthick}/\sqrt{(2)} + d_{edgetop}/2 =$ **1501** mm

**Moment and shear due to self weight**

Ultimate self weight udl — $w_{edgeult} = 1.4 \times w_{edge} =$ **8.4** kN/m

Average ultimate slab udl (approx) — $w_{cornerslab} = \max(0\ kN/m, 1.4 \times w_{slab} \times (\phi_{depthick}/(\sqrt{(2)} \times 2) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$

$w_{cornerslab} =$ **0.4** kN/m

Self weight and slab bending moment — $M_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}^2/2 =$ **9.9** kNm

Self weight and slab shear force — $V_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner} =$ **13.2** kN

**Moment and shear due to udls**

Maximum ultimate udl — $w_{cornerudl} = ((1.4 \times w_{Dudl}) + (1.6 \times w_{Ludl})) \times \phi_{depthick}/\sqrt{(2)} =$ **12.3** kN/m

Bending moment — $M_{cornerudl} = w_{cornerudl} \times l_{corner}^2/6 =$ **4.6** kNm

Shear force — $V_{cornerudl} = w_{cornerudl} \times l_{corner}/2 =$ **9.2** kN

**Moment and shear due to line loads in x direction**

Bending moment — $M_{cornerlinex} = w_{\Sigma ultlinex} \times l_{corner}^2/2 =$ **16.9** kNm

Shear force — $V_{cornerlinex} = w_{\Sigma ultlinex} \times l_{corner} =$ **22.5** kN

**Moment and shear due to line loads in y direction**

Bending moment — $M_{cornerliney} = w_{\Sigma ultliney} \times l_{corner}^2/2 =$ **0.0** kNm

Shear force — $V_{cornerliney} = w_{\Sigma ultliney} \times l_{corner} =$ **0.0** kN

**Total moments and shears due to point loads**

Bending moment about x axis — $M_{cornerpointx} =$ **0.0** kNm

Bending moment about y axis — $M_{cornerpointy} =$ **0.0** kNm

Shear force — $V_{cornerpoint} =$ **0.0** kN

**Resultant moments and shears**

Total moment about x axis — $M_{\Sigma cornerx} = M_{cornersw} + M_{cornerudl} + M_{cornerliney} + M_{cornerpointx} =$ **14.5** kNm

Total shear force about x axis — $V_{\Sigma cornerx} = V_{cornersw} + V_{cornerudl} + V_{cornerliney} + V_{cornerpoint} =$ **22.4** kN

Total moment about y axis — $M_{\Sigma cornery} = M_{cornersw} + M_{cornerudl} + M_{cornerlinex} + M_{cornerpointy} =$ **31.4** kNm

Total shear force about y axis — $V_{\Sigma cornery} = V_{cornersw} + V_{cornerudl} + V_{cornerlinex} + V_{cornerpoint} =$ **44.9** kN

Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears

Design bending moment — $M_{\Sigma corner} = (M_{\Sigma cornerx} + M_{\Sigma cornery})/2 =$ **22.9** kNm

Design shear force — $V_{\Sigma corner} = (V_{\Sigma cornerx} + V_{\Sigma cornery})/2 =$ **33.7** kN

**Reinforcement required in top of edge beam**

K factor — $K_{corner} = M_{\Sigma corner}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) =$ **0.011**

Lever arm — $z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)}) =$ **366** mm

Area of steel required for bending — $A_{scornerbend} = M_{\Sigma corner}/((1.0/\gamma_s) \times f_y \times z_{corner}) =$ **144** mm²

Minimum area of steel required — $A_{scornermin} = A_{sedgetopmin} =$ **339** mm²

Area of steel required — $A_{scorner} = \max(A_{scornerbend}, A_{scornermin}) =$ **339** mm²

***PASS - $A_{scorner} <= A_{sedgetop}$ - Area of reinforcement provided in top of edge beams at corners is adequate***

**Corner beam shear check**

Average web width — $b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2 =$ **580** mm

Applied shear stress — $v_{corner} = V_{\Sigma corner}/(b_w \times d_{edgetop}) =$ **0.151** N/mm²

Tension steel ratio — $\rho_{corner} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) =$ **0.176**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_{ccorner}$ = **0.376** N/mm$^2$

***$v_{corner}$ <= $v_{ccorner}$ + 0.4N/mm$^2$ - Therefore minimum links required***

Link area to spacing ratio required $A_{sv}$_upon_$s_{vreqcorner}$ = 0.4N/mm$^2$ × $b_w$/((1.0/$\gamma_s$) × $f_{ys}$) = **0.534** mm

Link area to spacing ratio provided $A_{sv}$_upon_$s_{vprovedge}$ = $N_{edgelink}$×π×$\phi_{edgelink}{}^2$/(4×$s_{vedge}$) = **0.628** mm

***PASS - $A_{sv}$_upon_$s_{vreqcorner}$ <= $A_{sv}$_upon_$s_{vprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate***

**Corner beam deflection check**

Basic allowable span to depth ratio Ratio$_{basiccorner}$ = **7.0**

Moment factor $M_{factorcorner}$ = $M_{\Sigma corner}$/($b_{edgetop}$ × $d_{edgetop}{}^2$) = **0.344** N/mm$^2$

Steel service stress $f_{scorner}$ = 2/3 × $f_y$ × $A_{scornerbend}$/$A_{sedgetop}$ = **122.491** N/mm$^2$

Modification factor $MF_{corner}$=min(2.0,0.55+[(477N/mm$^2$-$f_{scorner}$)/(120×(0.9N/mm$^2$+$M_{factorcorner}$))])

$MF_{corner}$ = **2.000**

Modified allowable span to depth ratio Ratio$_{allowcorner}$ = Ratio$_{basiccorner}$ × $MF_{corner}$ = **14.000**

Actual span to depth ratio Ratio$_{actualcorner}$ = $l_{corner}$/ $d_{edgetop}$ = **3.898**

***PASS - Ratio$_{actualcorner}$ <= Ratio$_{allowcorner}$ - Edge beam span to depth ratio is adequate***

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## DESIGN SUMMARY

The proposed timber sections have been overdesigned using a low grade timber to account for uncertainties in the timber used and the likely use of traditional timber connections that would require cut-outs, thereby reducing the effective section at these locations.

The main structural timber sizes (C14 strength assumed) are:

| | | |
|---|---|---|
| Rafters: | 250mm x 250mm timbers | (Min 350mm diameter) |
| Inner ring columns: | 250mm x 250mm timbers | (Min 350mm diameter) |
| Outer ring columns: | 200mm x 200mm timbers | (Min 250mm diameter) |
| Outer wall rafter supports: | 100mm x 100mm timbers | (Min 250mm diameter) |
| Bracing to walls | 100mm x 100mm timbers | (Min 140mm diameter) |

Recommended that timbers spanning between rafters be min 100mm x 100mm (C14 timber assumed) at max 300mm centres from 1.5m span and above. For span 1.5m and under where the rafters are tapering closer together the spacing can be increased to 600mm.

Structure to bear onto 200mm deep concrete raft foundation of RC30 concrete with A393 mesh top and bottom with boot to perimeter as detailed on page 34 above.

Timber posts of outer and inner timber columns to be fixed down onto slab as per detail recommended on page 33. It is recommended that for external walls 100mm x 100mm C14 posts support the end of the rafters. These be taken down to 150mm x 47mm timber base plate around perimeter. Base plate fixed down onto concrete raft via mechanical fixings to resist 3.5 kN/m shear load.

As alternative to diagonal bracing members it is assessed that the use of sheathed timber frame shear walls connected to the columns and fixed down to the concrete raft would also provide good lateral stability. In terms of the structures stability it is important that the diagonal braced timbers are effectively connected to the columns and so close cooperation on site with the engineer is recommended to agree on a suitable connection

It is understood that at site construction the means of connecting timbers may be developed. It is therefore important that the engineer also has close involvement at this stage to agree and sign off on the proposed connections

Method of Construction

It is assessed that the inner and outer loadbearing timber rings will be constructed initially and fixed down onto the concrete raft. These will require being temporarily supported until the rings are fully braced and self-supporting.

On completion of the rings the timber rafters can be installed, bearing onto each ring. The rafters may require temporarily supported until the outer wall is constructed. If adequately fixed onto the rings however at that point they would cantilever safely at each end until supported. The outer wall would then be constructed prior to the secondary roof structure being built over the rafters