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CHRISTCHURCH CITY LIFELINES – ASSESSMENT AND REPAIR OF CONCRETE POTABLE WATER RESERVOIRS FOLLOWING THE FEBRUARY AND JUNE 2011 CHRISTCHURCH EARTHQUAKES.

Ian BILLINGS[1] and Nicholas CHARMAN[2]

[1] Technical Director – Structural Engineering, Beca, Auckland, New Zealand
ian.billings@beca.com
[2] Technical Director – Structural Engineering, Beca, Brisbane, Australia
nicholas.charman@beca.com

ABSTRACT: The Canterbury region, in New Zealand's South Island, is experiencing a period of significant seismic activity that started in September 2010 and that continues in 2012. Christchurch City's concrete reservoirs within the Port and Cashmere Hills are located very close to the epicentre of the devastating magnitude 6.3 22 February 2011 earthquake and suffered damage varying from nil through to major - and the city lost 40% of its potable water storage. Two reservoirs were inoperable (including Christchurch's largest), three barely operable and requiring major repair works and a further fifteen reservoirs requiring lesser repair.

Key Words: Christchurch City New Zealand, 22 February and 13 June 2011 earthquakes, concrete reservoirs, performance, assessment and repair

INTRODUCTION

Background

On 22 February 2011 a (Richter) magnitude 6.3 earthquake occurred in Christchurch, in the South Island of New Zealand, and resulted in the deaths of 182 people and caused widespread damage to buildings and infrastructure. On 13 June 2011 a second magnitude 6.3 event caused further damage. Very strong ground shaking was experienced within a few kilometres of the 22 February and 13 June epicentres (and which were approximately 10 km from the city). A magnitude 7.1 earthquake with epicentre 40 km west of the city had occurred earlier on 4 September 2010.

Christchurch City – potable water network / lifeline

Christchurch City Council has an extensive potable water supply network / lifeline comprising piping, deep wells, pump stations and bulk storage and service reservoirs. It provides water to approximately 320,000 residents.

Over 50 bulk storage and service reservoirs form part of the potable water network with the majority of these located in the Port Hills and Cashmere Hills of Christchurch City.

Many reservoirs in the Port Hills and Cashmere Hills are within a 5 km radius of the 22 February and 13 June 2011 earthquakes' epicentres where significant residential and infrastructure damage occurred. The total water storage capacity of reservoirs in these areas is in the order of 102,000m^3.

Bulk storage and service reservoirs – details

Overview

The majority of the network's reservoirs located within the Port Hills and Cashmere Hills are of concrete construction with a few others being timber or steel. The concrete reservoirs are of varying age (1900's through to 2000's), geometry, construction type and volume and are typically founded on platforms cut into the underlying tuff and basalt rock.

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 singly reinforced, and in a couple of instances circumferentially post-tensioned, 150 thick precast walls.

The reservoirs are primarily of circular plan geometry with a few being rectangular. The largest bulk storage reservoir is Huntsbury No.1 with a capacity of approximately 35,000m^3. Other bulk storage reservoirs include Worsleys Road No.1 and No.2 – 10,000 m^3 each, and McCormacks Bay No.1 and No.2 – 5,000 m^3 each.

The bulk storage reservoirs generally have seismic actuated valves on inlet/outlet mains pipes for security with operation and supply. The smaller service reservoirs typically do not have these types of valves.

Design background

New Zealand's current 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 these types of structures for earthquake loading including; calculation of hydrodynamic effects, connectivity and transfer of seismic shears from roof-to-wall and wall-to-base. Many reservoirs constructed prior to this standard was first issued are potentially deficient with respect to some of these specific design aspects. On some of the older reservoirs roof-to-wall, and also wall-to-base, connection retrofit works were observed at some sites. Also, the level of horizontal earthquake loading that the reservoirs were originally designed for is expected to vary considerably, relative to their respective age and the applicable requirements at that time. Due to the increased seismicity, the earthquake design hazard factor for Christchurch was increased by 36% in May 2011 and it is thus likely that few, if any, reservoirs would fully meet current earthquake loading requirements for water retaining structures.

As reservoirs are generally key items of lifeline infrastructure they need to remain functional / operational after a design earthquake event with only minimal repairs necessary. Repairs would then be undertaken from outside the reservoir or left until a maintenance period arises and the reservoir emptied. Under earthquakes which cause ground accelerations greater than the design level earthquake a hierarchy of failure is required that minimises damage and potential for loss of contents.

Contents

This paper describes:

- The post-earthquake inspection, assessment and reinstatement process
- A summary of the inspections and damage observations of 43 concrete reservoirs, following the 22 February and 13 June earthquakes, that are located in the Port Hills and Cashmere Hills
- Seismic assessment findings
- Reinstatement details and current progress
- Case studies on five of the most significantly damaged reservoirs
- Conclusions and lessons learned.

METHODOLOGY - INSPECTION, ASSESSMENT & REINSTATEMENT

The inspection, assessment and reinstatement process for the reservoirs was developed over a number of months as restoration priorities were identified by Christchurch City Council for the supply network and as the detailed work progressed. A summary of this process is described in the following sections.

Inspections

In the days following the 22 February 2011 magnitude 6.3 earthquake physical inspections were undertaken on all reservoirs within the supply network. These inspections included damage mapping (structural, geotechnical and infrastructure / pipework) and assessing each reservoir to a grading schedule that had been developed by Christchurch City Council (CCC). This provided the Council with a snapshot of its reservoirs with respect to operability and level of damage. A further round of these 'rapid' inspections was undertaken immediately following the 13 June 2011 earthquakes and similarly following the 4 September 2010 earthquake [Davey, 2010].

Detailed structural inspections and investigations were required on the most significantly damaged reservoirs, depending on the damage observed and correlation with seismic assessment results. These detailed investigations included; internal inspections, surveying, selected invasive investigation (coring of concrete, exposing of roof-to-wall dowels) and materials testing.

The extent of detailed geotechnical investigation necessary was similarly tailored to suit specific sites depending on initial site observations, requirements from the structural assessments and for development of appropriate reinstatement concepts where applicable. This included digging of test pits, bore-holes (vertical and inclined) and ground penetrating radar (GPR) surveying.

Closed Circuit Television (CCTV) and leak detection pipeline inspections were carried out, where considered necessary, on inlet and outlet mains pipes and stormwater lines.

Detailed seismic assessments

Prioritising of detailed seismic assessments

Following completion of the initial inspections after the 22 February 2011 earthquake, development and implementation of repairs on damaged reservoirs commenced. For the few reservoirs that were extensively damaged, detailed seismic assessments had begun and reinstatement options and design reports were being developed including cost estimates and construction programmes. For other reservoirs reinstatement repairs were planned or underway.

As a result of damage to completed and partially completed repairs in the 13 June 2011 earthquake it was subsequently agreed with CCC that detailed seismic assessments would be undertaken and engineered retrofit options developed for all reservoirs moderately and extensively damaged, and those designated Importance Level 4 – post disaster utilities [AS/NZS 1170.0 and NZS 1170.5]. Engineered repair and retrofit would provide increased security of supply to Christchurch's damaged potable water network as further significant events are expected during the on-going period of increased seismicity (note - magnitude 5.8 and 6.0 events occurred 23 December 2011).

Seismic assessment procedure

Reservoir structural element capacities / actions have been assessed at each site using a %NBS 'Percentage of New Building Standard' philosophy [NZSEE, 2006] - the standard that would apply to a new reservoir / structure constructed at the site. This is a similar approach to that being used for assessment of risks and the grading of existing building structures. The 'New Building Standard' for the reservoirs is as specified by Christchurch City Council; 100 year design working life and either Importance Level 3 or 4, with other earthquake parameters generally as provided in NZS3106 and NZS1170.5. Comparisons have also been made between current earthquake design requirements and available strong motion data recorded at nearby stations / sites.

The assessments have considered – strength; including wall, roof-to-wall connection and wall-to-base connection capacities, and sliding, overturning and calculation of convective wave heights.

The seismic assessment results for each reservoir are being included in design reports, associated risks / critical structural weaknesses assessed and described, and options for retrofit / strengthening presented where applicable.

Repair / reinstatement development

The primary priority in the repair and reinstatement of the damaged reservoirs was maximising water storage for the 2011-2012 summer peak demand period. This included consideration of; multiple structural repair and reinstatement options for each reservoir site, constructability input / preferences and, for those reservoirs most significantly damaged, staging of repair works. The staging typically required completing the minimum repair work in the time available to achieve operability for the peak water demand and leaving the remaining repairs for completion during periods of reduced demand.

Repair and reconstruction delivery

The work was carried out for Christchurch City under a design and construction delivery with close collaboration between the City Council, constructor Fulton Hogan and consultant Beca. Fast tracking of assessment, design, approval and consenting was necessary to meet the very tight programme that had been developed to maximise available storage for the summer water demands. This was very successful and included the team being able to respond effectively to the dynamic situation posed by a further major earthquake during reconstruction.

PERFORMANCE OF PORT HILLS AND CASHMERE HILLS CONCRETE RESERVOIRS

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 below.

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 (McCormacks No.1, Clifton 3, Upper Balmoral)
5	Asset inoperable. Major repairs or replacement required	2 (Huntsbury No.1, McCormacks No.2)

Overall, the 43 concrete reservoirs are considered to have performed reasonably well structurally with approximately 75% of the reservoirs either not requiring any repair or only requiring minor repair. Twenty reservoirs 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 a number of these require removal from service for repairs to be undertaken. In general, damage observed during inspections following the 13 June 2011 events has not impacted the grading results shown in Table 1. Inspections following the 4 September 2010 earthquake noted minor damage to two of these reservoirs only [Davey, 2010].

The two most severely damaged reservoirs, Huntsbury No.1 and McCormacks No.2, account for approximately 40% of the network's storage capacity.

Damage observations – summary

Structural observations

The damage observed at the 43 reservoir sites has been collated and is summarised in Table 2.

Table 2: Common structural damage observed

Damage observed	Number of Reservoirs / Sites With Observed Damage
Roof / roof to wall connections	12
Wall damage / leaking (excl. roof to wall damage)	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. Connections having dowels through the roof and into the wall performed particularly poorly, with damage including bent reinforcement and failed bolts observed in six reservoirs. Concrete spalling or prying at dowel locations was also often noted, possibly due to inadequate concrete cover and the wall thickness. The observed performance suggests dowelled roof-to-wall connections have only limited robustness.

Leakage through walls or through wall-to-base connections was observed in nine reservoirs and although there are a number of reservoirs with noticeable leakage, as noted in Table 1 some of these remain in service to maximise storage for the summer water demands. Of note is that leakage was observed from post-tensioning anchorages on at least two reservoirs.

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.

Details of the five most significantly damaged reservoirs are included in the Case Studies later in this paper.

Geotechnical observations

Significant geotechnical issues were identified at three reservoir sites - Huntsbury, McCormacks Bay and Murray Aynsley, with lesser issues at a few others and further investigation still to be completed.

Extensive geotechnical investigation of the Huntsbury reservoir site indicated the likely existence of a shear zone within the rock beneath the reservoir and has resulted in partial abandonment of the site. Refer Case Study 1 for additional detail.

The McCormacks Bay site requires extensive, and costly, stabilisation due to on-going rock falls, collapse of existing retaining walls and cracking, settlement and slumping of the access road and reservoir platform. Refer Case Study 2 for additional detail.

Murray Aynsley Reservoir is located approximately 4.0m from the edge of a 15-25 m high quarry cliff which is gradually collapsing. Relocation of the reservoir is likely to be necessary in the future.

At other sites identified as requiring further geotechnical investigation, slumping and cracking of access roads has occurred along with minor-moderate rock falls.

Pipework

Damage to pipe work was observed at a few reservoir locations only. The lack of significant damage to pipe work is thought to be a result of these reservoir sites generally being founded on rock. More significant damage to inter-connecting infrastructure has occurred at sites founded on weaker / liquefiable materials.

Desktop seismic assessments – findings

Progress to date

Seismic assessments have been completed for the eight reservoirs that require the most significant repair. The analyses and assessments have used a %NBS philosophy as outlined previously, representing current design requirements for a new reservoir / structure constructed at the site.

Seismic demand 22 February 2011 earthquake

Approximately twenty of the concrete reservoirs within the Port Hills lie inside a 5 km radius of the 22 February 2011 earthquake epicentre. A similar number lie within a 5 km radius of the 13 June 2011 earthquake epicentre. Comparison of NZS1170.5 against horizontal acceleration data from the nearest strong motion stations in the Port Hills indicates that some reservoirs may have experienced accelerations of up to in the order of 50% greater than the relevant code implied values (based on hazard factor $Z = 0.3$* and return period factor $R \geq 1.8$). *Note 'Z' for Christchurch was increased from 0.22 to 0.3 in May 2011.

Summary and discussion

Typical findings from the seismic assessments completed to date include; roof-to-wall and wall-to-base vulnerabilities, a potential deficiency in resistance to sliding and insufficient freeboard to roofs. The results indicate reasonable correlation with the damage observed in those particular reservoirs.

As noted above, reservoirs nearest the February and June 2011 earthquake epicentres may have experienced earthquake loading considerably in excess of New Building Standard. This has not been assessed in detail but the overall functional performance of the reservoirs, other than a few exceptions, is considered reasonably good as the ten most significantly damaged reservoirs are in service for the 2011-2012 summer demand (albeit some are leaking and with reduced capacity at the Huntsbury site).

Repair and reinstatement

Design

Other than Huntsbury No.1, the reservoirs are generally all being reinstated without significant change to their storage capacity. Huntsbury is being reconstructed but with a significantly reduced capacity, refer Case Study 1.

Consistent with the damage observed, roof-to-wall repair and retrofit is the most common and this has typically comprised construction of concrete ring beams either attached at the top of the reservoir wall or to the roof. For damaged or vulnerable wall-to-base connections, concrete ring beams have been designed to provide a direct shear connection between the wall and foundation to improve overall structural performance and robustness. Severely damaged base-slabs have been reinstated with new fully continuous reinforced concrete overlays. Leaking joints are typically being bandaged.

Repair details for the five most significantly damaged reservoirs are provided in the Case Studies.

Status of repair and reinstatement

The ten most significantly damaged reservoirs are all in service and their status, at time of writing, is summarised in Table 3.

Table 3: Status of the ten most significantly damaged reservoirs (at time of writing)

Reservoir (CCC Condition Grading)	**Removed from service and reinstatement completed**	**CCC Condition Grading**	**Further reinstatement required**
Huntsbury No.1	✓ (Stage 1 reservoir & pump station)	5	✓ (Stage 2 reservoir)
McCormacks No.1	× (kept in service)	4	✓ (full repair req'd)
McCormacks No.2	✓ (Stage 1 repairs)	5	✓ (Stage 2 repairs)
Upper Balmoral	✓	4	×
Clifton 3	✓	4	×
Mt Pleasant No. 2-2	✓	3	×
Murray Aynsley	✓	3	Future relocation?
Mt Pleasant 4	× (kept in service)	3	✓
Monks Spur 3	× (kept in service)	3	✓
Clifton 4	× (kept in service)	3	✓

CASE STUDIES

The following reservoir Case Studies summarise damage observed, results of seismic assessments (where undertaken) and reinstatement details/concepts including commentary on the staging of repairs to maximise storage for the peak summer water demand.

CASE STUDY 1 - HUNTSBURY NO.1 RESERVOIR

Summary – structural details

- In situ reinforced concrete construction. Construction date circa 1953
- Rectangular geometry approximately 77.4 x 63 m in plan, 8 m clear internal height
- Storage capacity 35,000 m^3
- Location: approximately 3.0 km from the 22 February 2011 earthquake epicentre,
- Partially buried, cut into the crest of Huntsbury Spur
- *This reservoir was significantly damaged during the 22 February 2011 earthquake resulting in functional failure and complete loss of contents*.

Inspections and observations

Structural

The main structural damage was cracking of the base-slabs with crack widths of up to 35mm measured and movement of slabs both horizontally and vertically by up to 50mm (the internal areas of base-slab comprise a double slab constructed in offset individual slabs). This cracking, as indicated in Figure 1, had a distinctive pattern extending diagonally across the reservoir and over a zone of some 20 m width. A similar pattern of cracking was also observed in the roof. Discrete localised cracking was observed in the walls, along with opening of some vertical construction joints, and the central, low height, dividing wall was severely damaged at one location coincident with the base-slab diagonal cracking pattern. The adjoining pump station was also significantly damaged plus the main 600 mm diameter inlet / outlet pipe. Further mapping of the base-slab following the 13 June 2011 earthquakes and other significant aftershocks identified additional cracking and also continued movement of slabs horizontally and vertically.

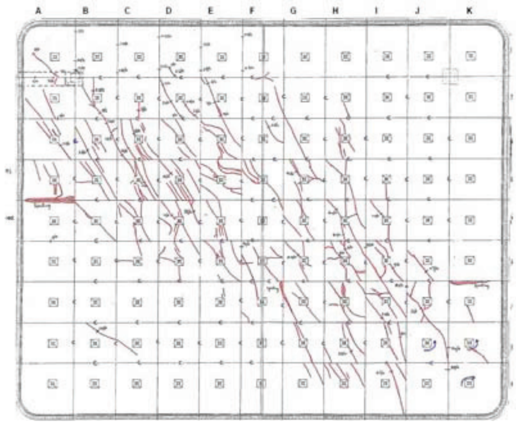

Fig. 1: Huntsbury No.1 Reservoir – base-slab cracking (22 February 2011 damage mapping).

Geotechnical

Extensive geotechnical investigation, including ground penetrating radar surveying, coring of vertical and inclined boreholes beneath the base-slab, trenching under the base-slabs, inspection of surrounding property and roads, and level surveying were undertaken at the Huntsbury site.

Observations noted similarly orientated cracking or fracturing, to that in the base-slab and roof, within the adjacent reservoir tunnel and roads either side of the spur on which the reservoir is founded.

Some further cracking and movement was also observed in roads adjacent the reservoir following the 13 June 2011 earthquakes, along with increased failure of the tunnel crown.

Commentary on findings

Investigations as to where 32,000 m^3 of water drained to have been inconclusive but it appears likely that a significant volume drained through the construction joints and cracks in the base-slab (and into the underlying fractured basalt) and, some drained through the mains pipes as the seismic actuated shut-off valves take time to close.

The distribution and orientation of cracking observed within the reservoirs structure and in the tunnel is indicative of a zone of shear movement, extending diagonally across the reservoir footprint, along planes of weaknesses in the underlying rock. This has been confirmed by the inclined boreholes and trenching carried out inside the reservoir. The 22 February 2011 earthquake has reactivated an underlying ancient shear zone and resulted in movement on planes of weakness in the rock. As further movement could occur in future similar events, and in view of the increased seismicity, it was assessed that the reservoir should not be reinstated over the shear zone.

A detailed structural seismic assessment was completed for the reservoir. Based on current design requirements it indicated vulnerabilities with the roof and wall construction joints for in-plane loading and that the walls were marginal for out of plane bending. However the cause of the damage is considered primarily attributable to movement in the underlying shear zone.

Reinstatement

As noted above, a reservoir reinstated or constructed over the shear zone was not considered a viable option. However, as the Huntsbury site remained critical in CCCs immediate and long term network water storage requirements a range of structural options were developed that considered the shear zone location and its risks. The final configuration adopted for reconstruction of the site is two significantly smaller reservoirs constructed in the corners of the current reservoir footprint as shown in Figure 2. These new reservoirs utilise lengths of the existing reservoir's walls and columns with additional new walls plus base-slab and roof overlays. Construction and commissioning of the Stage 1 6,200 m^3 first replacement reservoir was completed mid December 2011. Construction of the second replacement reservoir is anticipated to be completed in mid-2012. The estimated total storage volume at the completion of reconstruction is 13,600m^3, providing just under 40% of the reservoir's original capacity. A replacement pump house has been constructed remote from the reservoirs.

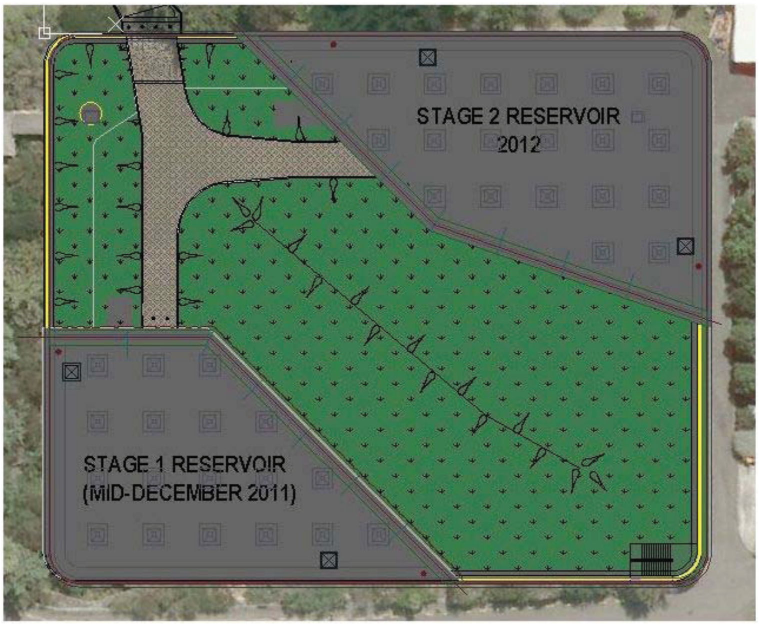

Fig. 2: Huntsbury No.1 Reservoir – proposed site reinstatement.

CASE STUDY 2 - McCORMACKS BAY RESERVOIRS No.1 AND No.2

Summary – structural details

- Precast wall panels, post-tensioned circumferentially, prestressed vertically. Construction dates 1984 (No.1), 1995 (No.2)
- Circular geometry 30 m internal diameter, approximately 8 m clear internal height
- Storage capacity 5,000 m^3 each
- Location: approximately 2.0 km from the 22 February 2011 earthquake epicentre
- Ground supported on platform cut into rock
- *Reservoir No.2 was removed from service following the February earthquake due to excessive leakage through wall joints and base-slab cracks. Reservoir No.1 remains operational but is leaking through wall joints, base-slab cracks and also through tendon anchorages.*

Inspections and findings

Structural

Damage noted to Reservoir No.2 includes; cracking in base-slabs, spalling and cracking around the wall foundation ring-beam, vertical and horizontal movement of wall relative to foundation ring beam, cracking or opening of wall construction joints, damage to roof-wall connections and minor internal column concrete spalling. Damage noted to Reservoir No.1 is very similar to No.2 but its roof is more severely damaged due to extensive failure of roof-wall dowel connections, and water appears to be leaking from the circumferential post-tensioning tendon anchorages. The resultant roof movement, once the dowels have failed, has smashed the overhanging roof nib, resulted in precast roof tee units engaging the wall and punching out wall concrete, and damaged webs in the tee units. The pump house between the two reservoirs also has structural damage including spalled concrete and cracking of wall and slab concrete. Refer Figures 3 and 4 for indicative reservoir structural damage observed.

Fig. 3: No.1 Reservoir – significant roof damage.

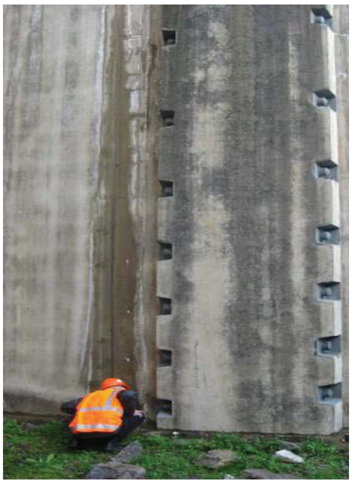

Fig. 4: No.1 Reservoir – leaking vertical wall joints.

Fig. 5: Access road – cracking and slumping.

Geotechnical

The access road to the reservoirs has suffered significant cracking, settlement of up to 500mm and has slumped towards the slope below the reservoirs, refer Figure 5. The stacked basalt block gravity wall supporting this road partially collapsed resulting in rockfall hazards to the road and residents below (emergency temporary rockfall fences were installed to mitigate short term rockfall risks). The original construction platform for the reservoirs was cut into rock and there is a near vertical face of up to approximately 21 m height behind the reservoirs. The 22 February and 13 June 2011 earthquakes caused significant loosening and dislodgement of material from the rock face. Rock and material has piled up behind the reservoirs and some rocks have likely impacted the reservoir wall.

Commentary on findings

The detailed seismic assessment identified vulnerabilities with the roof-wall connections (10-50%NBS), the wall to base connections (35-70%NBS), the walls (80-90%NBS) and a potential deficiency in resistance to sliding. These results are generally consistent with the observed damage.

Given the magnitude of horizontal accelerations experienced at the site on 22 February, the post-tensioning tendons may have been loaded slightly beyond their proof stress / limit of proportionality. The possible effect of this is that the residual wall compression under normal hydrostatic loading may have been reduced, and the reservoirs' residual lives compromised slightly.

Reinstatement

Structural repair and retrofit requirements are extensive and costly including; construction of a full base-slab overlay and internal ring beam to tie the wall in to the base-slab and to provide a direct seismic shear transfer mechanism, a ring beam around the top of the wall (No.2 reservoir) and bandaging of all wall vertical construction joints full height. Reservoir No.1 repairs are still being developed at the time of writing but expected to be similar.

Reinstatement work for the two reservoirs has been staged to assist with achieving maximum network storage for the 2011-2012 summer water demands. Stage 1 repairs to Reservoir No.2 were completed mid-December comprising; base-slab overlay and ring beam, and bandaging of wall joints up to around 2.0 m above slab level. These were the minimum repairs identified to achieve operation and this reservoir is now back in service but is leaking through wall joints above the part-height bandaging. Once water demand has reduced in the second quarter of 2012, Reservoir No.1 will be emptied to enable full repair to be undertaken followed by completion of the Reservoir No.2 repairs.

Geotechnical repairs at the site are also extensive and will continue till late 2012. A permanent piled replacement retaining wall is currently proposed to retain the edge of the access road. A grid of rock anchors and soil nails installed over the cut face behind the reservoirs is also currently proposed.

CASE STUDY 3 - CLIFTON 3 RESERVOIR

Summary – structural details

- In situ reinforced concrete construction. Construction date circa 1948
- Circular geometry, 14.4 m internal diameter, approximately 3.3 m internal clear height
- Storage capacity 455 m^3
- Location: approximately 3.7 km from the 22 February 2011 earthquake epicentre
- *Repair of damaged external roof nib undertaken following the February earthquake. Removed from service following the 13 June earthquake for engineered repairs and retrofit*

Summary - inspections, findings, repair and reinstatement

Fig. 6: Clifton 3 – internal column failure.

Fig. 7: Failure of roof overhanging nib.

Damage noted included failure of the internal column (Figure 6), sagging of the roof and shearing of the roof overhang at the roof-wall joint (Figure 7). Under horizontal earthquake loading the roof has impacted the top of the wall and resulted in its overhanging nib shearing off. Excessive translation of the roof subsequently occurred and resulted in the column failing due to only limited shear and flexural capacity. Simple repair of the roof nib was completed following the 22 February earthquake but this failed during the 13 June event.

Repair and retrofit included break-out and reconstruction of the damaged top and bottom of the column, jacking of the roof vertically and fitting of a reinforced concrete ring beam at the top of the wall to restrain the roof against lateral earthquake movement. A kerb has been constructed at the base of the column to provide additional restraint. Minor repairs to floor sealants and wall cracking were also required. The reservoir was returned to service late December 2011.

CASE STUDY 4 - UPPER BALMORAL RESERVOIR

Summary – structural details

- Precast 150 thick wall panels, post-tensioned circumferentially, conventionally reinforced vertically (single reinforcement layer with 40-45 mm concrete cover). Construction date 1986
- Circular geometry, 18.3 m internal diameter, approximately 4.0 m internal clear height
- Storage capacity 1,000 m^3
- Location: approximately 2.0 km from the 22 February 2011 earthquake epicentre
- *Repair of damaged roof-wall connections undertaken following the February earthquake. Removed from service following the 13 June earthquake for engineered repair and retrofit.*

Summary - inspections, findings, repair and reinstatement

Fig. 8: Upper Balmoral – concrete shear failure at pilaster locations due to roof beam impact.

Fig. 9: Upper Balmoral – concrete damage at roof beam wall support.

The roof comprises precast roof beams that slot into the reservoir wall and pilasters. Under horizontal earthquake loading the roof beams have impacted the wall and resulted in major cracking and damage to the top of the wall (Figure 8) and to the beams. At pilaster locations, the beams have punched off the wall concrete at the ends of the beams (Figure 9). Excessive roof movement has also resulted in damage to the top of the internal column.

The detailed seismic assessment identified vulnerabilities in the roof-wall connections (15%NBS), the internal column (50%NBS) and the wall to base connection (35-55%NBS).

Simple repair of the damaged wall sections that support the roof was completed following the 22 February 2011 earthquake but subsequently failed during the 13 June event. Engineered repair and retrofit included fitting of a reinforced concrete ring beam at the top of the wall (tied to the roof beams), break-out and reconstruction of the damaged top of the column plus wall and beam repairs. An internal ring beam has been constructed to tie the wall into the base-slab and to provide a more direct seismic shear transfer mechanism. The reservoir was returned to service late December 2011.

CONCLUSIONS AND LESSONS LEARNED

Christchurch City's concrete reservoirs within the Port Hills and Cashmere Hills are located very close to the epicentre of the 22 February 2011 earthquake and suffered damage varying from nil through to major. Approximately 75% of the reservoirs either did not require any repair or only required minor repairs. Substantial repair through to re-construction are required at five locations.

Horizontal acceleration response spectra from strong motion stations in the Port Hills area indicate that some reservoirs in this area are likely to have experienced accelerations greater than current relevant code implied design values.

Damaged roof-to-wall connections were observed in many reservoirs with damage to walls, base-slabs and internal columns limited to a few reservoirs only. Dowelled roof-to-wall connections have performed poorly and these types of details appear to have limited robustness.

Seismic assessments have been completed for the eight reservoirs that require the most significant repair and typical findings include; roof-to-wall and wall-to-base/foundation connection vulnerabilities, a potential deficiency in resistance to sliding and insufficient freeboard to roofs. It is likely that there are a number of reservoirs throughout New Zealand with similar vulnerabilities.

Observations and detailed assessments indicate that thin walled singly reinforced reservoirs, a feature of modern design and construct delivery, appear to have limited robustness.

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.

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

Repair and reinstatement of the most severely damaged reservoirs has been staged to maximise network storage capacity for the 2011-2012 summer peak demand. Storage has successfully been achieved, in time for this peak period, at the most significantly damaged reservoir sites.

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