Upper Hutt City Council Water Reservoirs
Seismic Assessment

OPUS

Report for Capacity Infrastructure Services
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Upper Hutt City Council Water Reservoirs – Seismic Assessment

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1 Introduction

Capacity Infrastructure Services, on behalf of Upper Hutt City Council, has engaged Opus International Consultants to assess the seismic performance of the Upper Hutt reservoirs to identify the reservoirs which will retain water for emergency supply after a major earthquake, and to identify any that may require strengthening.

The reservoirs that were included in the study are shown in Table 1. They are all circular concrete tanks located above ground.

Table 1: Reservoir details

Reservoir	Storage (m^3)	Year Built	Construction Type
Trentham 2	9100	1984	Precast, post tensioned walls with insitu joints and a dome roof.
Trentham 1	2270	1948	Insitu reinforced concrete with a grid of columns supporting the roof beams.

2 Inspections

All of the reservoirs were inspected on 9 February 2010 by Robert Davey (Opus structural engineer), Ian McDonald (Opus structural engineer), Pathmanathan Brabhaharan (Opus geotechnical engineer), Keith Woolley (Capacity project manager) and John Baines (Capacity engineer). The purpose of the inspections was to confirm that the available drawings are consistent with the as built construction; assess the tank condition, and identify any evidence of cracking, leakage, spalling or corrosion that that might impact on the tank performance, or indicate weaknesses; and assess the stability of the reservoir site and potential vulnerability to ground damage under earthquake.

The insides of the reservoirs were not inspected.

All of the reservoirs appeared to be in good condition with no evidence of significant leaking that would indicate structural defects.

3 Geotechnical Evaluation

The focus of the geotechnical inspection was to assess the stability of the reservoir sites and the potential for earthquake induced slope failure to affect the reservoir foundations or cause material to slump onto the sides of the tanks.

Trentham 1: Slopes moderately steep and unlikely to affect tank.

Trentham 2: Gully slope on one side of tank, but failure unlikely to undermine tank, because of narrow gully.

4 As Built Data and Material Properties

Original construction drawings were available for most of the tanks and these provided the data required for the seismic analyses. There is some uncertainty about the construction of Plateau. We have assumed that it is the Manacon type.

The seismic performance of the tanks is very sensitive to the yield strength of the hoop reinforcement in particular. Wire strengths for post tensioning cables have been taken from BS 5896:1980 [2], using the 0.1% proof load as the yield strength. Mild steel reinforcing steel strengths have been taken from the drawings where available and based on the New Zealand Society for Earthquake Engineering guidelines for assessing the seismic performance of existing structures [3] where the steel strength is not given.

In cases where the concrete compressive strength is not known 40 MPa has been adopted for the newer reservoirs and 30 MPa for the older reservoirs in these analyses. It should be noted however that concrete strength is not a very significant factor in the seismic performance of circular tanks.

5 Seismic Loads

Seismic loads have been calculated from the current design code requirements for new reservoirs.

Procedures for calculating earthquake loads for the design of new tanks are specified in the recently published standard NZS 3106:2009 "Design of concrete structures for the storage of liquid" [1]. This standard refers to the earthquake loadings standard NZS 1170.5 [5] for the seismic zone hazard factors; the return period factor for the relevant limit state and tank importance level and design life; and the spectral shape factor for the site subsoil type and response mode.

The current NZS 1170.5 seismic zone factor for Upper Hutt (0.42) is based on the Institute of Geological and Nuclear Science's 2000 National Seismic Hazard Model (NSHM). The NSHM has been updated since NZS 1170.5 was published [6], however we understand that the zone factor for the Upper Hutt region has not changed significantly with this updated model.

The seismic forces specified in the reservoir standard are a function of the Importance Level (IL) of the facility as specified in AS/NZS 1170.0:2002 [4]. Facilities with special post-disaster function are classified as IL4, and their design loads are based on the following return periods:

Serviceability Limit State (operational continuity):	500 years
Ultimate Limit State (collapse avoidance):	2500 years.

In other words the design standards require operational continuity for earthquake shaking that has an average recurrence interval of 500 year or less at the site. We interpret this to mean that tanks should retain their contents following the Serviceability Limit State (SLS) level of shaking, while limited cracking and leakage is permitted following the Ultimate Limit State (ULS) loading so long as the tank is not so seriously damaged that its stability is affected.

The 500 year loading is similar to that expected from the 50%ile Wellington Fault scenario load. For this study we have also assessed operational continuity following the ULS (2500 year) level of shaking, which is similar to the loading expected from the 84%ile Wellington Fault scenario loads. The ULS loads used in the study are therefore based on a ductility factor μ=1.0, i.e. elastic response, rather than μ=1.25 (i.e. non-linear response) as is permitted by NZS 3106:2009.

The earthquake loading is influenced by the foundation soil stiffness characteristics. We have assessed the soil class at all sites to be Class B, weak rock.

6 Seismic Assessment Methodology

6.1 General

The procedures specified in NZS 3106:2009 have been used to analyse the response of the tanks to the seismic loads.

The following aspects have been investigated:

1) Hoop Response
2) Vertical reinforcing steel content
3) Wall to foundation connection
4) Foundation to floor connection
5) Roof to wall connection

6) Sloshing wave heights and clearance.

6.2 Hoop Response

The hydrostatic and hydrodynamic (seismic) water pressures in circular tanks are resisted primarily by hoop forces in the tank walls. Prestressed or mild steel hoop reinforcement is provided to control vertical hoop cracking under hydrostatic pressure. When earthquake shaking occurs the resulting hydrodynamic pressures will increase the static water pressures and hence the hoop forces. When the hoop forces exceed the pretension forces (if prestressed) the concrete will crack. If the steel does not yield the cracks will close up as the pressures reduce. If the steel yields, the cracks will not close fully, as the net hydrostatic and hydrodynamic pressures on the wall will never be sufficient to force compression yielding of the reinforcement. Repeat cycles of outward hydrodynamic pressures causing yield will progressively increase the permanent crack widths.

For operational continuity (i.e. minimal leakage) following an earthquake, the hoop reinforcement should not be stressed beyond its yield point. It should also be noted that if prestressed reinforcement is yielded, its prestressing capability will be reduced or totally negated. This means that the serviceability performance of prestressed tanks could not be restored post-earthquake by simply grouting the cracks.

The permanent crack widths are a function mainly of the nonlinear (post-yield strain) hoop displacement response and the crack spacing. The amplitude of nonlinear displacement, and consequently the crack widths, will vary around the perimeter of the tank, with the largest displacements on the axes of maximum ground shaking. It will also vary up the height of the tank with the largest displacements generally occurring at a third to mid-height of the tank. More frequent, closer spaced cracks will result in smaller crack widths.

A procedure for estimating maximum crack spacing is given in NZS 3106. In precast, prestressed concrete tanks, the cracks will initially occur at the precast wall panel joint locations, and it is possible that a large proportion of the hoop displacement will be absorbed by these cracks, which could potentially be protected by joint sealants. However we cannot exclude the possibility that cracks will also form between these joints.

There are no simple methods available to estimate crack widths that result from hoop yield strains. Nonlinear time history analyses would provide realistic assessments of the hoop displacements from which crack widths could be estimated. However these analyses are beyond the scope of this study.

We have used an approximate nonlinear displacement response method to estimate the hoop displacement response of Trentham 1 which is a cast insitu reinforced concrete tank and expected to be loaded beyond its yield point in the SLS earthquake. From these results we have estimated the permanent crack widths to be approximately 6mm wide.

It is emphasised that these are ball-park estimates only and could vary - 50%, +100%. However it is clear that crack widths of this size will result in rapid emptying of the tank.

For this study we have adopted the performance scale shown in Table 2, which is based on the hoop force ratio.

$$\text{Hoop Force Ratio} = \frac{\text{Hoop Strength}}{\text{Earthquake Force}}$$

Where:

Hoop Strength = the reliable hoop yield strength of the tank wall.
Earthquake forces = the hoop force in the tank due to combine hydrostatic and SLS or ULS hydrodynamic load.

Table 2: Hoop response performance scale

Hoop Force Ratio		Performance Class	Performance Standard
SLS	ULS		
$\geq$1.0	$\geq$1.0	A	Exceeds current minimum design standards for IL 4 reservoirs. Contents retained following 84%ile Wellington Fault Earthquake.
$\geq$1.0	<1.0$\geq$0.85	B	Complies with current minimum design standards for IL 4 reservoirs. Contents retained following 50%ile Wellington Fault Earthquake.
$\geq$1.0	<0.85	C	Complies with current minimum SLS, but not ULS, design standards for IL 4 reservoirs. Contents retained following 50%ile Wellington Fault Earthquake.
<1.0	<0.85	D	Does not comply with current design standards for IL4 reservoirs. Loss of contents probable following Wellington Fault Earthquake.

Tanks will also be classified D if they have critical weaknesses listed below.

6.3 Vertical Reinforcing Steel Content

The vertical reinforcement is required primarily to control cracking from vertical bending imposed on the walls by radial displacements from the hoop response. The code gives guidance on the minimum ratio of reinforcement required in a potential plastic hinge zone. This is to assist the wall in distributed cracking rather than one larger crack appearing.

Note however that this is only a problem if the hoop response displacements could be large, i.e. Hoop Force Ratio < 0.8.

6.4 Wall to Foundation Check

Failure of the connection between the wall and the foundation would cause considerable leakage at the bottom of the tank wall.

6.5 Foundation to Floor Check

If there is no structural connection between the foundation and the floor there is a risk that the joint could open causing the contents to leak into the ground. This defect is common in tanks that were designed before the 1986 version of NZS 3106 was published.

6.6 Roof to Wall Connection Check

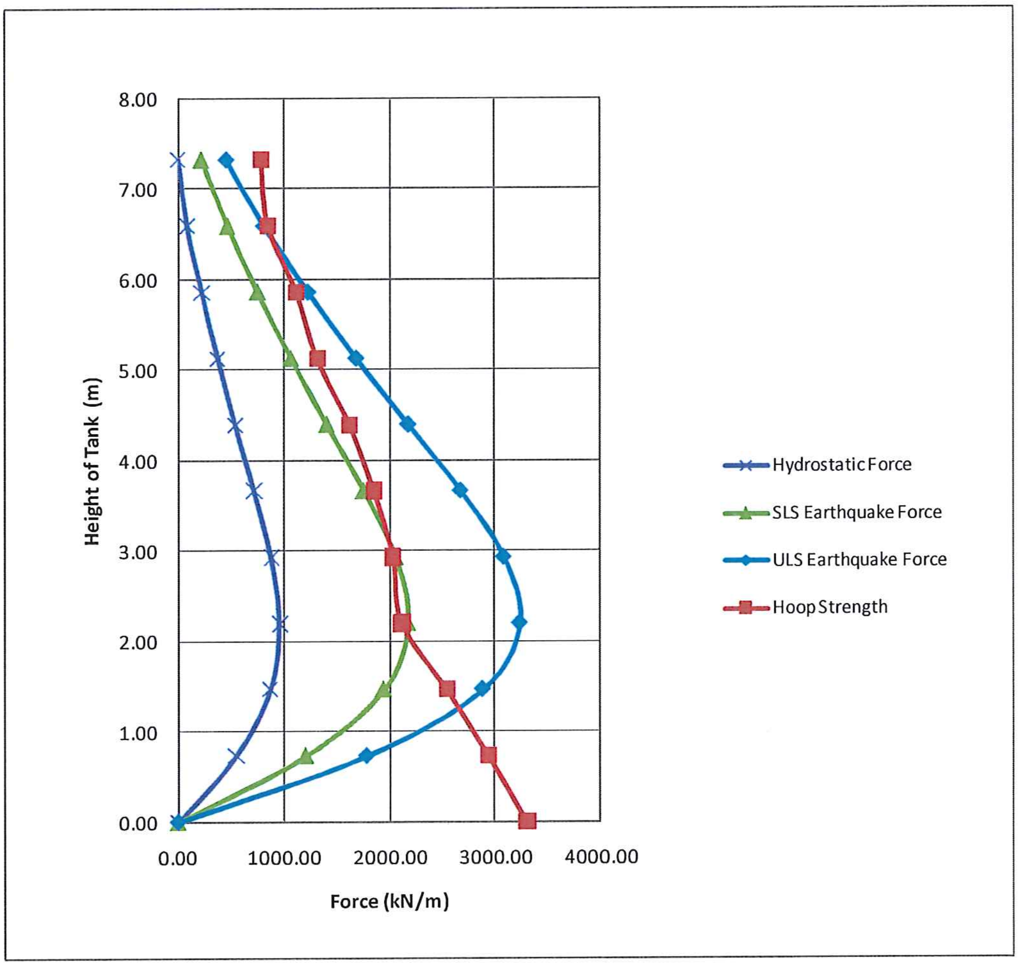
Seismic forces on reservoir roofs are usually resisted by a nib that engages with the wall. The adequacy of this connection is assessed.

6.7 Wave Heights and Clearance

The convective wave heights have been estimated for the SLS (500 year) loading and compared to the clearance from top water level to the underside of the roof. If the there is insufficient clearance uplift pressures on the roof may cause it to lift off its supports and be damaged. This damage would require repair, but would not necessarily prevent the tank retaining water, other than that lost by sloshing.

7.3 Trentham 2

Hoop Response



Hoop force ratio = 1.0 SLS, 0.65 ULS.

Vertical Wall Bending

Minimum tension reinforcement in the potential plastic hinge zone

Required	Provided	Minimum Requirement
0.001216	0.001752	OK

Foundation to Wall Connection

The wall is doweled to the foundation. The capacity calculations are based on the capacity of the active dowels and active tie bars in the connection for the total base shear.

Shear Action	Shear Capacity	Capacity/Action
42223 kN	32673 kN	<1

Foundation to Floor Connection

Structural connection between floor and foundation	OK

Roof to Wall Connection

The capacity of the roof and wall connection has been based on the external nib. The reinforcement is not detailed particularly well so it is based on the concrete strength.

Shear Action	Shear Capacity	Capacity/Action
6676 kN	10475 kN	>1

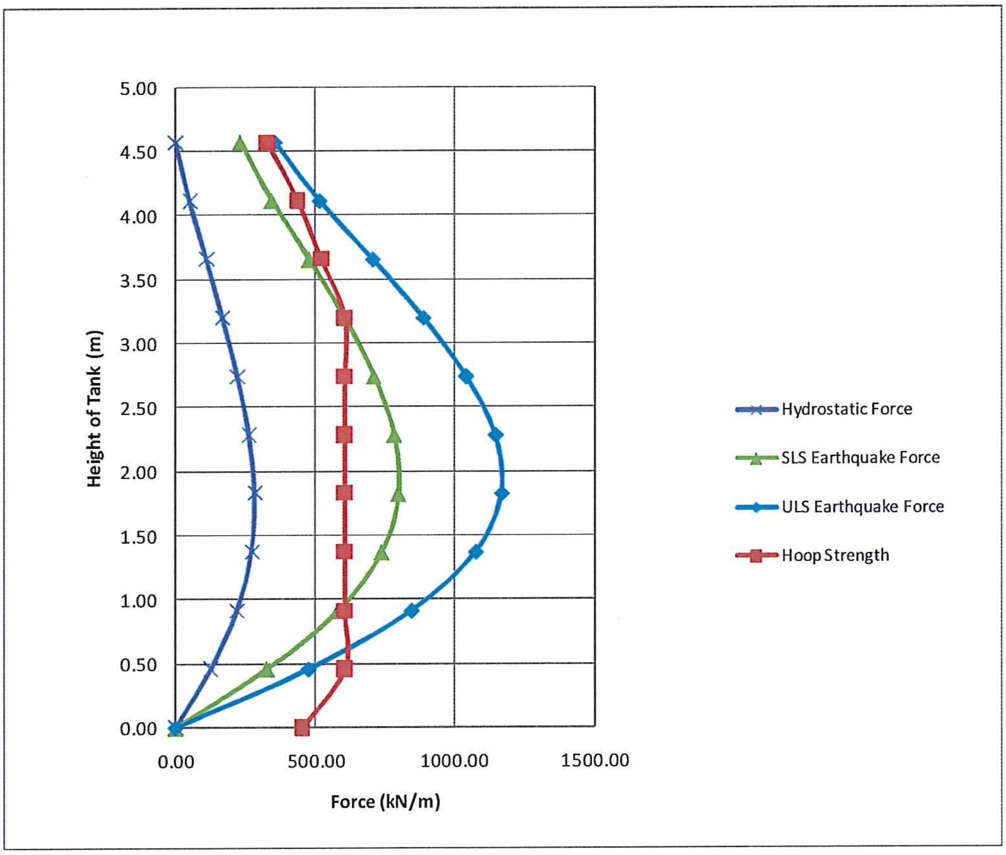
Also some of the load is expected to be taken by the rubber pads.

Convective Wave Height

Wave height	Clearance Height	
0.61 m	0.487 m	OK

7.10 Trentham 1

Hoop Response



Hoop force ratio = 0.75 SLS, 0.52 ULS.

Vertical Wall Bending

Minimum tension reinforcement in the potential plastic hinge zone

Required	Provided	Minimum Requirement
0.00603	0.00148	NG

Foundation to Wall Connection

The wall is well connected to the foundation, the detailing is good with continuity of reinforcement.

Shear Action	Shear Capacity	Capacity/Action
14670 kN	11287 kN	<1

Foundation to Floor Connection

Structural connection between floor and foundation	NG

Roof to Wall Connection

Well detailed internal nib. A conservative estimate used to confirm adequacy.

Shear Action	Shear Capacity	Capacity/Action
4281 kN	10964 kN	>1

Convective Wave Height

Wave height	Clearance Height	
0.57 m	0.610 m	OK

8 Performance Classification

The earthquake performance of the reservoirs relative to current design standard NZS 3106:2009 is classified in Table 2.

Table 2: Earthquake performance classification

Reservoir	Storage (m^3)	Year built	Hoop force ratio		Class	Critical weaknesses
			SLS	ULS		
Trentham 2	9100	1984	1.0	0.65	C	
Trentham 1	2270	1948	0.75	0.52	D	Floor to foundation connection. Vertical Reinforcement.

The reservoirs with performance class A, B and C meet the current design code performance requirements for essential facilities, i.e. operational continuity following the 500 year shaking. Performance class A reservoirs exceed current design standards for IL4 facilities and are expected to retain contents following 2500 year return period shaking.

Performance class D tanks have a high risk of loss of contents from leakage if subjected to ground shaking with return periods less than 500 years, including that caused by rupture of the Wellington Fault.

Damage is expected to the Chatsworth and Sylvan Heights tank farms tanks from ground subsidence and from tank sliding causing rupture at the pipe junctions.

In some cases the calculated sloshing wave heights are greater than the clearances between the top water levels and the roof. If the water pressures exceed the weight of the roof, this will result in sections of the roof being lifted off the supports and water will slosh out of the tank. While this may damage the roof structure, it is not expected to affect the water retaining capacity of the tanks so no remedial work is recommended to be necessary.

9 Remedial Options

The following are options for upgrading the performance of the Class D tanks.

Trentham 1

Trentham 1 is an older cast-insitu reinforced concrete tank. Unsurprisingly given its age, the hoop reinforcement falls short of current seismic design standards. The hoop reinforcing is expected to yield in strong earthquake shaking resulting in permanent outward hoop displacements and permanent vertical cracking with rapid loss of contents. The floor is also not connected to the wall foundation, which could result opening of and leakage from, the floor joint.

It is possible to improve the reservoir's earthquake performance significantly by adding hoop reinforcement to the walls and connecting the wall footings to the floor slab.

One approach to adding hoop reinforcement is to construct a steel shell around the tank and fill the gap with grout. This approach was adopted for the Gracefield and Maungaraki reservoirs in Lower Hutt, although in those cases the remedial work was required to deal with severely corroded prestressing steel as well as insufficient hoop reinforcement. It would be relatively straight-forward to connect the wall footings to the floor slab with drilled and epoxied dowels. The rough order estimate of cost for the upgrading using the steel shell option is $430,000. A less costly option of adding external, possibly prestressed, reinforcement might also be feasible.

Another remedial option would be to add a flexible, internal liner to the tank wall to prevent water flowing through the cracks. The roght order cost of this option is $80,000.

The SLS hoop force ratio will increase to 1.0 if the water depth is reduced from 4.5m to 3.5m.

10 Post-Earthquake Repair

Tanks that have been cracked by earthquake pressures but without the hoop reinforcing steel yielding should be readily repairable by sealing the cracks, either by grouting or applying a membrane coating. Reinforced concrete tanks that have yielded hoop steel will require the larger cracks to be grouted and a membrane coating applied.

Tanks with prestressed hoop reinforcement that has yielded will require strengthening by, for example, adding a steel shell or new prestressing cables, as well as application of a membrane coating.

11 Conclusions

The following ten Upper Hutt City Council reservoirs meet the earthquake performance requirements for essential (importance level 4) reservoirs specified in the new design standard for concrete reservoirs NZS 3106:2009. There is a low risk that they would lose their contents following a major earthquake centred for example on the Wellington Fault where it passes through Upper Hutt:

Trentham 2

The following six reservoirs have poor earthquake resistance and are expect to have large cracks in the walls and movement on joints leading to a rapid loss of contents following a Wellington Fault earthquake:

Trentham 1

12 References

[1] NZS 3106:2009, "Design of concrete structures for the storage of liquids", Standards New Zealand.

[2] BS 5896:1980, "Specification for High tensile steel wire and strand for the prestressing of concrete", British Standards.

[3] NZSEE, 2006, "Assessment and improvement of the structural performance of buildings in earthquake", Recommendations of a New Zealand Society for Earthquake Engineering study group on earthquake risk buildings.

[4] AS/NZS 1170.0:2002, "Structural design actions, Part 0 general principals", Standards New Zealand.

[5] NZS 1170.5:2004, "Structural design actions, Part 5 earthquake actions - New Zealand, Standards New Zealand.

[6] Stirling, M.W. & the Earthquake Hazards Team, 2007. Updating the national seismic hazard model for New Zealand. 8th Pacific Conference on Earthquake Engineering, Singapore, 5-7 December 2007.