



## SPECIFIC ENGINEERING DESIGN CALCULATIONS

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### PROJECT DETAILS:

**PROJECT:** Proposed Tank  
**CLIENT:** Ross Tank  
**SITE:** Exploration Way  
Whitby  
**ARCHITECT:** N/A  
**JOB REF:** 18549  
**DATE:** 3 November 2015  
**BY:** Gareth Jew

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### DESIGN SUMMARY:

- Specific design calculations in accordance with NZS3101 & NZS 3106 & NZSEE 'Seismic Design Of Storage Tanks: 2009'



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# GENERAL SPECIFICATION

## BRIEF OUTLINE OF STRUCTURAL REQUIREMENTS / SPECIFICATIONS

### 1.0 GENERAL

Engineering details are to be read in conjunction with all other relevant documentation.

Do not scale any drawings. For setting out information refer to Architect's documentation.

The contractor is to ensure temporary stability of the building and ground at all times. Any temporary loading conditions imposed by the construction techniques are to be checked by the contractor.

All workmanship and construction practice is to be to the latest editions of NZ Standards (and other relevant standards).

All work is to be carried out to the NZ Building Code and any Territorial Authority requirements.

Any engineering discrepancies should be immediately referred to the Engineer for resolution.

For any nominated products, acceptable alternative products may be submitted by the Contractor for approval.

Contractor and fabricators shall confirm all dimensions prior to commencement of work. Discrepancies, if found, shall be brought to the immediate attention of the Architect.

### ABBREVIATIONS

UB	Universal Beam e.g. a 250UB 31.4 is a 252x146 'I' beam and weighs 31.4kg per metre
TFB	Tapered Flange Beam e.g. 125TFB 13.1
UC	Universal Column e.g. a 200UC 46.2 is a 203x203 'I' beam and weighs 46.2kg per metre
EA	Equal Angle e.g. 50x50x6 EA has 6mm thick sides
UA	Unequal Angle e.g. 125x75x10 UA
CHS	Circular Hollow Section e.g. 102x8.6 CHS has an 8.6mm wall thickness
SHS	Square Hollow Section e.g. 89x89x3.6 SHS has 3.6mm wall thickness
RHS	Rectangular Hollow Section e.g. 89x38x4.0 RHS
PFC	Parallel Flange Channel e.g. 200PFC is a 200x75 channel
P/C	Pre-camber
PL	Steel Plate
SG	Stress Graded timber, available in grade 6, 8, 10, and 12
G	Only available in G8, verified green timber, typically used in wet service situations
GL	Glulam Timber
c/c	Centre to centre distance (also CTRS)

### 2.0 DESIGN LOADS

2.1 All imposed loads have been calculated in accordance with AS / NZS 1170.1, and structural elements designed for the following loads.

2.2 Permanent Loads

ELEMENTS	LOADINGS kPa (Plan area)
Timber Floors	0.5
Timber Decks	0.5
Heavy/Light Roof	0.85/0.45

ELEMENTS	LOADINGS kPa (Face area)
Timber Framed walls	0.5
Timber Framed walls + brick veneer	2.1
20/25 Series Masonry	4.1/5.1

2.3 Imposed Loads

Residential Elements	LOADINGS	
	Udl Q <sub>b</sub> kPa	Pt Loads Q <sub>b</sub> kN
Decks	2.0	1.8
Roofs	0.25	1.1
Garages	2.5	13*
All Other floors	1.5	1.8

\*For garages with timber floors, this can be reduced to 9kN acting over 300mm x 300mm.

## 2.4 Specific Loads

The following specific loads have been allowed for: Nil.

## 3.0 SITE WORKS – GENERAL

- 3.1 Origin of levels is the LINZ datum – refer to Survey drawings.
- 3.2 All drawings to be read in conjunction with the specification.
- 3.3 Existing services are shown indicatively only. Before construction commences, it is the contractor's responsibility to locate the position and depth of all services that could be adversely affected by their operations. Inform Architect of any variations.
- 3.4 All work to be in accordance with the Territorial Authority standards.
- 3.5 Concrete for site works only to be 10MPa unless noted otherwise.
- 3.6 Catchpit positions and grate levels are approximate. Final positions are to suit kerb levels.
- 3.7 All work on public drains is to be carried out by an approved licensed drain layer, to the relevant Territorial Authorities' standards.

## 4.0 SITE WORKS – EARTHWORKS

- 4.1 Temporary batters to have a 1V:1H maximum slope, but also refer to any specific requirements in the geotechnical report.
- 4.2 Work to include the stripping of vegetation, topsoil, and any surface deleterious matter. This includes the breaking out of substructures and hard paving where required.
- 4.3 Care is to be exercised adjacent to existing trees that are to be retained. Any excavations shall be performed by hand in the vicinity of existing trees.
- 4.4 The subgrade shall be inspected by the Engineer and proof rolled as necessary. Any soft spots shall be over-excavated and compacted back to level with imported GAP65 material.

## 5.0 FOUNDATIONS

- 5.1 Unless noted otherwise, foundations have been designed in accordance with the 'good ground' assumption as defined by NZS 3604 2011 for:
  - 100kPa safe bearing capacity (under unfactored loads).
  - 150kPa dependable bearing capacity (under factored loads).
- 5.2 All founding material is to be approved by the Engineer before any concrete, including blinding concrete, is placed. Note that all footings are to be founded in original ground or certified engineered earth fill as approved by the Engineer. Refer also to the requirements of any specific Geotechnical Report.
- 5.3 No services excavations shall be formed within a 1:1 line of the underside of any foundation unless specifically instructed by the Engineer.
- 5.4 Slabs on grade are to be constructed onto prepared basecourse.
- 5.5 All backfill behind retaining walls, if any, is to be of approved free-draining material. Drain coils are to be fitted to drain, to approved silt traps.

## 5.0 REINFORCED CONCRETE

6.1 Minimum 28 day compressive concrete strengths to be as follows:

Exposure Zone (as defined in NZS 3604: 2011)	GRADE (MPa)
B	17.5
C	20
D	25

6.2 Concrete sizes indicated on the drawings do not include the thickness of applied finishes.

6.3 Chamfers to be formed on all exposed corners, generally 15mm x 15mm.

6.4 Sizes indicated show depth x width for beams. Depth excludes depth of slab unless noted otherwise.

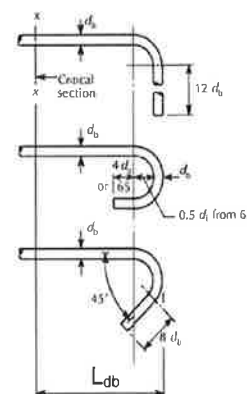
6.5 If construction joints are not shown, their location shall be the contractor's responsibility to suit the conditions of the design. The Contractor is to allow for all construction joints necessary to complete the work, without causing distress to the structure. Location to be agreed with Engineer. In general, for slabs on grade, saw cuts (25mm deep x 5mm wide) are to be formed to produce a grid of 5.0m x 5.0m maximum, after the initial cure (approximately 24 – 48 hours after concrete pour).

6.6 All penetrations are to be located to the written approval of the Engineer and as shown on the drawings. Services to be cast in where shown.

6.7 Ensure adequate gaps maintained between suspended concrete work and all non-structural elements.

6.8 Finish to concrete to be as follows (if not specified elsewhere):

ELEMENT	TYPE OF FINISH (to NZS 3411)
Walls and all exposed concrete faces	F5 with special features – see Architects drawings
Floor slab	U3 for application of floor finish – see drawings



6.9 For all proprietary concrete materials, full calculations and shop drawings are to be submitted to the Engineer in adequate time by the Contractor for approval. These are to show full compliance with the design intent. Where necessary this documentation is to be submitted to the Territorial Authority for the necessary Building Consents.

## 7.0 REINFORCEMENT

### Hook definitions;

- A 90° turn plus an extension of at least 12 bar diameters at the free end of the bar
- A semi-circular turn plus an extension of at least 4 bar diameters but not less than 65mm at the free end of the bar
- A stirrup hook, which is defined as a 135° turn around a longitudinal bar plus an extension of at least 8 stirrup bar diameters for plain bars and 6 stirrup bar diameters for deformed bars at the free end of the bar embedded in the core concrete of the member

### Minimum lap length as follows:

- Masonry lap lengths;  
Grade 300 reinforcing = 40 x bar diameter  
Grade 500 reinforcing = 70 x bar diameter

- Concrete lap lengths;

$$L_{db} = \frac{(0.5\alpha_a f_y) d_b}{\sqrt{f_c}}$$

Where  $\alpha_a = 1.3$  for top reinforcement where more than 300mm of fresh concrete is cast in the member below the bar, or 1.0 for all other cases.

For example; ( $\alpha_a = 1.0$ )

Concrete Strength	Steel Strength	Lap length required
20MPa	300MPa	34 x $d_b$
20MPa	500MPa	56 x $d_b$
25MPa	300MPa	30 x $d_b$
25MPa	500MPa	50 x $d_b$
30MPa	300MPa	28 x $d_b$
30MPa	500MPa	46 x $d_b$

**Minimum bend diameters of Class E steel bars, with yield strength 300MPa or 500MPa**

a. Normal

- i. Bar diameter ( $d_b$ ) 6-20mm =  $5d_b$
- ii. Bar diameter ( $d_b$ ) 24-32mm =  $6d_b$

b. Stirrups or ties

i. For **deformed** bars:

1. Bar diameter ( $d_b$ ) 6-20mm =  $4d_b$
2. Bar diameter ( $d_b$ ) 24-32mm =  $6d_b$

ii. For **plain** bars:

1. Bar diameter ( $d_b$ ) 6-20mm =  $2d_b$
2. Bar diameter ( $d_b$ ) 24-32mm =  $3d_b$

For example;

Bar Diameter	Type	Required Bend Diameter ( $d_b$ )
R6	Stirrup or tie	12mm
R10	Stirrup or tie	20mm
D10	Stirrup or tie	40mm
D10/H10	Normal	50mm
D12	Stirrup or tie	48mm
D12/H12	Normal	60mm
D16/H16	Normal	80mm
D20/H20	Normal	100mm
D24/H24	Normal	144mm
D32/H32	Normal	192mm

**Minimum cover to reinforcement:**

a. Masonry:

Exposure Classification	Minimum cover (mm)
Coastal Frontage*	60
Coastal Perimeter	50
External (inland)	45
Closed interior	35

b. Concrete:

Exposure Classification	Concrete Strength (MPa)				
	17.5	20	25	30	40
	Minimum cover (mm)				
Closed Interior	30	25	25	20	20
External (Inland)	50	40	35	30	25
Coastal Perimeter	65	50	40	35	30
Coastal Frontage*	-	-	50	45	40
Cast against Ground	75	75	75	75	75
Cast against Ground with DPC	50	50	50	50	50

\* Within 100m of high tide mark or within 500mm of high tide mark to the direction of the prevailing wind

7.1 All reinforcement shall comply with the following:

SYMBOL	TYPE
R	Structural grade plain bars to NZS 3402 (300MPa)
D	Structural grade deformed bars to NZS 3402 (300MPa)
HD/H	Deformed bars grade 500 to NZS 3402 (500MPa)
HR	Plain bars grade 500 to NZS 3402 (500MPa)

7.2 Minimum distribution steel shall be D12 at 300 centres.

7.3 Reinforcement laps to be to NZS 3109. Mesh lap to be one mesh + 25mm.

7.4 Adequate support to all reinforcement to be made by the use of spacers, stools and the like.

7.5 All bending of reinforcement to be to NZS 3109.

7.6 Notation used for reinforcement fixing:

B/W	Both ways
B/F	Both faces
E/F	Earth face
V	Vertical
H	Horizontal
N/F	Near face
BOT	Bottom
ABR	Alternative bars reversed

## 8.0 STRUCTURAL TIMBER

- 8.1 Timber beams shall be propped until dry to prevent premature creep.
- 8.2 All normal timber sizes given are nominal dimensions unless otherwise stated.
- 8.3 All glue laminated timber sizes are actual dimensions.
- 8.4 Timber treatment shall be specified and shall comply with the recommendations of the Timber Preservation Authority. Generally all timber to be treated to the requirements of the Building Code. Refer to Architects drawings / specifications.

<b>Hazard</b>	<b>Hazard Class</b>	<b>Examples</b>
Low	H1.1	Interior linings, internal wall
	H1.2	Wall framing, skillion roof framing
Moderate	H3.1	Claddings, fascia, wall framing
	H3.2	Deck bearers, joists, boards
High	H4	Fence posts, landscaping timber
Severe	H5	House piles, retaining walls
Marine	H6	Marinas, wharf piles

- 8.5 All structural timber to be stress graded SG8 Pinus Radiata unless noted otherwise.
- 8.6 Where timber connections are exposed to the weather, all steel connection shall be galvanised (unless in the sea spray zone in which case they are to be stainless steel). Faces and edges of steel elements which rest against tanalised timber shall be painted before positioning. Minimum connections are 5mm plate and 2/M16 bolts unless noted otherwise.
- 8.7 All bolts and nuts to have washers as the following table:

	<b>ROUND (mm)</b>	<b>SQUARE (mm)</b>
M6	30 x 1.6	25 x 25 x 1.6
M8	36 x 2.0	32 x 32 x 2.0
M10	45 x 2.5	40 x 40 x 2.5
M12	55 x 3.0	50 x 50 x 3.0
M16	65 x 4.0	57 x 57 x 4.0
M20	75 x 5.0	65 x 65 x 5.0
>M20	85 x 6.0	75 x 75 x 6.0

## 9.0 STRUCTURAL STEELWORK

- 9.1 Steelwork drawings show the design intent. The Contractor is to prepare shop drawings for review. No fabrication is to commence until shop drawings are reviewed by the Engineer and written notification issued.
- 9.2 All reactions are shown in kN.
- 9.3 All protective coatings to steelwork to be in accordance with the architectural specifications. Generally, all exposed structural steel is to be galvanised, unless noted otherwise. All external flitch beams are to have plates galvanised and top, bottom and side edges full sealed.
- 9.4 All steelwork beams, girders, trusses and the like are to be precambered to compensate for the effects of dead load deflection as noted on the drawings.
- 9.5 Where the weight of a steel section is omitted the heaviest commonly available steel section shall be utilized.
- 9.6 All bolts to a minimum of M12 and all welds to be a minimum of 6mm continuous fillet (SP).
- 9.7 All beam connection plates to be a minimum thickness of 10mm. All other plates to be a minimum thickness of 6mm. All cap plates to be a minimum thickness of 3mm. All hollow sections to be sealed with cap plates.
- 9.8 M12 bolts where specified to the grade 4.6/S. All other bolts to be grade 8.8/S, unless noted otherwise.
- 9.9 All HD bolts to be fabricated from material with a minimum yield stress of 250MPa, set to template, checked for level and position before casting. All grout under base plates to have a compressive strength of at least 25MPa.

- 9.10 All nuts and turning bolts are to have approved washers. All exposed nuts, bolts and washers to be galvanised and to have protective coatings to match general steelwork, unless noted otherwise.
- 9.11 All cold formed sections, including cold rolled purlins to be to AS 1538. To be formed from zinc coated high strength steel strip with a minimum yield stress of 450MPa. All to be galvanised with a minimum coating of 300g/m<sup>2</sup>. All purlin connections to be to manufacturer's recommendations.
- 9.12 All rod bracing to be a minimum of grade 300 plain reinforcing bar, unless noted otherwise.
- 9.13 Steel members shall be to the following grades, unless noted otherwise.

<u>Member</u>	<u>Grade</u>
UB, UC, TFC, PFC, TFB, Angles	300+
RHS, SHS, CHS	350
WB, WC	300+
Stainless steel	250+ (grade 315L)

- 9.14 All sealed sections to be galvanized shall have vent holes. To be shown on shop drawings.
- 9.15 Purlin cleats are to be in accordance with the manufacturer's standard details except where the top flange of the purlin is more than 300mm above supporting steelwork 75 x 75 x 8 angle cleats shall be used. Purlins shall be fixed using approved flanged bolts and washers.
- 9.16 Ceiling systems, ductwork, etc. to be suspended from purlins should be fixed with hook bolts through purlin web. The flanges of the purlins or girts shall not be holed.

## 10.0 REINFORCED MASONRY

- 10.1 Generally all concrete block masonry construction to be in accordance with NZS 4229:2013, and NZS 4210:2001.
- 10.2 Refer to figure 8.1, NZS 4229:2013 for D16 Trimmers around openings.
- 10.3 The top of all the concrete block walls to be continuous Bond Beam B1 or B2, as per figure 10.1 and 10.2, NZS 4229:2013. Continuous Bond Beam type B3 (figure 10.2, NZS 4229:2013) to be provided at mid-floor level of 2 storey walls.
- 10.4 Minimum reinforcing to be as follow unless shown otherwise:

Bond	15 Series		20 Series	
	Vertical	Horizontal	Vertical	Horizontal
Running	D12@800crs	D12@800crs	D12@600crs	D12@600crs
Stack	D10@400crs	D10@400crs	D12@400crs	D12@400crs

- 10.5 In general only load bearing masonry is shown on the structural drawings. All non-load bearing masonry is to be separated from structure by a 12mm compressible joint. All load bearing masonry, including retaining walls, shall be a minimum of Grade B.
- 10.6 Concrete blocks shall have a minimum compressive strength of 12.5MPa, to NZS 4210. Density to be greater than 1750kg/m<sup>3</sup>.
- 10.7 Mortar shall comply with the requirements of NZS 4210 to a nominal mix of:  
1 part cement / 3 parts sand / ½ part lime.
- 10.8 All cores with reinforcement to be grout filled as a minimum. Additional covers filled as noted on the drawings. Grout shall have a slump of 230mm and a compressive strength in accordance with NZS 3604:2011 clause 4.5.3.
- 10.9 In general, walls to be full height before grouting cores. Washout ports to be provided at bottom course where vertical bars are located for all walls where poured height exceeds 1.2m. Mortar joints to be 10mm thick with blocks fully bedded and perpend filled. Joints to be tooled at exposed or rendered surfaces.

Cores are to be cleaned of all mortar fins and droppings through washout ports which are not to be closed until inspected by Engineer.

Grout to be added to ensure fill of cores with a maximum continuous pour height of 3600mm.

- 10.10 No construction of masonry on suspended floors is to take place until floor is fully depropped.
- 10.11 All Grade A masonry to be supervised by a registered mason, including grouting. All masonry to be inspected by Engineer prior to grouting.
- 10.12 Chasing of load bearing masonry only permitted as shown on Engineer's drawings.

These calculations and associated sketches have been prepared using the information supplied by the client at the time of engagement. Ian Hutchinson Consultants Limited take no responsibility for any discrepancies in the information supplied or any subsequent changes made without their knowledge. The calculations are solely for the benefit of the client and the appropriate Territorial Authority. No responsibility is taken for use by other parties without prior arrangement.



**Tank Dimensions**

Radius (inside)	$R =$	2.89 m
Height (inside)	$H =$	2.05 m
Wall thickness	$t_w =$	110 mm
Base thickness	$t_b =$	125 mm
Roof thickness	$t_r =$	125 mm
Radius of foundation	$R_b =$	3 m

**Material Properties**

Concrete strength	$f'_c =$	35 MPa
Concrete density	$\rho_c =$	2400 kg/m <sup>3</sup>
Young's Modulus (Concrete)	$E =$	27 GPa
Assume Poisson's ratio	$\nu_c =$	0.15

**Foundation Details**

Shear modulus	$G_s =$	220 MPa
Assume Poisson's ratio	$\nu_s =$	0.33
Mass density of soil	$\rho_s =$	1800 kg/m <sup>3</sup>
Elastic (Young's) modulus	$E_s =$	585 MPa
Shear Wave Velocity	$v_s =$	350 m/s

**Seismic Masses****Liquid Mass**

Product: Water	Packaging Group	Schedule 4
Specific Gravity	$SG =$	1
Unit weight of product	$\gamma_l =$	9.81 kN/m <sup>3</sup>
Unit mass of product		1000 kg/m <sup>3</sup>
Total product mass	$m_l =$	53790 kg

Assume tank is rigid on a flexible foundation.

H/R Ratio	$H/R =$	0.71
From Fig C3.8	$m_1/m_l =$	0.57
From Fig C3.17	$m_f/m_l =$	0 $\therefore$ tank can be considered rigid
	$m_0/m_l =$	0.43
Impulsive mass for rigid tank	$m_0 =$	23130 kg
Equivalent mass for first convective mode	$m_1 =$	30660 kg

**Wall Mass**

$$m_w = 10014 \text{ kg}$$

**Roof Mass**

$$m_t = 8482 \text{ kg}$$

**Base Mass**

$$m_b = 8482 \text{ kg}$$

**Heights of Masses**

From Fig C3.9

$$h_0/H = 0.4$$

$$h_1/H = 0.56$$

Height of rigid impulsive mass above base

$$h_0 = 0.82 \text{ m}$$

Height of first convective mode above base

$$h_1 = 1.148 \text{ m}$$

**Periods****Impulsive mode**

Calculated in accordance with C3.6

Impulsive mass for rigid body mode

$$m_r = 41626 \text{ kg}$$

Initially guess:

$$\alpha_x = 1.0$$

$$\alpha_\theta = 0.75$$

Horizontal translation stiffness of foundation

$$K_x = 3.16\text{E}+09 \text{ N/m}$$

Rocking stiffness of foundation

$$K_\theta = 1.77\text{E}+10 \text{ Nm}$$

Period of impulsive mode

$$\check{T}_0 = 0.03 \text{ s}$$

Check  $\alpha_x$  and  $\alpha_\theta$ 

Using Equation C3.31

$$a = 2.06$$

From Fig C3.24

$$\alpha_x = 0.95$$

$$\alpha_\theta = 0.6$$

For new values of  $\alpha_x$  and  $\alpha_\theta$ 

$$K_x = 3.00\text{E}+09 \text{ N/m}$$

$$K_\theta = 1.42\text{E}+10 \text{ Nm}$$

Period of impulsive mode

$$\check{T}_0 = 0.03 \text{ s}$$

**Convective mode**

From Fig C3.20

$$T_1\sqrt{g/R} = 5.00$$

$$T_2\sqrt{g/R} = 2.70$$

Period of first convective mode

$$T_{c1} = 2.71 \text{ s}$$

Period of second convective mode

$$T_{c2} = 1.47 \text{ s}$$

**Vertical mode**

Mass of liquid and tank

$$m_v = 80769 \text{ kg}$$

Initially guess:

$$\alpha_v = 0.75$$

Vertical stiffness of foundation

$$K_v = 2.96\text{E}+09 \text{ N/m}$$

Period of first vertical mode

$$\check{T}_b = 0.03 \text{ s}$$

Check  $\alpha_v$

Using Equation C3.31

From Fig C3.24

For new value of  $\alpha_v$

Period of first vertical mode

$$\begin{aligned} a &= 1.64 \\ \alpha_v &= 0.55 \\ K_v &= 2.17 \times 10^9 \text{ N/m} \\ \ddot{T}_b &= 0.04 \text{ s} \end{aligned}$$

### Earthquake Seismic Coefficients

From Figure 3.1(d)

From Figure 3.2(c)

$$\begin{aligned} \mu &= 1.25 \text{ ULS} \\ &1.00 \text{ SLS1 and SLS2} \\ \xi &= 12.00\% \text{ Impulsive mode (ULS and SLS1)} \\ &0.50\% \text{ Impulsive mode (SLS2)} \\ &0.50\% \text{ Convective mode} \\ &13.00\% \text{ Vertical mode} \end{aligned}$$

$$S_p = 1.0 \text{ for ULS and SLS1 and SLS2}$$

$$\begin{aligned} k_f(\mu, \xi) &= 0.634 \text{ ULS and SLS1 Impulsive mode} \\ &1.67 \text{ ULS and SLS1 Convective mode} \\ &0.651 \text{ ULS and SLS1 Vertical mode} \\ &1.67 \text{ SLS2} \end{aligned}$$

Importance Level 4

Subsoil Class C

$$\begin{aligned} Z &= 0.40 \\ R &= 1.80 \text{ ULS} \\ &1.00 \text{ SLS2} \\ &0.25 \text{ SLS1} \\ N(T_i, D) &= 1.00 \end{aligned}$$

### Impulsive Coefficient

From NZS1170.5:2004 for

$$\begin{aligned} \ddot{T}_0 &= 0.03 \text{ s} \\ C_h(T_0) &= 2.36 \\ C(T_0) &= 1.70 \text{ ULS} \\ &0.94 \text{ SLS2} \\ &0.24 \text{ SLS1} \\ C_d(T_0) &= 1.33 \text{ ULS} \\ &0.60 \text{ SLS2} \\ &0.39 \text{ SLS1} \end{aligned}$$

### Convective Coefficient

From NZS1170.5:2004 for

$$\begin{aligned} T_{c1} &= 2.71 \text{ s} \\ T_{c2} &= 1.47 \text{ s} \\ C_h(T_{c1}) &= 0.49 \\ C_h(T_{c2}) &= 0.90 \\ C(T_{c1}) &= 0.35 \text{ ULS} \\ &0.20 \text{ SLS2} \\ &0.05 \text{ SLS1} \\ C(T_{c2}) &= 0.65 \text{ ULS} \\ &0.36 \text{ SLS2} \end{aligned}$$

	0.09 SLS1
$C_d(T_{c1}) =$	0.59 ULS
	0.33 SLS2
	0.08 SLS1
$C_d(T_{c2}) =$	1.08 ULS
	0.60 SLS2
	0.15 SLS1

### Vertical Coefficient

From NZS1170.5:2004 for

$\ddot{T}_b =$	0.04 s
$0.7C_h(T_b) =$	1.36
$C(T_b) =$	0.98 ULS
	0.54 SLS2
	0.14 SLS1
$C_d(T_b) =$	0.93 ULS
	0.35 SLS2
	0.23 SLS1

### **Base Shear**

#### Impulsive shear

Rigid impulsive mode	$V_r =$	302 kN
Flexible impulsive mode	$V_f =$	241 kN
Total impulsive mode	$V_i =$	543 kN

#### Convective shear

First convective mode	$V_1 =$	178 kN
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#### Total shear

$V_T =$	571 kN
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### **Base Moment**

#### Impulsive moment

Rigid impulsive mode	$M'_r =$	285 kNm
Flexible impulsive mode	$M'_f =$	398 kNm
Total impulsive mode	$M_i =$	683 kNm

#### Convective moment

First convective mode	$M'_1 =$	226 kNm
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#### Total moment

$M_{OT} =$	720 kNm
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Restoring moment

$$M_R = 794 \text{ kNm}$$

Since  $M_{OT} < M_R$ , overturning will not occur

Seismic Forces in Wall**Hoop Force**Hydrostatic

Hydrostatic pressure

$$p_h = 20.1 \text{ kPa}$$

From Fig A-3 at 0.1H

$$N_{\theta nh} = 0.48$$

Using Equation A.5

$$N_{\theta h} = 27.9 \text{ kN/m}$$

Hydrostatic hoop stress

$$f_{hh} = 254 \text{ kPa}$$

Impulsive

From Fig C3.3

$$q_0 = 0.55$$

Using Equation A.4, Impulsive pressure

$$p_i = 20.7 \text{ kPa}$$

From Fig A-8

$$N_{\theta ni} = 0.575$$

Using Equation A.8

$$N_{\theta i} = 34.5 \text{ kN/m}$$

Impulsive hoop stress

$$f_{hi} = 313 \text{ kPa}$$

Convective

Using Equation A.3, Convective pressure

$$p_1 = 14.0 \text{ kPa}$$

From Fig A-13

$$N_{\theta n1} = 0.425$$

Using Equation A.6

$$N_{\theta 1} = 17.2 \text{ kN/m}$$

Convective hoop stress

$$f_{h1} = 157 \text{ kPa}$$

Vertical

Vertical pressure

$$p_v = 18.7 \text{ kPa}$$

Using Equation A.9

$$N_{\theta v} = 26.0 \text{ kN/m}$$

Vertical hoop stress

$$f_{hv} = 236 \text{ kPa}$$

**Vertical Bending Moments**Hydrostatic

From Fig A-3 at 0.1H

$$M_{znh} = 0.1$$

$$M_{zh} = 0.64 \text{ kNm/m}$$

Impulsive

From Fig A-8 at 0.1H

$$M_{zni} = 0.1$$

$$M_{zi} = 0.66 \text{ kNm/m}$$

Convective

From Fig A-13 at 0.1H

$$M_{zn1} = 0.0425$$

$$M_{z1} = 0.19 \text{ kNm/m}$$

Vertical

$$M_{zv} = 0.60 \text{ kNm/m}$$

**Check Stress/Force Levels**

Assume tank is prevented from uplifting.

Max tension from combined hoop stress	$f_{\max} =$	676 kPa
Max tension force	$N_{\max} =$	127.4 kN/m

Max combined bending stress	$M_{b,\max} =$	1.55 kNm/m
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Tank does uplift so check radial membrane stress in base.

Use iterative process from Section C4.4.2

Guess	$r =$	2.81 m
Therefore	$\tau =$	0.97
Using Equation C4.19	$\theta^* =$	1.54
Using Equation C4.20	$k =$	0.82
Using Equation C4.21	$M_R =$	540 kNm
	$=$	$M_{OT}$ , OK

Using Equation C4.24	$\bar{E} =$	27.2 GPa
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Thus	$f_{rb} =$	1.47 MPa
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Bending moment in slab	$f_{rb} \times Z_{slab} =$	3.8 kNm/m
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**Roof Pressure**

Say 0.1m of freeboard available (overflow pipe diameter)

From Fig C3.26	$d_{\max} =$	1.45 m
	Freeboard =	0.18 m
	$h_r =$	1.28 m
	$c_s =$	4

	$\bar{u} =$	3.36 m/s
--	-------------	----------

	$p_b =$	12.5 kPa
--	---------	----------

	$p_r =$	35.1 kPa
--	---------	----------

Length of water contact	$L_{wc} =$	2.9 m
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Max shear force on roof to wall connection	$V_{rw} =$	33.8 kN/m
--	------------	-----------

Moment in roof from buoyancy pressure	$M_{roof} =$	13.02 kNm/m
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Earthquake Force Wall Design

Design Parameters

Water level above base slab, Hw  
2.05 m  
Water pressure at base, Pw  
20.1 kPa  
Reservoir Diameter, D  
5.78 m  
Hoop tension force at base  
58.1 kN

Height	Hydrostatic		Vertical		Impulsive		Convective		Total Seismic	Total Force
	N0hn	N0h = N0h1	Nv=Cd(Tv)	N0i	N0in	N0i	N01n	N01=N01nf	(Ne)	(N0h + Ne)
Top	0	0.00	0.00	3.00	0.05	3.00	1	40.53	40.64	40.64
0.9H	0.1	5.81	2.06	14.98	0.25	14.98	0.95	38.50	41.37	47.18
0.8H	0.225	13.08	4.63	26.97	0.45	26.97	0.95	38.50	47.24	60.31
0.7H	0.35	20.34	7.21	35.96	0.6	35.96	0.9	36.48	51.73	72.07
0.6H	0.475	27.61	9.78	44.95	0.75	44.95	0.85	34.45	57.47	85.08
0.5H	0.575	33.42	11.84	50.94	0.85	50.94	0.85	34.45	62.63	96.05
0.4H	0.65	37.78	13.38	59.93	1	59.93	0.75	30.40	68.52	106.30
0.3H	0.725	42.14	14.93	65.93	1.1	65.93	0.65	26.34	72.55	114.69
0.2H	1	58.12	20.59	62.93	1.05	62.93	0.5	20.26	69.25	127.37
0.1H	0.8	46.50	16.47	44.95	0.75	44.95	0.3	12.16	49.39	95.89
Base	0	0.00	0.00	0.00	0	0.00	0	0.00	0.00	0.00

Note: Units of N0 = Mpa



From spreadsheet, page 7;

hydrostatic hoop stress:

$$\begin{aligned} f_{hh} = f_h &= \frac{58.1 \text{ kN/m}}{816.8 \text{ mm}^2/\text{m}} \quad (\text{SE82 mesh} + \text{D12s at } 200\text{c/c}) \\ &= \underline{\underline{71.1 \text{ MPa}}} \end{aligned}$$

Vertical moment stress:

$$\text{Moment} = 0.64 \text{ kNm/m} \quad (\text{page 5})$$

Converting to stress:

$$E_c = 26541 \text{ MPa}$$

$$E_s = 200000 \text{ MPa}$$

$$\begin{aligned} A_s &= \text{SE82 mesh} + \text{D10's at } 300\text{c/c} \\ &= 513 \text{ mm}^2/\text{m} \end{aligned}$$

Steel assumed to be centrally placed

$$\therefore C = T$$

$$\therefore A_c f_c = n A_s f_s$$

$$\begin{aligned} n = \frac{E_s}{E_c} &= \frac{200000}{26541} \\ &= 7.5 \end{aligned}$$



Stress is linearly related to distance from neutral axis.

$$\therefore f_c \propto \frac{y}{2}, \quad f_s \propto 55 - y$$

$$A_c \times \frac{y}{2} = n A_s \times (55 - y)$$

$$1000 \times y \times \frac{y}{2} = 7.5 \times 513 \times (55 - y)$$

$$500y^2 = 211613 - 3848y$$

$$y = 17.1 \text{ mm} < C_{\max} = 22 \text{ mm ok!}$$

$$jd = 55 - \frac{17.1}{3}$$

$$= 49.3 \text{ mm}$$

$$f_s = \frac{M}{jd A_s}$$

$$= \frac{0.64}{49.3 \times 513}$$

$$f_v = \underline{\underline{25.3 \text{ MPa}}}$$

Earthquake hoop stress:

$$f_{eh} = \frac{72.55 \text{ kN/m}}{644 \text{ mm}^2/\text{m}} \quad (\text{page 7})$$

$$= \underline{112.7 \text{ MPa}}$$

Earthquake vertical stress

$$M = \sqrt{0.66^2 + 0.19^2 + 0.6^2} \quad (\text{page 5})$$

$$= 0.61 \text{ kNm/m}$$

$$f_{ev} = \frac{M}{A_{sjd}}$$

$$= \frac{0.61}{513 \times 49.3}$$

$$f_{ev} = \underline{24.1 \text{ MPa}}$$

## Temperature Stress

$$S_{FACT} = \frac{H^2}{2\alpha t}$$

$$H = 2.05m$$

$$\alpha = 3.0m$$

$$t = 110mm$$

$$S_{FACT} = \frac{2.05^2}{2 \times 3 \times 11}$$

$$= 6.4$$

$$C = .597$$

(vertical)

$$.711$$

(hoop inside)

$$-0.5$$

(hoop outside)

$$E_c = 26541 MPa$$

$$\alpha_c = 11 \times 10^{-6}$$

$$\theta = 30^\circ$$

$$f_{vert} = .597 \times 26541 \times 11 \times 10^{-6} \times 30$$

$$= 5.2 MPa$$

$$M_t = \frac{5.2}{2} \times \frac{110^3}{6}$$

$$= 5.24 kNm/m$$

$$E_c = 26541 \text{ MPa}$$

$$E_s = 20000 \text{ MPa}$$

$$A_s = 512 \text{ mm}^2/\text{m}$$

$$n = 7.5$$

$$n A_s = 3846 \text{ mm}^2$$

$$f_c A_c = f_s A_s$$

$$1000y \times \frac{y}{2} = 3846 \times (55 - y)$$

$$500y^2 = 211530 - 3846y$$

$$y = 17.1 \text{ mm}$$

$$jd = 55 - \frac{17.1}{3}$$

$$= 49.3 \text{ mm}$$

$$f_{tr} = f_s = \frac{5.24}{512 \times 49.3}$$

$$f_{tr} = \underline{\underline{208 \text{ MPa}}}$$

Hoop stress

$$C_{out} = -0.5$$

$$C_{in} = +711 \rightarrow \text{Critical}$$

$$f_{hoop} = 0.711 \times 26541 \times 11 \times 10^{-6} \times 30$$

$$= 6.22 \text{ MPa}$$

$$M = \frac{6.22}{2} \times \frac{110^2}{6}$$

$$= 6.3 \text{ kNm/m}$$

$$A_s = 775 \text{ mm}^2$$

$$nA_s = 5813 \text{ mm}^2$$

$$f_c A_c = f_s A_s$$

$$500y^2 = 319715 - 5813y$$

$$y = 20.1 \text{ mm} < 22 \text{ mm} \quad \text{OK}$$

$$jcd = 55 - \frac{20.1}{3}$$

$$= 48.3 \text{ mm}$$

$$f_{TH} = f_s = \frac{6.3}{775 \times 48.3}$$

$$f_{TH} = 168 \text{ MPa}$$





Refer NZS3106, cl. C4.2.4

Swelling stress to be derived from  
Thermal equivalent,  $T$ :

$$T = \frac{E_{sw}}{\alpha_c}$$

$$E_{sw} = 281 \times 10^{-6} \quad (\text{Table 2})$$

$$\alpha_c = 11 \times 10^{-6} \quad (\text{Table C4.2.3})$$

$$T = 25.5^\circ$$

$$\frac{\text{Temp. stress}}{30} \times 25.5 = \text{Swelling Stress}$$

$$\therefore \text{Swelling stress, vert.} = \frac{208}{30} \times 25.5$$

$$f_{swv} = 176.8 \text{ MPa}$$

$$\text{Swelling stress, hoop} = \frac{169}{30} \times 25.5$$

$$f_{swh} = 142.8 \text{ MPa}$$



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JOB:

Wellington Water

SUBJECT:

Shrinkage

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$$\epsilon_{sh} = 66 \times 10^{-6}$$

(Table 2)

$$\therefore \text{Shrinkage stress, vert} = \frac{176.8}{281} \times 66$$

$$f_{shv} = 41.5 \text{ MPa}$$

$$\therefore \text{Shrinkage stress, hoop} = \frac{142.8}{281} \times 66$$

$$f_{shh} = 33.5 \text{ MPa}$$



Strain induced stresses are to be reduced  
by factor  $R_F$ :  
Refer NZS3106 Figure 3

$$\rho = \frac{A_s}{bd}$$

$$\rho_{hoop} = \frac{775}{110 \times 1000}$$

$$= 0.007$$

$$\rho_{vert} = \frac{512}{110 \times 1000}$$

$$= 0.0047$$

$$R_{F,hoop} = 0.18$$

$$R_{F,vert} = 0.15$$

$$f_{TV} = 0.15 \times 208$$
$$= 31.2 \text{ MPa}$$

$$f_{TH} = 0.18 \times 168$$
$$= 30.2 \text{ MPa}$$

$$f_{swv} = 0.15 \times 176.8$$
$$= 26.5 \text{ MPa}$$

$$f_{swh} = 0.18 \times 142.8$$
$$= 25.7 \text{ MPa}$$





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$$f_{shv} = 0.15 \times 41.5 \\ = 6.2 \text{ MPa}$$

$$f_{shh} = 0.18 \times 33.5 \\ = 6.0 \text{ MPa}$$

## Stress Summary:

Liquid pressure, hoop:

$$f_{lh} = 71.1 \text{ MPa}$$

Liquid pressure, vert:

$$f_{lv} = 25.3 \text{ MPa}$$

Earthquake, hoop:

$$f_{eh} = 112.7 \text{ MPa}$$

Earthquake, vertical:

$$f_{ev} = 24.1 \text{ MPa}$$

Temperature, hoop:

$$f_{th} = 30.2 \text{ MPa}$$

Temperature, vertical:

$$f_{tv} = 31.2 \text{ MPa}$$

Swelling, hoop:

$$f_{sh} = 25.7 \text{ MPa}$$

Swelling, vertical:

$$f_{sv} = 26.5 \text{ MPa}$$



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Shrinkage, hoop:

$$f_{shh} = 6.0 \text{ MPa}$$

Shrinkage, vertical:

$$f_{shr} = 6.2 \text{ MPa}$$



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JOB:

Wellington Water

SUBJECT:

Load Combinations

BY:

GJ

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Critical load combinations for SLS, in wall

$$A_1: F_{sh} \text{ or } 0.5F_{sw}$$

$$A_2: F_{lp} + 0.5F_{sw}$$

$$B_5: F_{lp} + F_T + 0.7F_{sw}$$

$$B_6: F_{lp} + (E_{s1} \text{ or } E_{s2})$$

$$B_7: F_T + (0.7F_{sh} \text{ or } 0.35F_{sw})$$



Load case A2 critical by observation  
for group A load cases.

Crack widths to be restricted to  
0.2mm

$$A2: F_{lp} + 0.5 F_{sw}$$

$$\text{Vert: } 25.3 + 0.5 \times 26.5 \\ = 38.6 \text{ MPa}$$

$$\text{Hoop: } 71.1 \text{ MPa} + 0.5 \times 25.7 \\ = 84.0 \text{ MPa}$$

Crack width,  $w$ :

$$w = 2.0 \beta_1 \frac{f_s}{E_s} g_s$$

$$\beta_1 = \frac{y - kd}{d - kd}$$

$$y = 110 \text{ mm}$$

$$kd = 20.1$$

$$d = 55 \text{ mm}$$

$$\beta_1 = \frac{110 - 20.1}{55 - 20.1} \\ = 2.57$$

$$E_s = 200 \text{ MPa}$$

$$g_s = \sqrt{55^2 + 100^2} = 114 \text{ mm}$$



$$w = 2.0 \times 2.57 \times \frac{84}{200} \times 114$$
$$= 0.209 \text{ mm} \approx 0.2 \text{ mm}$$

Within 5%

Loads assumed to be maximum at same location, which is conservative, therefore crack width is adequate.

Vertical crack width ok by observation

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SUBJECT: Group B Loads

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Group B load cases to restrict steel stresses to 240MPa

Critical cases by observation:

- B5

- B6

$$\begin{aligned}
 B5: & F_{ip} + F_T + 0.7 F_{sw} \\
 & = 71.7 + 30.2 + 0.7 \times 25.7 \quad (\text{Hoop}) \\
 & = 112 \text{MPa} < 240 \text{MPa} \quad \text{ok!}
 \end{aligned}$$

Vertical direction ok by observation

$$\begin{aligned}
 B6: & F_{ip} + F_{sl} \\
 & = 71.7 + 112.7 \quad (\text{Using ULS EQ}) \\
 & = 184.4 \text{MPa} < 240 \text{MPa} \quad \text{ok!}
 \end{aligned}$$



## Checking tank reinforcing.

Refer NZS 3106.

Minimum hoop reinforcing:

$$\begin{aligned} \rho_{\min} &= 0.05 f'_c / f_y \\ &= \frac{0.05 \times 35}{300} \\ &= 642 \text{ mm}^2/\text{m} \quad (\text{for } 110 \text{ mm thick wall}) \end{aligned}$$

Wall reinforcing consists of SE82 mesh, as well as D10/D12 hoops at 150/200 mm crs.

$$A_s = \left( \frac{8^2 \times \pi}{4} \times \frac{1000}{200} \right) + \left( \frac{12^2 \times \pi}{4} \times \frac{1000}{200} \right)$$

$$= 817 \text{ mm}^2 > \rho_{\min} \quad \text{ok!}$$

Flexural reinforcement:

$$\begin{aligned} \rho_{p,\min} &= \frac{\sqrt{f'_c}}{4 f_y} \\ &= \frac{\sqrt{35}}{4 \times 500} \\ &= 325 \text{ mm}^2/\text{m} \end{aligned}$$



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As of S582 mesh:  $251\text{mm}^2/\text{m} < 325\text{mm}^2/\text{m}$  NG6

Additional reinforcing required in potential plastic hinge zone. Extend D10 starters from base to provide additional reinforcement.

As of D10's at 300mm centres:

$$= \frac{10^2 \times \pi}{4} \times \frac{1000}{300}$$

$$= 261\text{mm}^2$$

$$\therefore \Sigma A_s = 512\text{mm}^2/\text{m} < 325\text{mm}^2/\text{m} \text{ ok!}$$

Minimum reinforcement ratio for early-age thermal cracks:

Concrete shrinkage restrained in horizontal direction only, check ratio for horizontal direction

$$\rho_{\min} = \frac{f_{ct.3}}{f_y}$$

$$= \frac{1.92}{300}$$

$$= 704\text{mm}^2/\text{m}$$

$$A_s = 817\text{mm}^2/\text{m} \text{ so ok! (From pg. 1)}$$

From spreadsheet, the negative pressure on the tank roof, due to sloshing in an ULS earthquake:

$$M_{\text{roof}}: 13.0 \text{ kNm}$$

Moment to be countered by selfweight of tank roof.

$$l = 3.0 \text{ m}$$

$$t = 125 \text{ mm}$$

$$\gamma = 24 \text{ kN/m}^3$$

$$M_G = \frac{24 \times 0.125 \times 1 \times 3^2}{8}$$

$$= 3.375 \text{ kNm/m}$$

$$\therefore M^* = 13.0 - 3.38$$

$$= 9.6 \text{ kNm}$$

Moment capacity of tank roof,  $\phi M_n$ :

$$\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right)$$

$$a = \frac{A_s f_y}{\alpha_1 b_w f_c'}$$

Try 2 x SE82 mesh centrally placed

$$A_s = 502 \text{ mm}^2$$

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$$a = \frac{502 \times 500}{0.85 \times 1000 \times 30}$$

$$= 9.8$$

$$\phi M_n = 0.85 \times 502 \times 500 \times \left( 62.5 - \frac{9.8}{2} \right)$$
$$= 12.3 \text{ kNm} > 9.6 \text{ kNm}$$

Adopt 30MPa, 125mm thick concrete  
roof with 2x SE82 mesh, centrally placed



Checking steel stress in tank base, due to tank uplift.

$$M_{fb} = 3.8 \text{ kN/m}$$

(from spreadsheet)

$$E_c = 26541 \text{ MPa}$$

$$E_s = 200000 \text{ MPa}$$

$$A_s = 502 \text{ mm}^2/\text{m}$$

(2 rows of S18 mesh)

$$n = \frac{E_s}{E_c} = \frac{200600}{26541} \\ = 7.5$$

$$n A_s = 3848 \text{ mm}^2$$

$$C = T$$

$$\therefore A_c f_c = n A_s f_s$$

Stress is linearly related to distance from neutral axis.

$$\therefore f_c \propto \frac{y}{2}, f_s \propto 55 - y$$



$$1000 \times y \times \frac{y}{2} = 3765 (62.5 - y)$$

$$500y^2 = 235312.5 - 3765y$$

$$y = 18.3\text{mm} < C_{\text{max}} = 25\text{mm} \quad \text{ok!}$$

$$\begin{aligned} jd &= 62.5 - \frac{18.3}{2} \\ &= 53.4\text{mm} \end{aligned}$$

$$f_s = \frac{3.8}{53.4 \times 502}$$

$$= 141.7\text{MPa} < 240\text{MPa} \quad \text{ok!}$$

Adopt 125mm base with two rows of  
SE82 mesh, centrally placed

Refer spreadsheet,  $V_T = 571 \text{ kN}$

Effective tank radius =  $2.8 \text{ m}$

Effective area,  $A' = 2.8^2 \times \pi$   
 $= 24.8 \text{ m}^2$

Sliding resistance =  $\phi_{SL} \times A' \times S_u$   
 $= 0.8 \times 24.8 \times 100$   
 $= 1984 \text{ kN} > 571 \text{ kN} \text{ ok!}$