RUAMOKO solutions
CONSULTING STRUCTURAL ENGINEERS

EDMONDS ROTUNDA REBUILD CHRISTCHURCH

STRUCTURAL CALCULATIONS

Project No: 604 | Date: June 2014 | Engineer: COG

CALC INDEX

(A) LOADS / PRELIMINARY DESIGN

(B) GRAVITY DESIGN

(C) SEISMIC DESIGN

(D) FOUNDATION DESIGN

Unit 3, 213 Blenheim Road, PO Box 12294, Christchurch, New Zealand
P: +643 366 7714 F: +643 366 7715 E: info@ruamoko.co.nz W: www.ruamoko.co.nz

CALCULATIONS

Project Name: Edmonds Rotunda
Project No: 604 Date: 31/03/2014
Calcs By: COG Page A1

Seismic Loads to NZS 1170.5:2004

Analysis Type =	**General**	
Soil Category =	**D**	
Period =	**0.40**	s (stiff buildings use 0.4s)
Ch(T) =	3.00	(spectral shape factor)
Location =	**Christchurch**	(from north to south)
D =	**n/a**	(km's to faultline - adjust if reqd)
Z =	**0.3**	(as per CERA recommendations for chc post Feb2011 eq)
N(T,D) =	**1.00**	
Importance. Level =	**2 (normal)**	
Design life =	**50**	yrs
ULS annual prob of exc =	**1/500**	
SLS annual prob of exc =	**1/25**	

SLS1 Case - Structure and non-structural components do not require repair

R ULS =	**1.00**	
R SLS =	**0.33**	(as per CERA recommendations for chc post Feb2011 eq)
C(T) =	**0.90**	ULS (elastic site hazard spectrum)
C(T) =	**0.30**	SLS (elastic site hazard spectrum)

Sp ULS =	refer table below
Sp SLS =	**0.70**
Cd(T1) minimum =	**0.04**

Structural Ductility Factor, μ	Sp factor (refer *1 & *2 notes below)	kμ	Horiz. Design Action Coefficients Cd(T1) ULS*1	Cd(T1) ULS*2	Cd(T1) SLS1	
1	1.00	1.00	**0.63**	**0.90**	**0.21**	
1.25	0.93	1.14	**0.55**	**0.73**	**0.18**	
2	0.70	1.57	**0.40**		**0.18**	
3	0.70	2.14	**0.29**		**0.18**	
4	0.70	2.71	**0.23**		**0.18**	
5	0.70	3.29	**0.19**		**0.18**	
6	0.70	3.86	**0.16**		**0.18**	SLS Governs

Cells that are shaded denote that the coefficient is equal or lower than the minimum required value of: 0.035

*1 = use these ULS values for regular new buildings where detailing can handle μ=2.0 (Sp= 0.7 for all μ values)

*2 = use these ULS values for brittle detailing, overturning, sliding, or assessment of older existing buildings (Sp = 1.3-.3*μ as tabulated above)

CALCULATIONS

Project Name:	Edmonds Rotunda		
Project No:	604	Date:	31/03/14
Calcs By:	COG	Page No:	A2

Wind Loads to AS/NZS 1170.2:2011

Importance. Level =	**2 (normal)**	
Design life =	**50**	yrs
ULS annual prob of exc =	**1/500**	
SLS1 annual prob of exc =	**1/25**	
SLS1 Case - Structure and non-structural components do not require repair		
Region =	**A**	
V_R ULS =	**45**	m/s
V_R SLS =	**37**	m/s
Md =	**1.00**	(any direction)
Terrain Category	**3**	
height z	**12.5**	m
M(z,cat) =	**0.86**	
Ms =	**1.00**	
Mh =	**1.00**	
Mlee =	**1.00**	
Site over 500m elev?	**No**	
Mt =	**1.00**	
Vdes = Vsitβ =	**38.7**	**m/s (ULS)**
	31.8	**m/s (SLS)**

Design Wind Pressures

quls =	**0.90**	**kPa**
qsls =	**0.61**	**kPa**

NZS3604:1999 ULS speeds	L	up to	32 m/s
	M	up to	37 m/s
	H	up to	44 m/s
	VH	up to	50 m/s
	EH	up to	55 m/s

hence we have a **H** **Windzone**

CALCULATIONS

Project Name:	Edmonds Rotunda		
Project No:	604	Date:	31/03/14
Calcs By:	COG	Page No:	A3

Snow Loads to AS/NZS 1170.3:2003 (Incorporating B1/VM1 Amendment Dec 2008)

Region =	**N4**	
Elevation ho =	**20**	m above sea level
Class =	**Sub-Alpine**	
Importance. Level =	**2 (normal)**	
Design life =	**50**	yrs
ULS annual prob of exc =	**1/150**	
SLS annual prob of exc =	**1/25**	

SLS1 Case - Structure and non-structural components do not require repair

kp ULS =	**1.25**	
kp SLS =	**0.85**	
Sg (ULS) =	**0.90**	
Sg (SLS) =	**0.90**	
Exposure Reduction =	**Sub-Alpine**	
Ce =	**1**	
Roof Pitch =	**30**	degrees
μ_i =	**0.42**	

Design Roof Snow Loads:

Suls =	**0.38**	**kPa**
Ssls =	**0.26**	**kPa***

*SLS snow loads are modified for SLS:ULS probabilty factor ratio where B1/VM1 snow loads apply

CALCULATIONS

Project Name:	Edmonds Rotunda		
Project No:	604	Date:	31/03/14
Calcs By:	COG	Page No:	A4

Parts & Components Seismic Loads to NZS1170.5:2004

Risk Factor =	**1.0**	refer to table 8.1 (NZS1170.5:2004)
Hn =	**12.5**	m - height from base to uppermost seismic mass of structure
Hi =	**4.0**	m - height of the attachment of the part
Ch(0) =	**1.12**	for site soil class chosen in seismic calculations
C(0) =	**0.34**	site hazard coefficient for T = 0
Chi =	**1.67**	floor height coefficient
Tp =	**0.75**	period of the part (use less than 0.75secs typically)
Ci(Tp) =	**2**	part spectral shape coefficient
CpTp =	**1.120**	part design response coefficient
μp =	**1.00**	refer to table C8.2 (NZS1170.5 Supp 1:2004, pg 75)
Cph =	**1**	part response factor

Horizontal Design Actions:

Fph =	**1.12**	**times the weight of the part at**	**4**	**m**
	1.46	**times the weight of the part at**	**7**	**m**
	1.79	**times the weight of the part at**	**10**	**m**

SKETCHES/CALCS

Project Name: Edmonds Band Rotunda

Project No: 604 Date: 27 02 13.

Engineer: RBW Sheet No: A5 Scale:

Stage One Design - Prelim - to establish budgets.
- 100% NBS
- Rebuild base / Reuse dome.
- No basement.
- Reuse staircase.

LDE Geotechnical Report 14 09 11.

Seismic Mass of Dome = 650 kN c/o Stair calcs

Check that -

Upper cone panels	$2.3 \times 0.38 \times 3.9 \times 24 \times 8$	= 653 kN
" " cornice.	$0.55 \times 0.2 \times 3.9 \times 24 \times 8$	= 82
" " capitals	$2.4 \times 0.2 \times 0.5 \times 24 \times 8$	= 47
" section of columns	$2.4 \times 0.5^2 \times 0.785 \times 24 \times 8$	= 89
Dome - Copper -		= 10
- timber	$116 m^2 \times 0.35$	= 40
Ceiling - timber	$0.2 kPa$, $l_{min} = 0.15$	
SDL	0.05 ie $0.4 kPa$	
	$0.785 \times 9^2 \times 0.4$	= 25
		946 kN

$W_{t\,op}$.

SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: Date:

Engineer: Sheet No: A6 Scale:

Now using Limited ductility columns, pinned at top $\mu = 3$

We have 8 × cantilevered columns with ductile detailing. −500 ϕ at base − structurally

Can treat as "walls" max ductility $= \frac{5}{\beta_a}$

$A_r = \frac{4.8}{0.5} = 9.6$

big

$1.0 < \beta_a = 2.5 - \cancel{0.5 \times 9.6} < 2.0$

governs so $\mu_{max} = \frac{5}{1} = 5$

Use $\mu = 3$ for these columns

$C_d = 0.29$,

so $V_{base} = 0.29 \times 946 = 274$ kN

so $\bar{w}$ 8 columns

torsion

load/column $= \frac{274}{8} \times 1.1 = 38$ kN/col.

$M_{base} = 38 \times 4.8$ m

$= 181$ kN-m

SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: Date:

Engineer: Sheet No: A7 Scale:

$N_{col} = \frac{946}{8} = 118 kN$

$V_{col} = 38 \times$ dyn mag etc

$M_{col} = 181$ kN-m

$\bar{w}$ cover = 60 , $gD = 380$, $g = 0.76$

say $g = 0.7$

$\bar{w}$ table 500/30/0.7 $\frac{M_n}{D^3} = \frac{181 \times 10^6}{0.7 \times 500^3} = 2.07$

ϕ

gives $\rho_t = 0.015$ $\frac{N_n}{D^2} \quad \frac{118 \times 10^3}{500^2} = 0.47$

$A_{st} = 0.015 \times 500^2 \times 0.785$

$= 2947 mm^2$

use $\phi = 1.0$ for worst effect

$\boxed{8 - H025 \text{ provide}}$ - $A_{st} = 3925 mm^2$

Foundations

LDE recommend SWL = 70kPa to limit settlement to 25mm

Lateral spread upto 155mm

Need to transfer loads to gravel layer

LDE Recommend screw piles to Gravel layer

piles expected to be 2.7m to 3.7m deep

$\boxed{(\text{say } 3 \text{ to } 4m \text{ deep})}$

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SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: Date:

Engineer: Sheet No: A8 Scale:

Lateral movement → tie fdns together.

Run with a suspended main floor to keep masses down.

Clear span = 9.37 − 0.6
= 8.8m

say use 200 HollowCore or 200 Interspan +100 to 125 topping

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P: +643 366 7714 F: +643 366 7715 E: info@ruamoko.co.nz W: www.ruamoko.co.nz

SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: Date:

Engineer: Sheet No: A9 Scale:

Load on each pile

From Dome & top half of column =

946 ÷ 4 = 237 kN

Btm of column

0.5² × 0.785 × 2.5 × 24 = 12

Pilaster 0.7 × 0.8 × 2 × 24 = 27

Walls 0.3 × 3.6 × 2 × 24 = 52

Corbel 0.4 × 0.55 × 3.6 × 24 = 19

Floor at 2.4 + 3 = 5.4 kPa

3.6 × 4.2 × 5.4 = 81

Pile cap 1.2 × 1.1 × 0.45 × 24 = 14

Balaster = 12

Tie beam 0.3 × 0.4 × 4.2 × 24 = 12

G = 466

$Q = \frac{9}{2} \times 3.6 \times 5$ Q = 81

G + Q = 547

ie 55 tonne

SKETCHES/CALCS

Project Name: EDMONDS BAND ROTUNDA

Project No: 604 Date: 31/3/14

Engineer: COC Sheet No: B1. Scale:

GRAVITY DESIGN.

GRAVITY DESIGN IS PRIMARILY THE FLOOR STRUCTURE. ROOF STRUCTURE EXISTS.

CHECK HOLLOWCORE FLOORING

LOADS. ASSUME POTENTIAL OVERCROWDING.
ALLOW Q = 5.0 kPa.

ALLOW 150 TOPPING = $\frac{(150-65)\times 24}{1000}$ = 2.04 kPa SDL.

ALLOW 1.0 kPa OTHER SDL.

LIVE LOAD = = 5.0 kPa.
SDL = = 3.04 kPa.

FACTORED SUPERIMPOSED LOAD = 3.04 x 1.2 + 5 x 1.5
= 11.2 kPa.

SEE HOLLOWCORE CHART OVER

SPAN = 8.0m

OK AS SEVERAL LAYERS OF CONSERVATISM.

FACTORED LOAD = 11.2 kPa
MAX LOAD = 13.6 kPa (70m TOPPING)

TOPPING THICKNESS IN CENTRE = 150mm. : OK.

USE 200 HOLLOWCORE

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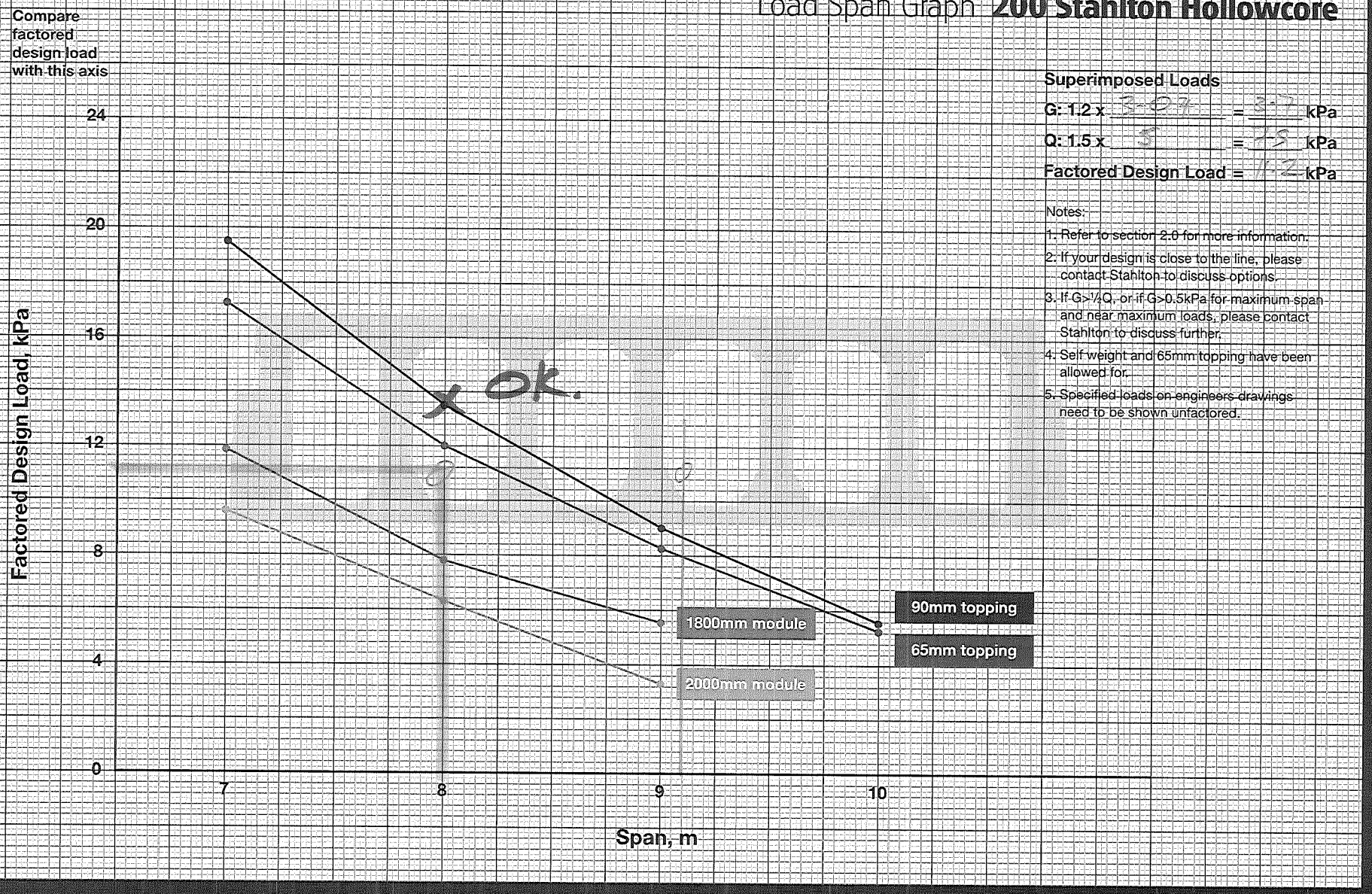
Load Span Graph **200 Stahlton Hollowcore**



Superimposed Loads

G: 1.2 x 3.00 **=** 3.7 **kPa**

Q: 1.5 x 5 **=** 7.5 **kPa**

Factored Design Load = 11.2 **kPa**

Notes:

1. Refer to section 2.0 for more information.
2. If your design is close to the line, please contact Stahlton to discuss options.
3. If G>¼Q, or if G>0.5kPa for maximum span and near maximum loads, please contact Stahlton to discuss further.
4. Self weight and 65mm topping have been allowed for.
5. Specified loads on engineers drawings need to be shown unfactored.

Project Name: [illegible] **Project Number:** 649

Level/Grid: Floor **Date:** 31/3/14

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Stahlton ENGINEERED CONCRETE

SKETCHES/CALCS

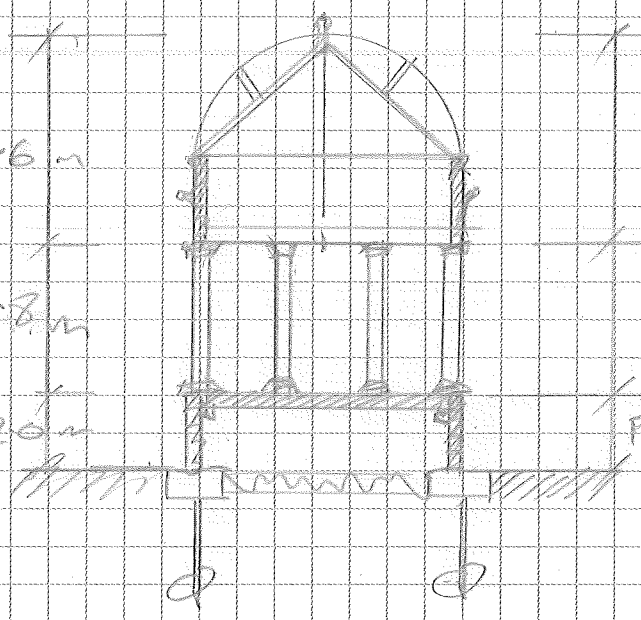
Project Name: EDMONDS ROTUNDA

Project No: 604 Date: 31/3/14

Engineer: COG. Sheet No: C1 Scale:

SEISMIC DESIGN.

REFER PG A1 FOR $C_d(T)$ CALCS
REFER A5-A6 FOR SEISMIC WEIGHT & JUSTIFICATION OF $\mu = 3$ FOR SEISMIC DESIGN OF COLUMNS.



PLINTH IS TECHNICALLY ABOVE GROUND, BUT OBVIOUS WEAK POINT IS THE COLUMN LEVEL. ∴ COLUMN DESIGN CRITICAL.

USE SEISMIC WEIGHT OF ROOF + HALF OF COLUMNS.

W_t = 950 kN. (PG A5)

USING $\mu = 3$

$C_d(T_1)$ = 0.29

∴ V_{base} = 0.29 × 950 = 276 kN.

LOAD/COLUMN = 276 × 1.1 (TORSION) × $\frac{1}{8}$

= 38.1 kN / COLUMN.

× $\frac{0.55}{0.29}$ = 72 kN

SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: 604 Date: 1/4/14

Engineer: COG Sheet No: C2 Scale:

$3.8 \times 4.8\,m = 18.2\ kNm$

AXIAL LOADS / COLUMN $= \frac{950}{8} = 119\ kN$

COLUMN DESIGN. - WITH 60mm COVER TO HD25 BARS $\phi = 0.7$

COLUMNS ARE 500mm DIAMETER.

$\frac{P}{D^2} = \frac{119}{0.5^2} = 0.48\ MPa \rightarrow$ USE $\phi = 1.0$

$\frac{M}{D^3} = \frac{18.2}{0.5^3} = 1.46\ MPa. \rightarrow$ USE $\phi = 0.7$

$\therefore = 2.09\ MPa$

USE 500/30/07 CHART FOR COLUMN DESIGN IN NZ REINFORCED CONC DESIGN HANDBOOK.

$\rho_t = 0.015$

$\therefore A_s = 500^2 \times \frac{\pi}{4} \times 0.015 = 2945\ mm^2$

8 HD25 $= 8 \times 491 = 3928\ mm^2 > 2945\ mm^2$ OK.

USE 8 - HD 25

SHEAR REINFORCING

$V_c = k_a k_n V_t A_{cv}$

$k_a = 1.0$

$k_n = \left[1 + \frac{3N^*}{A_g f'_c}\right] \geq 0$

$= \left[1 + \frac{3 \times 119 \times 1.35}{0.196 \times 30}\right] = 1.08$

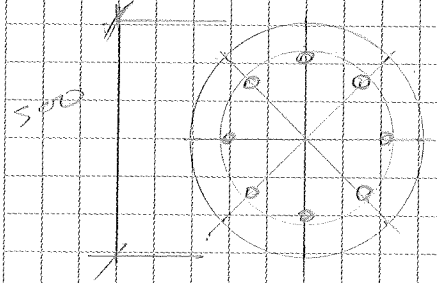
$V_t = (0.07 + 100\rho_w) \sqrt{f'_c}$

$= 1.0\ MPa.$

$A_{cv} = 113411\ mm^2\ (0.113\ m^2)$

$\therefore V_c = 1.0 \times 1.08 \times 1.0 \times 0.113 \times 10^3$

$= 122.5\ kN.$



SKETCHES/CALCS

Project Name: EDMONDS ROTUNDA

Project No: 604 Date: 1/4/14

Engineer: COG Sheet No: C3 Scale:

FOR SPIRALS – $V_s = \frac{\pi}{2} \cdot \frac{A_t \cdot f_{yt} \cdot d''}{s}$

$A_t = 112\ mm^2$ R12.
$f_{yt} = 300\ MPa$.
$d'' = 380 + 12 = 392\ mm$
$s = 120mm$ PITCH.

$V_s = 172.4\ kN$.

$\phi V_n = 0.75(122.5 + 172.4)$
$= 221\ kN \quad > 72 \therefore OK.$

USE R12 SPIRALS @ 120mm PITCH

TRANSFER OF ROOF LOADS to COLUMN.

$950\ kN \times 0.55 \quad (1.4 + 1.25)$
$= 523\ kN.$
$= 65.3\ kN$ / COLUMN.

SHEAR CAPACITY OF HD25
$= 0.9 \times 491 \times 500 = 221\ kN > 65 \therefore OK.$

USE HD25 DOWELS

SKETCHES/CALCS

Project Name: EDMONDS BAND ROTUNDA

Project No: 604 Date: 4/4/14.

Engineer: COG. Sheet No: C4 Scale:

CHECK FOUNDATION WALLS.

ADDITIONAL SEISMIC WEIGHT DUE TO FLOOR & UPPER PORTION OF PERIMETER WALLS:

WL FLOOR = 7.5 kPa × 76 m² = 570 kN SELF WEIGHT

+ 0.3 × 5 × 76 = 114 kN Q.C.

+ 0.1 × 24 × 0.8 × 3.2 × 8 = 49 kN BALUSTRADES

+ 1 × 0.8 × 24 × 3.2 × 8 = 590 kN PERIMETER WALL

= 1300 kN

WL AT FOUND WALL

= 946 + 1300 + 89 + 47

= 2385 kN.

VE AT FOUND WALLS

= 2385 × 0.29 = 692 kN

ASSUMING ALL SHEAR IS TAKEN BY 2 WALLS ONLY

THEN $\frac{692}{2}$ = 346 kN / WALL

WALLS ARE 3.8 m LONG, AND 0.25 m THICK.

∴ $v^* = \frac{346}{3.8 \times 0.25}$ = 0.36 MPa

SKETCHES/CALCS

Project Name: EDMONDS BAND ROTUNDA

Project No: 604 Date: 4/4/14

Engineer: CAG Sheet No: C5 Scale:

$V_u^* = 364$ kN @ $\mu = 3$

$\phi V_n = 1770$ kN (2-HD12@300 T2 each)

$M_u^* = 364 \times 2 = 728$ kNm

$\phi M_n = 3802$ kNm (2 HD16@300 T2 EW)

USE 250 THICK CONC WALLS 30MPa WITH HD16 @ 300 VERTICAL & HD12 @ 300 HORIZONTAL TO EACH FACE

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Project Name:	Edmonds Band Rotunda		
Project No:	604	Date:	4-Apr-14
Calcs By:	COG	Page No:	C6

FOUNDATION WALL FLEXURAL STEEL DESIGN

MATERIAL PROPERTIES

Concrete

Concrete strength, f'c	=	30	MPa
Ult conc. Strain, ε_u	=	0.003	

Steel

Yield Stress	=	500	MPa
Overstrength Yield Stress	=	675	MPa
Elastic Modulas	=	200000	MPa
Yield Strain, ε_s	=	0.0025	
Ult Steel Strain, ε_s	=	0.11	

FLEXURAL REINFORCING

Reinforcing Bar	=	HD16	
No of layers	=	2	
Spacing	=	300	mm
Cover	=	75	mm

GEOMETRY

Wall Length	=	3800	mm
Web thickness	=	250	mm

WALL ANALYSIS

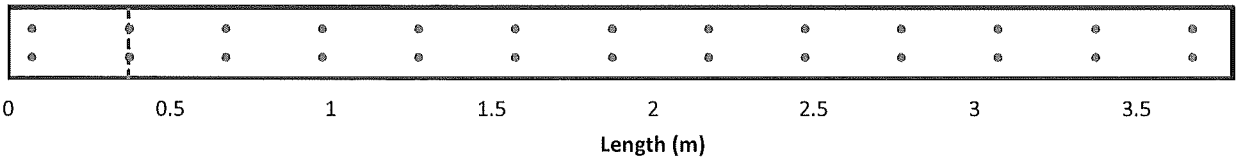
Axial Load, N*	=	0	kN
Ideal Capacity, M_n	=	4473	kNm
Dependable Capacity, ϕM_n	=	**3802**	**kNm**
Netral Axis Depth	=	*372.0*	*mm*
O'Strength capacity, M_o	=	**5983**	**kNm**
O'strength depth to n.a., c_o	=	*394.1*	*mm*

Reinforcing Size and Location

*Distance measured from comp. end

	Reinf. Bar Size	No. of Reinf. Bars	Area of Rein. (mm^2)	Reinf. Position* x_i (mm)
1	HD16	2	402	75
2	HD16	2	402	375
3	HD16	2	402	675
4	HD16	2	402	975
5	HD16	2	402	1275
6	HD16	2	402	1575
7	HD16	2	402	1875
8	HD16	2	402	2175
9	HD16	2	402	2475
10	HD16	2	402	2775
11	HD16	2	402	3075
12	HD16	2	402	3375
13	HD16	2	402	3675

Reinforcing Layout



NZ Blood

Shear Wall shear design

Wall	Level / Height	$M^*_{(analysis)}$	V^*	N^*	L	TW	$M^*_{(Tjd)}$	o.s factor, $\phi_{o,w}$	ϕ	ω_v	$\mu^* V_{E\,(1.25)}$	$V^*_{wall}=\omega_v\phi_{o,w}V_E$	Design V^*_{wall}	v_n^*
		kNm	*kN*	*kN*	*mm*	*mm*	*kNm*					*kN*	*kN*	*MPa*
W1 - 4.0m	**0-3.6**	692	346.0	0.0	3800.0	250.0	692.0	1.0	0.75	1.0	*432.5*	346.0	346.0	0.46
PHZ	**0-3.6**	692	346.0	0.0	3800.0	250.0	692.0	1.0	0.75	1.0	*432.5*	346.0	346.0	0.46
W2 - 3.3m	**3.6-7.2**	835	614.0	90.0	3300.0	250.0	835.0	1.0	0.75	1.0	*767.5*	614.0	614.0	0.93
PHZ	**0-3.6**	835	614.0	90.0	3300.0	250.0	835.0	1.0	0.75	1.0	*767.5*	614.0	614.0	0.93
W3&4 - 5.0m	**3.6-7.2**	3092	1628.0	135.0	5000.0	250.0	3092.0	1.0	0.75	1.0	*2035.0*	1628.0	1628.0	1.63
PHZ	**0-3.6**	3092	1628.0	135.0	5000.0	250.0	3092.0	1.0	0.75	1.0	*2035.0*	1628.0	1628.0	1.63
W5&6 - 5.0m	**3.6-7.2**	4579	2411.0	135.0	5000.0	250.0	4579.0	1.0	0.75	1.0	*3013.8*	2411.0	2411.0	2.41
PHZ	**0-3.6**	4579	2411.0	135.0	5000.0	250.0	4579.0	1.0	0.75	1.0	*3013.8*	2411.0	2411.0	2.41
W7-10 - 4.0m	**0-3.6**	3560	1872.0	235.0	4000.0	250.0	3560.0	1.0	0.75	1.0	*2340.0*	1872.0	1872.0	2.34
PHZ	**0-3.6**	3560	1872.0	235.0	4000.0	250.0	3560.0	1.0	0.75	1.0	*2340.0*	1872.0	1872.0	2.34

Potential Plastic Hinge Length

11.4.3			11.4.2.2	11.3.2	11.3.1.2
NZS3101	*Priestley (DBD)*	Design		minimum thickness	M*>N*x 0.05t
L_p	L_p	L_p	N*<0.05f'cAg	100mm	Min Design M*
3600	*1690*	3600	1662.5		0.00
			1662.5		0.00
3300	*1640*	3300	1443.75		1.13
			1443.75		1.13
3600	*1810*	3600	2187.5		1.69
			2187.5		1.69
3600	*1810*	3600	2187.5		1.69
			2187.5		1.69
3600	*1710*	3600	1750		2.94
			1750		2.94

f'_c = 35 MPa

fu = 675

fy = 500

fyt = 500

μ = *1.25*

k = *0.07*

Lsp = *365*

doubly reinforce

Therefore dimesional limitations of 11.4.2

NZ Blood

Shear Wall shear design

Shear 11.3.10
11.3.10 11.3.10.3.5

11.3.10.3.8

Additional design for ductility in Earthquakes

11.4.5

Anti-buckling
11.4.6.3 11.4.6.3

Limit to diam of bar

Plastic Hinge Regions

$V^* <= \phi V_n$ $v_n = v_c + v_s$

$V^* <= \phi V_n$

Wall	Level / Height	v_c	$M^*/V^* - L/2$	v_c	$V_c = v_c A_{cv}$	v_n^*	s2 =	$V_{s\ required}$	$A_{v\ required}$	$Av>0.7*b_w *s_2/f_{yt}$	Bar size	no of bars	$A_{v\ available}$	V_s	$\phi V_n = V_c + V_s$	Max diam bar	Vertical Bar Diamter	Vertical Reinf. Area at s_v	s_v	p_l	3/fy - limited
			mm		*kN*			*kN*	mm^2	mm^2				*kN*	*kN*			mm^2	*mm*		
W1 - 4.0m	**0-3.6**	1.60	-1900.0	1.60	1214.0	0.46	300.00	-752.6	-148.55	105	HD12	2	226.2	1146.1	1770.0	31.25	12	226.2	300	0.00302	
PHZ	**0-3.6**	1.60	-1900.0	1.60	1214.0	0.46	300.00	-752.6	-148.55	105	HD12	2	226.2	1146.1	1770.0	31.25	16	402.1	300	0.00536	0.006
W2 - 3.3m	**3.6-7.2**	1.62	-1650.0	1.62	1072.2	0.93	300.00	-253.6	-57.63	105	HD12	2	226.2	995.3	1550.6	31.25	12	226.2	300	0.00302	
PHZ	**0-3.6**	1.62	-1650.0	1.62	1072.2	0.93	300.00	-253.6	-57.63	105	HD12	2	226.2	995.3	1550.6	31.25	16	402.1	300	0.00536	0.006
W3&4 - 5.0m	**3.6-7.2**	1.62	-2500.0	1.62	1624.3	1.63	300.00	546.3	81.95	105	HD12	2	226.2	1508.0	2349.3	31.25	12	226.2	300	0.00302	
PHZ	**0-3.6**	1.62	-2500.0	1.62	1624.3	1.63	300.00	546.3	81.95	105	HD12	2	226.2	1508.0	2349.3	31.25	16	402.1	300	0.00536	0.006
W5&6 - 5.0m	**3.6-7.2**	1.62	-2500.0	1.62	1624.3	2.41	300.00	1590.3	238.55	105	HD16	2	402.2	2681.3	3229.3	31.25	12	226.2	300	0.00302	
PHZ	**0-3.6**	1.62	-2500.0	1.62	1624.3	2.41	300.00	1590.3	238.55	105	HD16	2	402.2	2681.3	3229.3	31.25	16	402.1	300	0.00536	0.006
W7-10 - 4.0m	**0-3.6**	1.66	-2000.0	1.66	1324.9	2.34	300.00	1171.1	219.59	105	HD12	2	226.2	1206.4	1898.5	31.25	12	226.2	300	0.00302	
PHZ	**0-3.6**	1.66	-2000.0	1.66	1324.9	2.34	300.00	1171.1	219.59	105	HD12	2	226.2	1206.4	1898.5	31.25	16	402.1	300	0.00536	0.006

d = 0.8*Lw

!.2. do not apply

Limited Duct

SKETCHES/CALCS

Project Name: EDMONDS BAND ROTUNDA

Project No: 604 Date: 4/4/14

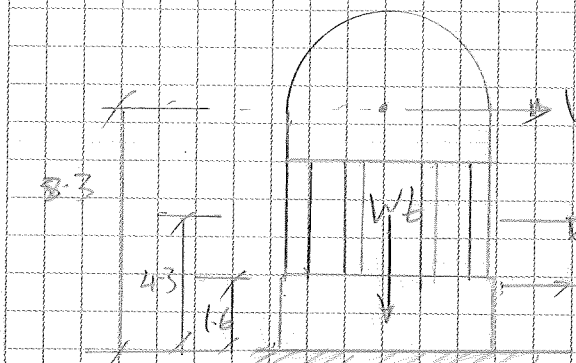
Engineer: COC Sheet No: D1 Scale:

CHECK OVERALL STABILITY.

SEISMIC WEIGHT = 2385 kN.

USE $\mu = 2$ FOR STABILITY ASSESSMENT → ASSUMES FOUNDATION ROCKING MODE.

$V_b = 2385 \times 0.4 = 954$ kN.



$M_o = 343 \times 8.3 + 71 \times 4.3 + 540 \times 1.6$

$= 4016$ kNm

$M_R = 2385 \times \frac{9.1}{2} = 10,850$ kNm $>> 4016$ ∴ OK.

→ NO ISSUES WITH OVERTURNING.

PILE LOADS

TOTAL V_G = 3728 kN (FROM A9)

= 466 kN/PILE - 8 PILES

COMBINED G+E = 652 kN/PILE WORST CASE

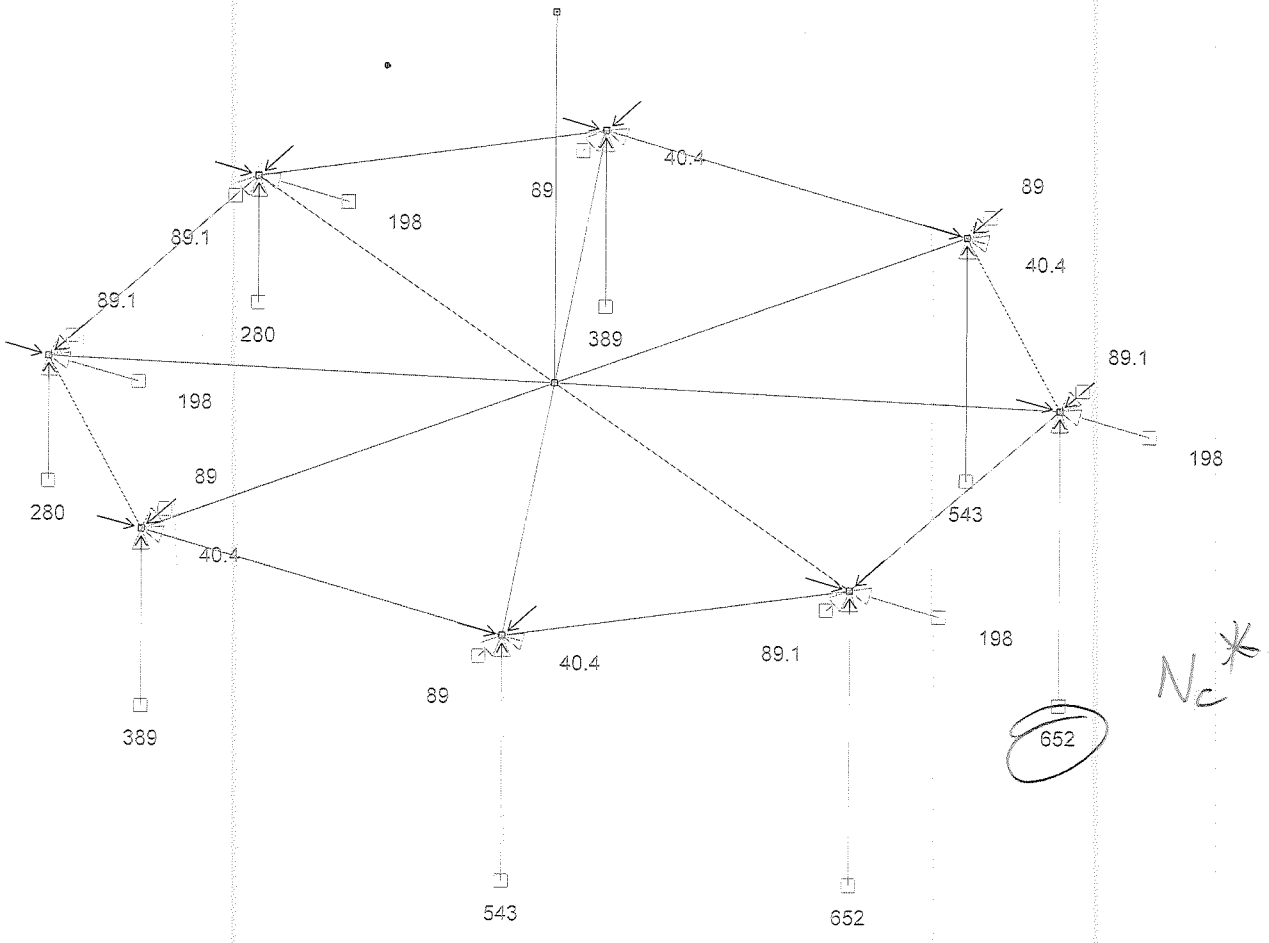
NO UPLIFT LOADS.

Microstran V9

Ruamoko Solutions Ltd
Job: Edmonds Band Rotunda Foundations
Foundation load check

04/04/2014
11:14:55 a.m.

Load Cases:
3 C G+Eu



Nc*

Y
Z
X

theta: 300 phi: 30

Overturning loads to piles

Support Reactions

SKETCHES/CALCS

Project Name: EDMONDS BAND ROTUNDA

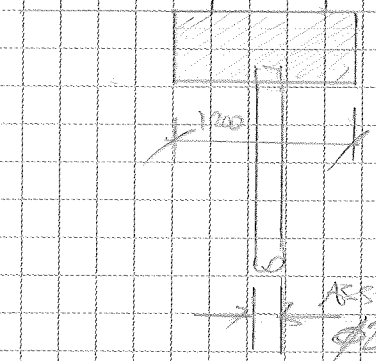
Project No: 6021 Date: 4/4/14

Engineer: COG. Sheet No: D3 Scale:

PILE LOADS THEREFORE

G = 466 kN
Q = 81 kN
E = 186 kN } ALL UNFACTORED.

PILE CAPS



PILE LOADS

1.2G + 1.5Q	=	681 kN
1.35G	=	629 kN
G + E	=	652 kN

$V^* = 681$ kN.

ASSUME LOAD IS TRANSFERRED IN OUTER 200mm OF PILE CAP

$$M^* = \frac{PL}{4} = \frac{681 \times 1.0}{4} = 170 \text{ kNm}$$

USE 6 HD20 E.W. TOP & BOTTOM.
ADD 6 LEGS R12 STIRRUPS @ 150 c/c.

CALCULATIONS

RUAMOKO solutions
CONSULTING STRUCTURAL ENGINEERS

Project Name:	Edmonds Band Rotunda		
Project No:	604	Date:	4-Apr-14
Calcs By:	COG	Page No:	

CONCRETE BEAM FLEXURAL DESIGN

GEOMETRY

Beam Length, L	=	1200 mm
Beam Depth, h	=	450 mm
Beam Width, b	=	1100 mm
Cover	=	75 mm
d	=	375 mm

α =	0.85
β =	0.85

MATERIAL PROPERTIES

Reinforcing Bar = HD20

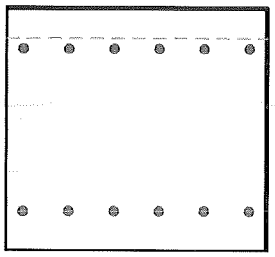
Concrete

Concrete stress, f'_c	=	30 MPa
Ult conc. Strain, ε_u	=	0.003

Steel

Yield Stress	=	500 MPa
Overstrength Yield Stress	=	675 MPa
Elastic Modulas	=	200000 MPa
Yield Strain, ε_s	=	0.0025
Ult Steel Strain, ε_s	=	0.11

Reinforcing Layout



BEAM ANALYSIS

Ideal Capacity, M_n	=	351 kNm	Neutral Axis Depth
Dependable Capacity, ϕM_n	=	**298 kNm**	55.8 mm
O'Strength Capacity, M_0		468 kNm	57.3 mm
Ideal Negative Capacity, M_n	=	*-351 kNm*	
Negative bending capacity, ϕM_n	=	*-298 kNm*	*394.2 mm*
Negative O'Strength Capacity, M_0	=	*-468 kNm*	*392.7 mm*

Reinforcing Layout

*Distance measured from comp. end

Bar Layer	No. of Reinf. Bars	Depth x_i (mm)	d_b (mm)	Area of Reinf. (mm^2)	Linear Strain ε	Stress (MPa)	Force (kN)	Moment (kNm)
1	6	75	20	1885	0.00103	206.1	388.50	7.45
2								
3								
4	6	375	20	1885	0.01715	500.0	942.48	300.82
5								
6								
7								
8								
9								
10								
						Total:	1331.0	308.3

steel force	=	1331
concrete force	=	-1331
sum of forces	=	0
steel moment	=	308
concrete moment	=	43

9.3.9.3.4

Beam	Desc.	bw *mm*	d *mm*	A_s *mm²*	V* *kN*	Max Nom Shear Stress, v_n *MPa*	v_b *MPa*	v_c *MPa*	V_c *kN*	$V_{s,reqd}$ *kN*	spacing, s *mm*	$A_{v,reqd}$ *mm²*	$A_{v,min}$	No. of legs	Stirrup Size *mm*	$A_{v,prov}$ *mm²*	ϕV_n *kN*	v_n* *MPa*
Pile Cap		1100	375	3927	681	6.0	0.59	0.59	244	557	150	445.8	113.0	6	12	678.6	928.4	2.65
						6.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	150	#DIV/0!	0.0	6	16	1206.4	#DIV/0!	#DIV/0!
						6.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	150	#DIV/0!	0.0	6	12	678.6	#DIV/0!	#DIV/0!
						6.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	150	#DIV/0!	0.0	6	12	678.6	#DIV/0!	#DIV/0!

f_c' = 30 MPa

ϕ = 0.85

f_{yt} = 500 MPa

k_a = 1.0

k_d = 1.0