Christchurch City lifelines – performance of concrete potable water reservoirs in the February 2011 Christchurch Earthquake and summary findings

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ABSTRACT: The magnitude M_w 6.2–6.3 earthquake that occurred in Christchurch on 22 February 2011 resulted in widespread damage to buildings and infrastructure.

Christchurch City's concrete reservoirs in the Port Hills and Cashmere Hills are located near the epicentre of the February earthquake and suffered damage of varying extent from nil through to major. Of 43 reservoirs, two were declared inoperable (including Christchurch's largest), three barely operable and requiring major repair works and a further fifteen reservoirs requiring lesser repair. The city lost 40% of its potable water storage as a result.

The Port Hills and Cashmere Hills concrete reservoirs are of varying vintage, construction type and storage capacity. These reservoirs form a useful database for assessment of performance in the February 2011 Christchurch Earthquake and also against current design standards.

Damaged roof-to-wall connections were observed in many reservoirs with damage to other elements such as walls, internal columns, base-slabs and wall-to-base connections observed in fewer reservoirs. Detailed seismic assessments, on eight of the reservoirs, are commonly identifying vulnerabilities with roof-to-wall and wall-to-base/foundation connections and therefore indicating reasonable correlation with the damage observed. Significant geotechnical issues were also observed at three of the reservoir sites.

A summary of damage observed and the performance of critical structural connections / details are presented in this paper along with reinstatement progress and key lessons learned. Key findings are that robustness in roof-to-wall and wall-to-base connections, the avoidance of joints in floor slabs and a suitable wall thickness provide increased reliability.

1 INTRODUCTION

1.1 Background / Port Hills and Cashmere Hills reservoirs

The magnitude M_w 6.2-6.3 earthquake that occurred in Christchurch on the 22 February 2011 resulted in the deaths of 185 people and caused widespread damage to buildings and infrastructure. Very strong ground shaking was experienced within a few kilometres of the earthquake epicentre.

Christchurch City Council's potable water supply network / lifeline includes over 50 bulk storage and service reservoirs with the majority located in the Port Hills and Cashmere Hills of Christchurch. The total potable water storage capacity of reservoirs in the Port Hills and Cashmere Hills is in the order of 102,000m^3. Approximately twenty five of these reservoirs are within a 5 km radius of the 22 February earthquake epicentre where significant residential and infrastructure damage occurred.

The majority of the reservoirs are of concrete construction, with construction dates varying from early 1900's through to 2000's, and of various geometry, construction type and storage capacity. Older reservoirs are typically of in situ reinforced concrete construction with the more modern structures being precast, often post-tensioned circumferentially and occasionally vertically. A number of the more recent reservoirs are of design and construct delivery comprising 150 thick walls, either singly or doubly reinforced and in a couple of instances circumferentially post-tensioned. The reservoirs are primarily of circular plan geometry.

1.2 Design overview

New Zealand's standard for the design of water retaining structures is *NZS3106:2009 Code of Practice for Concrete Structures for the Storage of Liquids* (first issued in 1986). This standard includes detailed methodologies for the design of earthquake loading including; calculation of hydrodynamic effects, connectivity and transfer of seismic shears from roof-to-wall and wall-to-base. On some of the older reservoirs, designed prior to NZS3106 being issued, roof-to-wall and wall-to-base connection retrofit works were observed at some sites.

1.3 Detailed seismic assessments

Detailed seismic assessments are being undertaken on all reservoirs moderately and extensively damaged and those designated Importance Level 4 (post disaster utilities) [AS/NZS 1170.0, NZS 1170.5]. These assessments are using a Percentage of New Building Standard (%NBS) philosophy, where %NBS is the assessed structural performance of the existing reservoir compared with current design requirements for a new reservoir at the site, expressed as a percentage. The 'New Building Standard' for Christchurch City Council (CCC) reservoirs is; 100 year design working life and either Importance Level 3 or 4, with other earthquake parameters as specified in NZS 3106 and NZS 1170.5. Comparison of NZS 1170.5 against horizontal acceleration data from the nearest strong motion stations in the Port Hills indicates that some reservoirs may have experienced accelerations in the order of up to 50% greater than the relevant code implied values (based on hazard factor $Z = 0.3$ and return period factor $R \geq 1.8$).

2 PORT AND CASHMERE HILLS CONCRETE RESERVOIRS – CONDITION GRADING, DAMAGE SUMMARIES

2.1 Initial earthquake response inspection summary

Inspections were undertaken, following the 22 February 2011 earthquake, on 43 concrete reservoirs located in the Port Hills and Cashmere Hills. The reservoirs were all evaluated, based on the damage observed, to CCC's condition grading schedule and the results are presented in Table 1. Inspections following the 4 September 2010 earthquake noted minor damage to four of these reservoirs only [Davey 2010].

Table 1: Reservoir condition grading following 22 February 2011 earthquake

CCC Condition Grade	Description of grading	Number of Reservoirs
1	No repairs required – undamaged	23
2	Minor repairs required. Asset operable	10
3	Repairs required but asset still operable	5
4	Substantial repairs required. Asset barely operable	3
5	Asset inoperable. Major repairs or replacement required	2

Of the 43 concrete reservoirs, twenty were identified as requiring repair; two were declared inoperable, three barely operable and requiring substantial repair, and a further fifteen reservoirs requiring minor to moderate repair and although currently remaining essentially fully operable will require removal from service for repairs to be undertaken. The two reservoirs declared inoperable, Huntsbury No.1 and McCormacks Bay No.1, resulted in an immediate reduction of approximately 40% of the network's storage capacity.

2.2 Damage summary for the five most significantly damaged reservoirs

Brief summaries of observations for the five most significantly damaged reservoirs are provided in Table 2 [Charman and Billings, 2011]. Inspections following the September 2010 earthquake did not identify any damage to these reservoirs, the damage all attributable to the 22 February 2011 event.

Table 2: Summary observations – five most significantly damaged reservoirs

Reservoir (Storage capacity)	Distance from Feb. earthquake epicentre	Condition Grade	Details of damage observed
Huntsbury No.1 (35,000 m^3)	3.0 km approx.	5 (Inoperable)	Major cracking and movement of base-slabs (cracks up to 35 mm wide). Roof slab cracking and slippage at construction joints. Wall cracking and opening of joints. Loss of entire 32,000 m^3 contents. Significant geotechnical issues, including the existence of a shear zone within the rock beneath the reservoir.
McCormacks No.2 (5,000 m^3)	2.0 km approx.	5 (Inoperable)	Cracking in base-slabs. Spalling and cracking around wall-foundation ring beam. Movement of wall relative to foundation beam. Failure of roof-wall connections and resulting in roof and wall damage. Leakage through wall joints. Significant geotechnical issues at site including rock falls, collapse of retaining walls and cracking, settlement and slumping of the access road and reservoir platform.
McCormacks No.1 (5,000 m^3)	2.0 km approx.	4	Similar to McCormacks No.2, but also leakage through tendon anchorages and more significant damage to its roof and top of wall.
Upper Balmoral (1,000 m^3)	2.0 km approx.	4	Major cracking and damage at top of wall / pilasters from roof beam impact. Damage at top of internal column supporting roof.
Clifton 3 (455 m^3)	3.7 km approx.	4	Failure of internal column, sagging of roof, shearing of roof overhang at roof-wall interface

3 COMMON STRUCTURAL DAMAGE

The structural damage observed at the 43 reservoir sites in the Port Hills and Cashmere Hills has been collated by areas of particular interest and is summarised in Table 3.

Table 3: Typical structural damage observed

Damage observed	Number of reservoirs / sites with observed damage
Roof / roof to wall connections	12
Wall damage / leaking	7
Internal column damage	4
Cracked / damaged wall-to-base connection or base-slabs	5
Pump house damage	8

The most commonly observed damage was at roof-to-wall connections and this is discussed in Section 4.1.

Leakage through either wall joints or wall-to-base connections was observed in nine reservoirs and some of these have remained in service to maximise storage for the summer water demand and while other repairs are undertaken. Of note is that leakage was observed from post-tensioning anchorages on at least two reservoirs. Wall joints and wall-to-base connections are discussed in Sections 4.2 and 4.3 respectively.

Internal column damage has generally been limited to those reservoirs with major damage to roof-to-wall connections and only Clifton 3 and Upper Balmoral require significant reconstruction.

Damage to pump houses has also occurred, varying from minor block wall damage to more substantial damage at a couple of sites including significant wall cracking and spalled concrete exposing reinforcement.

Significant geotechnical issues exist at three of the reservoir sites, refer Section 5 for commentary.

4 PERFORMANCE OF CRITICAL STRUCTURAL CONNECTIONS / DETAILS

As noted in Section 3 the most commonly observed damage in the Port and Cashmere Hills reservoirs was to roof-to-wall connections. Other areas where damaged was observed, albeit in a lesser number of reservoirs, were wall-to-base connections and wall joints in circular reservoirs and also base-slabs.

Typical findings from the Percentage of New Building Standard (%NBS) seismic assessments completed on eight reservoirs to date include; roof-to-wall and wall-to-base vulnerabilities, a potential deficiency in resistance to sliding and insufficient freeboard to roofs. These results indicate reasonable correlation with the damage observed in those particular reservoirs.

Damage observations and assessment findings noted above are discussed in the following sections.

4.1 Roof-to-wall connection details

Damaged roof-to-wall connections were observed in twelve of the Port Hills and Cashmere Hills reservoirs. Dowel connections through the roof into the wall have performed particularly poorly while damage has also occurred with other roof-to-wall connection details.

Results from the detailed seismic assessments, including %NBS values, and damage observed for various roof-to-wall details are described in Sections 4.1.1 and 4.1.2.

4.1.1 *Performance of roof-to-wall dowels - McCormacks, Mt Pleasant 4, Moncks Spur 3, Clifton 4*

Roofs to the two McCormacks Bay reservoirs are of composite construction, comprising precast prestressed tee units with an in situ topping and a 300 overall depth overhang around the top of the wall – refer to Figure 1 for details. Restraint bolting is provided between the roof and the walls, with the bolts passing through a pocket in the roof unit flanges and screwed into inserts at the top of the wall. Reservoir No.2 (1995) has 120 bolted inserts compared to 40 inserts in Reservoir No.1 (1984). The %NBS values estimated from detailed seismic assessments, for this detail, are 45-50% and 10-15% for Reservoirs No.2 and No.1 respectively.

The calculations suggest the likely performance on 22 February 2011 has been that, once bolts have yielded / failed the roof has translated with the overhanging nib impacting the reservoir wall. The strength of this nib for bending, shear and tension is limited and in Reservoir No.1 excessive roof movement has occurred which has smashed the overhanging roof nib, resulted in the precast roof tee units engaging the wall, punching out of wall concrete and damage occurring at the ends of the tee unit webs (Figures 2 and 3). Less damage has been observed in the roof of Reservoir No.2 possibly due to it having three times the number of bolted connections. However, physical investigations undertaken on the Reservoir No.2 bolted connections indicate that of the three connections that were cored / exposed two of the bolts had fractured.

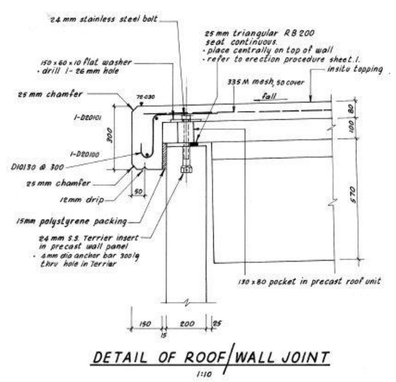

Figure 1 - McCormacks roof-to-wall dowel detail

Figure 2 - McCormacks roof / wall impact damage

Figure 3 - McCormacks roof overhang

Mt Pleasant 4 reservoir (constructed circa 1964) has retrofitted roof-to-wall dowels. These dowels are plain reinforcing bar extending between the roof and the 250 mm thick wall with rubber sleeves over the depth of the roof. Physical investigations indicate that these have yielded and undergone significant deformation, though the resultant impact on the reservoir does not appear to be significant.

Moncks Spur 3, constructed in 2000, has a similar detail to the McCormacks Bay reservoirs but with a rubber sleeve in the roof in lieu of a pocket. The assessed %NBS for the Monks Spur 3 detail is approximately 35%. Physical investigations indicate these roof-to-wall dowels have yielded and deformed. Spalling at the top of the 150 mm thick reservoir wall was also observed at some of the dowel locations, possibly due to prying action and insufficient concrete cover to the dowels.

Clifton 4, constructed in 1987, has cast-in inserts at the top of its 150 thick wall with reinforcing bars screwed into the inserts and bent down into the roof topping. The assessed %NBS for the Clifton 4 detail is approximately 80-100%. Physical investigations suggest the bars appear undamaged but spalling at the top of the reservoir wall has occurred at a number of the wall insert locations.

4.1.2 *Performance of alternative roof-to-wall details*

Alternative roof-to-wall details have been observed at other sites. The details and performance of those for Clifton 3, Upper Balmoral and Worsleys Road reservoirs are outlined below.

The Clifton 3 roof (construction circa 1948) is not physically connected to the top of the reservoir wall but has an edge overhanging ring beam which achieves a slight overlap with the top of the wall externally. Under horizontal earthquake loading the roof has impacted the top of the wall and resulted in sections of the overhanging nib shearing off. Excessive translation of the roof subsequently occurred and resulted in the central column failing and requiring substantial reconstruction.

The Upper Balmoral (1986) roof comprises precast beams that slot into the 150 thick reservoir wall and pilasters with compressible packing between the beams and the wall and with grout filling the gap at the ends of the beams at pilasters (refer Figure 4). Under horizontal earthquake loading the roof beams have punched off the thin skin of pilaster concrete with the beams then impacting the wall and resulting in major cracking and damage to the top of the wall (Figure 5). Although there is reinforcement connecting the roof to the wall this passes through a duct cast into the beams and thus provides minimal resistance to lateral loading. Calculations indicate this detail achieves approximately 15%NBS.

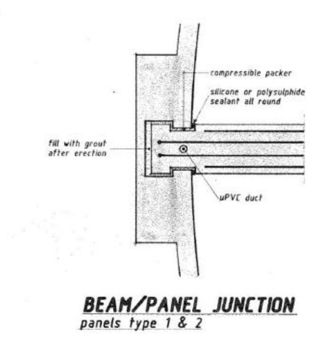

Figure 4 - Upper Balmoral roof detail

Figure 5 - Upper Balmoral wall damage

Figure 6 - Worsleys Rd reservoir No.1 epoxy bonded shear plates

The two Worsleys Road reservoirs were constructed circa 1968. The roofs for these reservoirs are of in situ construction and post-tensioned. As for the Clifton 3 reservoir, there is no physical connection between the roof and the wall but the perimeter of each roof steps up and across the wall with a 25 mm gap between the roof and the wall all around. The roof is free to slide on top of the wall until this gap closes up and it impacts the inside surface. Castellated / interlocking steel shear plates were retrofitted to Reservoir No.1 in the mid 1990's, epoxy bonded to the roof and the wall (with nominal temporary locator bolts). Approximately 50% of these shear plates were damaged in the September 2010 earthquake with further damage to all remaining pairs of plates occurring 22 February 2011 (refer Figure 6). This has resulted in areas of wall and roof concrete subsequently requiring repair. Of note is that the damage to the Reservoir No.2 roof and wall appears to be minimal with only some dislodging of the compressible packing between the roof and the wall observed.

4.2 **Performance of wall construction joints**

Water leakage through wall vertical construction joints was observed in four reservoirs.

Clifton 4 and both McCormacks Bay reservoirs have circumferentially post-tensioned walls. Clifton 4 has surface sealants, providing additional water-tightness protection, at each wall in situ joint line (but possibly deteriorated) while the McCormacks Bay reservoirs rely only on residual compression for water-tightness. Calculations suggest the McCormacks Bay tendons were loaded beyond their proof stress / limit of proportionality and which may have reduced the residual wall compression slightly.

Mt Pleasant No.2-2 (construction 1957) is conventionally reinforced vertical and circumferentially. Leakage was observed from eight vertical construction joints suggesting some yielding of the reinforcement has occurred. It is unknown whether the joints have a central water-stop but the joints do not have an internal surface sealant.

4.3 **Wall-to-base connection details**

Damaged wall-to-base connections were observed in limited reservoirs only, however current and previous seismic assessments indicate vulnerabilities with this connection. A lack of a robust wall-to-base connection may result in circumferential sliding, adversely stressing and compromising the performance of the reservoir wall(s) and foundation ring beam.

4.3.1 *McCormacks, Clifton 4 and Upper Balmoral – wall-to-foundation ring beam connections*

Each McCormacks Bay reservoir wall is supported off a ring beam foundation in a slot formed between an external kerb and the internal base-slab. The wall is connected to the foundation at each in situ joint wall pour with reinforcing bars cranked into the external kerb pour. This in situ wall joint reinforcement provides the majority of the connection capacity, for transfer of circumferential shear, between the wall and foundation. Additional capacity is achieved by the pilasters keying in to the foundation along with frictional contributions and the reserve capacity in the wall (which requires sliding to occur). As for the roof-to-wall connections there are noticeable differences in the wall-to-base details between the two reservoirs: No.2 has twice the in situ joint reinforcement compared to No.1 and the interface between the external surface of wall and external kerb was scabbled for No.2 but had 2 coats of bitumen emulsion for No.1. Of interest is their relative performance on 22 February 2011, when considering assessed %NBS values of 55-70% for Reservoir No.2 and 35-50% for Reservoir No.1. More damage was observed in Reservoir No.2 including cracking of the foundation ring beam, spalling of concrete at pilaster locations and noticeable movement between the wall and ring-beam. Less damage is apparent in Reservoir No.1 at this connection.

Clifton 4 and Upper Balmoral have similar wall-to-foundation details to the McCormacks reservoirs. The design performance is similar and assessed %NBS values are 70-90% and 50% respectively. Observations suggest some earthquake damage occurred at Clifton 4, including cracking of the foundation ring beam external kerb at wall joint locations, leakage through the wall, possible leakage from beneath the reservoir at the wall-to-foundation ring beam connection and movement between the base slab and central column. No significant damage was apparent at the base of the Upper Balmoral wall despite its low %NBS and the reservoir's proximity to the February 2011 earthquake epicentre.

4.3.2 *Moncks Spur – wall-to-foundation ring beam connection*

Monks Spur 3, which is conventionally reinforced, has essentially a monolithic connection between the wall and base-slab with bars bending from the wall into the base-slab, a scabbled interface and a fully continuous base-slab. It achieves in the order of 100%NBS and other than some localised spalling of the slab against the wall internal surface this connection appears relatively undamaged.

4.3.3 *Worsleys Rd Reservoirs – retrofit wall-to-foundation ring beam connection*

Each Worsleys Road reservoir wall was originally constructed seated on an in situ ring beam with an external kerb poured after stressing and without any physical connection between the wall and foundation. An internal ring beam was retrofitted to the base of the wall in the mid 1990's with horizontal dowels into the wall and vertical dowels extending through the two layers of base-slab into the original ring beam foundation. A detailed assessment of this ring beam has not yet been completed for current design requirements but it appears to have performed adequately.

4.4 **Base-slabs – double layer construction**

A number of reservoirs in the Port Hills and Cashmere Hills, both older and more recent structures, have base-slabs comprising two 100-150 mm thick slab layers. Each layer typically comprises individual segments/units (without continuity) with the top layer offset from the bottom layer. Frequently the slabs are not connected to perimeter walls / foundations or internal column foundations.

Cracking was observed in the McCormacks Bay base-slab top layers (both reservoirs) generally coincident with construction joint locations in the bottom layer. Calculations indicate sliding capacity is deficient even though the bottom layer has some nominal connection to the wall and column foundations. Combined with the low capacity of the wall-to-foundation connection the cracking and movements that have occurred appear consistent with the seismic assessment findings.

Neither of the two slab layers in the Clifton 4 reservoir have any connection to the wall or column foundation. Calculations indicate a deficiency with sliding resistance and observations suggest some movement has occurred as noted in Section 4.3.1. Upper Balmoral has similar detailing and construction but with no apparent damage despite an assessed sliding vulnerability.

The Worsleys Road reservoirs two layer base-slabs were originally independent of the walls and internal columns with flexible sealants between adjacent structural faces. A ring beam connecting the wall, foundation and the perimeter slabs (Section 4.3.3) has been retrofitted to provide continuity.

Huntsbury No.1 reservoir's internal slabs were also individually constructed 'tiles' with flexible bituminous sealant (full depth) in joints between top layer slabs. Whether a fully continuous base-slab would have performed significantly better, given the underlying geotechnical conditions, is unknown but modern design of continuous base-slabs in water retaining structures requires a significantly higher reinforcing content leading to a much more robust slab.

5 GEOTECHNICAL OBSERVATIONS

Significant geotechnical issues were observed at the Huntsbury No.1, McCormacks Bay and Murray Aynsley reservoir sites.

5.1 Huntsbury No.1 reservoir site

At the Huntsbury reservoir site extensive geotechnical investigation indicated that an underlying shear zone, extending diagonally across the reservoir footprint, exists and which is considered the main reason for the observed structural damage. The Christchurch Earthquake appears to have reactivated an underlying shear zone resulting in movement occurring on planes of weakness in the rock. Given the on-going period of increased seismicity and likelihood that this shear zone could remain active for some time, the final configuration of the site reinstatement is two smaller reservoirs constructed in the corners of the current reservoir footprint as shown in Figure 7.

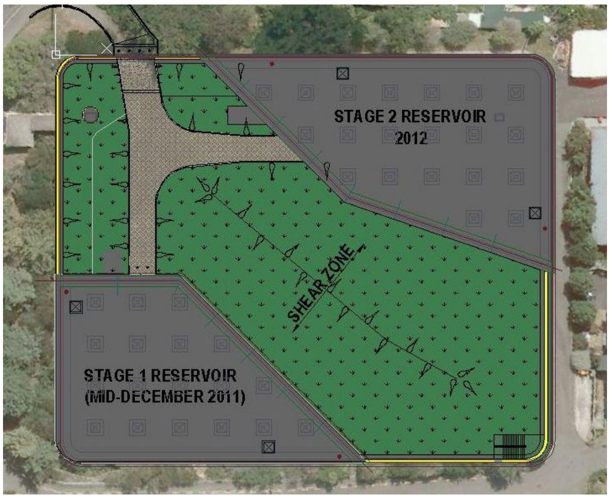

Figure 7 – Huntsbury No.1 Reservoir – shear zone and proposed site reinstatement

5.2 McCormacks Bay reservoirs site

The McCormacks Bay reservoirs site requires extensive stabilisation. The access road to the reservoirs has significant cracking, settlement of up to 500 mm and slumping towards the slope below the reservoirs (Figure 8). The stacked basalt block gravity wall supporting this road partially collapsed on 22 February 2011 resulting in rockfall hazards to the road and residents below, and requiring the installation of emergency temporary rockfall fences to mitigate short term rockfall risks. A permanent piled replacement retaining wall is currently proposed along the edge of the road.

Figure 8 – McCormacks Bay reservoirs – access road damage and rock fall around reservoirs

The construction platform for the reservoirs was cut into rock and there is a near vertical face, with a height of up to approximately 21 m, behind the reservoirs. The large ground accelerations that were experienced at the site on 22 February and 13 June 2011 led to significant loosening and dislodgement of material from the rock face. Rock and material has piled up behind the reservoirs and some rocks may have impacted the reservoir wall (Figure 8). It is currently proposed that reinforced steel mesh and a grid of rock anchors and soil nails be installed over the full extent of the cut face.

5.3 **Murray Aynsley reservoir site**

Murray Aynsley reservoir is located approximately 4 m from the edge of a 15-25 m high old quarry cliff. Sections of the cliff face collapsed on 22 February 2011 and the ground up to the reservoir has severe tension cracking. Continued regression of the slope would likely lead to undermining of the reservoir and relocation of the reservoir appears likely to be necessary in the future.

6 STATUS OF REPAIR AND REINSTATEMENT

As at January 2012, the status of the five most severely damaged reservoirs is as follows:

Table 4: Status of repair and reinstatement

Reservoir	**Summary of reinstatement details and status**
Huntsbury No.1	Stage 1 6,200 m^3 reservoir and pump station completed December 2011, and in service. Construction of the second, Stage 2, replacement reservoir is anticipated to be completed by around mid-2012. The total estimated storage volume for the site, at completion of reconstruction, is 13,600m^3 (40% of original 35,000 m^3 capacity). New retaining walls, remaining demolition and landscaping are currently expected to be complete by late 2012.
McCormacks No.1	Reservoir No.1 remains in service but is leaking. Full repair and retrofit expected to commence around April 2012 once water demand reduces – details are likely to be similar to reservoir No.2.
McCormacks No.2	Stage 1 repairs completed December 2011 and reservoir returned to service. Stage 1 repairs comprised; base-slab overlay and ring beam, and bandaging of wall joints up to around 2.0 m above slab level. Stage 2 repairs include completion of wall joint bandaging, repairs to the roof and retrofitting of a concrete ring beam around the top of the wall – programmed for mid-late 2012. Geotechnical work at the site is extensive and expected to continue till late 2012.
Upper Balmoral	Repairs completed December 2011 and reservoir returned to service. Repair and retrofit included a concrete ring beam around the top of the wall (tied to the roof) and an internal ring beam connecting the base of the wall to the base-slab.
Clifton 3	Repair and retrofit completed, returned to service December 2011. Repair / retrofit included break-out and reconstruction of the damaged column, jacking of the roof vertically and fitting of a concrete ring beam at the top of the wall to restrain the roof against lateral movement

7 CONCLUSIONS AND LESSONS LEARNED

Approximately 75% of Christchurch City's concrete reservoirs within the Port Hills and Cashmere Hills either did not require any repair or only required minor repairs following the devastating 22 February 2011 Christchurch Earthquake. Substantial repair through to re-construction are required at five locations.

Damaged roof-to-wall connections were observed in many reservoirs with damage to walls, wall joints, base-slabs and internal columns limited to fewer reservoirs only. Findings from completed seismic assessments include; roof-to-wall and wall-to-base/foundation connection vulnerabilities, a potential deficiency in resistance to sliding and insufficient freeboard roofs. It is likely a number of reservoirs throughout New Zealand have similar roof-to-wall and wall-to-base vulnerabilities.

Roof-to-wall connections with dowels through the roof into the wall have performed particularly poorly and their overall robustness appears questionable. Dowel size, design and connection detailing for the roof and at the top of the wall require careful evaluation for the future use of this method. Wall-to-foundation connection details appear to have generally performed better though damage was observed at a number of reservoirs and typically in those with discrete (non-continuous) base-slab panel construction. A simple code compliant design philosophy with interconnected walls, foundations and continuous internal base-slabs provides a direct mechanism for the transfer of seismic shear from the wall in to the foundation / base-slab and to the ground - therefore increasing overall robustness and reservoir performance. Where previous construction and design techniques possibly limited pour sizes, modern design and construction can readily incorporate watertight fully continuous slabs.

Robustness is an important parameter that requires consideration in the design and detailing of these lifeline structures to meet functional / operational requirements following a design earthquake. The performance of the Port Hills and Cashmere Hills reservoirs, in the 22 February and 13 June 2011 earthquakes, indicates that robustness in roof-to-wall and wall-to-base connections, the avoidance of joints in floor slabs and a suitable wall thickness would all provide increased reliability. The use of a double protection system to achieve water-tightness at construction joints is also recommended.

Significant geotechnical issues were observed at three reservoir sites in the Port Hills and Cashmere Hills. At Huntsbury, which was Christchurch City's largest reservoir, this has resulted in reconstruction to a reduced storage capacity. The Murray Aynsley reservoir appears likely to require relocation in the future.

Repair and reinstatement continues into 2012 and has been staged to maximise storage capacity for the 2011-2012 summer water demands. Once currently proposed repairs are completed the potable water network will only have 80% of the capacity it had prior to the 22 February 2011 earthquake. The 20% decrease in network capacity is a result of the reduced storage capacity available from reconstruction of the Huntsbury reservoir site.

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