Appendix E

CH2M Beca Hospital Prince of Wales Reservoir, Conceptual Design Options - 2013

Report

Hospital Prince of Wales Reservoir Conceptual Design Options

Prepared for Wellington City Council (Client)

By CH2M Beca Limited

1 February 2013

Volume 1 of 2 – Text and Appendices

Revision History

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Approved by	Mark Sneddon	pp	19 February 2013
on behalf of	CH2M Beca Limited		

Volume 1 of 2 – Report Text and Appendices

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Volume 2 of 2 – Drawings (separate document).

List of Abbreviations

Unless the context requires otherwise the following abbreviations and their meanings are used within this document:

Brief	WCC's Request for Tender Document
Capacity	Capacity Infrastructure Services
CBD	Central Business District
Council	Wellington City Council
c'	Cohesion
GW	Greater Wellington Regional Council
g	Gravity
HAZOP	Hazard and Operability Study
HPOW	Hospital Prince of Wales
NZS	New Zealand Standard
P&ID	Process and Instrumentation Diagram
RCRRJ	Reinforced Concrete Rubber Ring Jointed
RFT	Request for Tender
RMA	Resource Management Act
RP	Return Period
SLS	Serviceability Limit State
SSSHA	Site Specific Seismic Hazard Assessment
TBC	To Be Confirmed
TWL	Top Water Level
ULS	Ultimate Limit State
uPVC	Un-plasticised Polyvinyl Chloride
WCC	Wellington City Council
WE	Wellington Electricity
ϕ	Angle of internal friction

Executive Summary

1 CH2M Beca Limited (Beca) has been commissioned by Wellington City Council (WCC) to undertake concept and preliminary design works for the proposed Hospital Prince of Wales Reservoir which involves construction of a 35,000m3 reservoir to be buried within the Town Belt.

2 This report develops and assesses a number of concept options for the shape and location of the reservoir including:

a. Determining the optimum proportions of tanks for least cost with different shapes.

b. Undertaking a qualitative multi criteria analysis to rank the various concept options for a range of engineering, environmental and risk factors

c. Assessing the cost of all concept options with a breakdown of the recommended concept option including the variation of cost for designs based on different levels of earthquake return period.

3 A primary objective is to identify a preferred concept to be taken forward to preliminary and final design.

4 Also, in accordance with the brief, we have considered and included the following technical assessments within this report;

a. Reservoir piping and pipe tunnel configurations

b. Site specific seismic design considerations for structure and slope stability for a range of potential earthquake return periods and hence a range of potential seismic design standards

c. Reservoir failure mode and probable secondary flow routes with suggested mitigation methods

d. Basis of Design Statements setting out the proposed basis of the future detailed design with respect to structural, mechanical and geotechnical aspects.

5 The reservoir concept options considered were:

a. Rectangular and circular,

b. For circular reservoirs both a straight vertical wall with a flat roof (1:10 slope or flatter) and a variation where the vertical walls are surmounted with a conical roof (1:3 slope to 1:4 slope) which would in part be water retaining.

c. Single and multi-cell reservoirs which would allow the reservoir to be taken out of service while still providing partial capacity.

6 Multi-cell reservoirs were discounted because they are substantially more expensive due to the additional walls and, while they do deliver redundancy, this is not required by the brief.

7 The rectangular reservoir arrangements proposed involved insitu reinforced concrete construction and are a traditional arrangement. These reservoirs generally perform well although walls are more prone to cracking compared to a circumferentially post-tensioned circular reservoir.

8 The circular reservoir arrangements proposed involved either an in-situ reinforced or post tensioned concrete floor slab and roof slab construction with walls of reinforced precast panels circumferentially post tensioned.

9 The conical roof variation of the circular reservoirs would use precast circumferentially post tensioned panels for both the walls and the sloping sections of the cone. Beca are unaware of any precedent of this form of construction for the roof of water reservoirs in New Zealand although concrete dome roofs have historically been used on reservoirs within the Wellington region but the roof is not water retaining.

10 Beca consider that further preliminary design would be necessary to establish confidence in the ability to design such an arrangement, particularly with respect to the joint between the walls and the conical sections and between the segments forming the conical roof. These connections will need to be designed and detailed to not leak for normal service and after a major seismic event.

11 A costing model was developed and used to determine the optimal aspect ratio (height/base dimension) for minimum cost with circular and rectangular reservoirs.

12 Siting options were also addressed, with the reservoir being either on the centre of the ridge or towards the west adjacent an existing gully. The site adjacent the gully has been discounted as not feasible on geotechnical grounds and undesirable on environmental grounds.

13 Consideration was given to the steepness of the backfill against the walls of the circular reservoir form and options were considered with both normal compacted fill slopes and with steep slopes of reinforced fill. This was to evaluate the effect on reservoir backfill quantities and cost. Some variation of backfill slopes around the perimeter of the reservoir is expected to provide a best fit solution for the site.

14 On the basis of the optimal aspect ratio for minimum cost, the maximum plan dimensions for the reservoirs to fit the site, and floor and water level constraints, and siting and backfill options, six reservoir options were developed as follows

a. R1 Cylindrical reservoir sited on ridge with normal compacted fill

b. R1.1 Cylindrical reservoir sited towards gully with gully used for fill disposal

c. R1.2 Cylindrical reservoir sited on ridge with gully used for fill disposal

d. R2.0 Rectangular reservoir on ridge and normal compacted fill

e. R3.0 Cylindrical reservoir with truncated cone roof (1:3 slope) sited on the ridge.

f. R3.1 Variation of R3.0 with flatter conical roof (1:4 slope) and raised floor and roof levels compared to R3.0

15 Application of the relative cost model indicated there would be little difference in cost between rectangular and circular forms hence reservoir geometry was not a major cost factor but could influence the water tightness and seismic resilience.

16 Assessment of Options R3.0 and R3.1 on the basis of the reservoir requirements indicate that having permanent hydrostatic water load on the underside of the roof does not meet the viability criteria of a proven track record. Accordingly Options R3.0 and R3.1 were rejected.

17 A multi-criteria analysis for the viable options was undertaken involving qualitative assessment of four factors and separately on cost. The outcome was ranking in decreasing order of preference R1.0, R1.2, R2.0, R1.1.

18 As indicated in 12 above the gully site has been shown to be unacceptable and hence option R1.1 was not preferred.

19 The rectangular shape does not fit in well with the existing ground profile and has a less favourable form for seismic resilience and water tightness. Accordingly Option R2.0 was not preferred.

20 The use of the gully as a disposal site for excess excavated material is considered undesirable on environmental grounds. Accordingly Option R1.2 was not preferred.

21 As a result of the proceeding considerations Option R1.0 was identified as the preferred option.

22 A structural design criterion for R1.0 was assessed for costing purposes as a defined median level to allow consider lessor of greater or lesser seismic loading requirements. The structural design criteria were a Design Working Life of 100 years (exposure period); SLS2 Limit State return period seismic event of 1,000 years and ULS Limit State return period seismic event of 5,000 years.

23 A total project cost estimate was prepared for Option R1.0 based on the above structural design criteria. The cost estimate for Option R1.0 is $17.9M with an estimated lower bound of $17.0M (-5%) and an estimated upper bound of $19.7M (+10%). This is significantly greater than the current WCC financial allocation for design (as commissioned) and construction which we understand is $14.0M.

24 The report identifies the GW and WCC resource consents expected to be required for the options. A similar number of consents are required for all options. Landscape and visual effects during construction have been assessed as High for all options with ‘end use’ landscape and visual effects assessed as Low to High depending on the option. Option R1.0 is assessed as having a Low to Moderate degree of ‘end use’ landscape and visual effect.

25 As required by the brief the report addresses the physical modifications and resulting cost variations associated with reducing the design serviceability limit state (SLS2) earthquake return period (for operational continuity) from 1,000 years to 500 years and increasing it to 1,500 years. The Design Working Life for a SLS2 500 year return period earthquake is reduced to 50 years and increased to 150 years for a SLS2 1,500 year return period earthquake.

Modifications to the design will primarily involve a pro rata decrease or increase in the reservoir wall thickness with subsequent marginally less or greater level of reinforcement and post tensioning. This would result in an overall cost saving (compared with the 1,000 year SLS2 return period earthquake) of $0.5M for a 500 year return period earthquake and an increase of $0.5M for a 1,500 year return period earthquake.

26 Design Working Life for concrete durability of the structures has been considered as 100 years for all options.

27 The separate and referenced report “Hospital Prince of Wales Reservoir Geotechnical Report” Rev B ‘Final’ dated 3 October 2012 has concluded that the proposed reservoir platform will remain stable during the ultimate limit state earthquake derived in the separate referenced report “Hospital Prince of Wales Reservoir Seismic Hazard Assessment” Rev B ‘Final’ dated 21 December 2012.

28 Consideration has been given to the possible mechanisms of failure of pipework associated with the reservoir during a seismic event, the potential rate of water loss that could result, the flow paths the water would take, and the possible impacts.

29 The conclusion was that the likely mode of failure would be at the reservoir/outlet pipe interface and any flow would be routed down Rolleston Street in a combination of underground stormwater drains and surface overland flow . Such a failure is considered unlikely and will be mitigated against by the use of resilient pipework and the incorporation of a seismic activity actuated valve with a backup manually operated valve. In the unlikely event these were all to fail it is assessed that flows of the order of 1,200 litres/second might occur down Rolleston street, similar to the design reservoir overflow discharge rate.

30 As required by the brief consideration was given to options for the location and configuration of the pipe-work tunnel. The identified preferred option is a single tunnel located below the floor level of the reservoir oriented towards Rolleston Street and with an access door in the side of the tunnel to minimise visual effects.

1 Introduction

1.1 Background

Wellington City Council (WCC) are seeking to construct a completely buried 35,000m3 concrete reservoir within the Upper Prince of Wales Park in Mount Cook (see Figures 1.1 and 1.2) to service the Wellington Hospital and Central Business District. The facility will have a special post disaster function to supply water for the Wellington Regional Hospital and therefore is required to have an equivalent operational continuity limit state design seismic return period event as the Wellington Regional Hospital (1000 year return period).

The reservoir form and location selected are required to recognise the sensitivity of undertaking the required construction works within the Town Belt including environmental considerations.

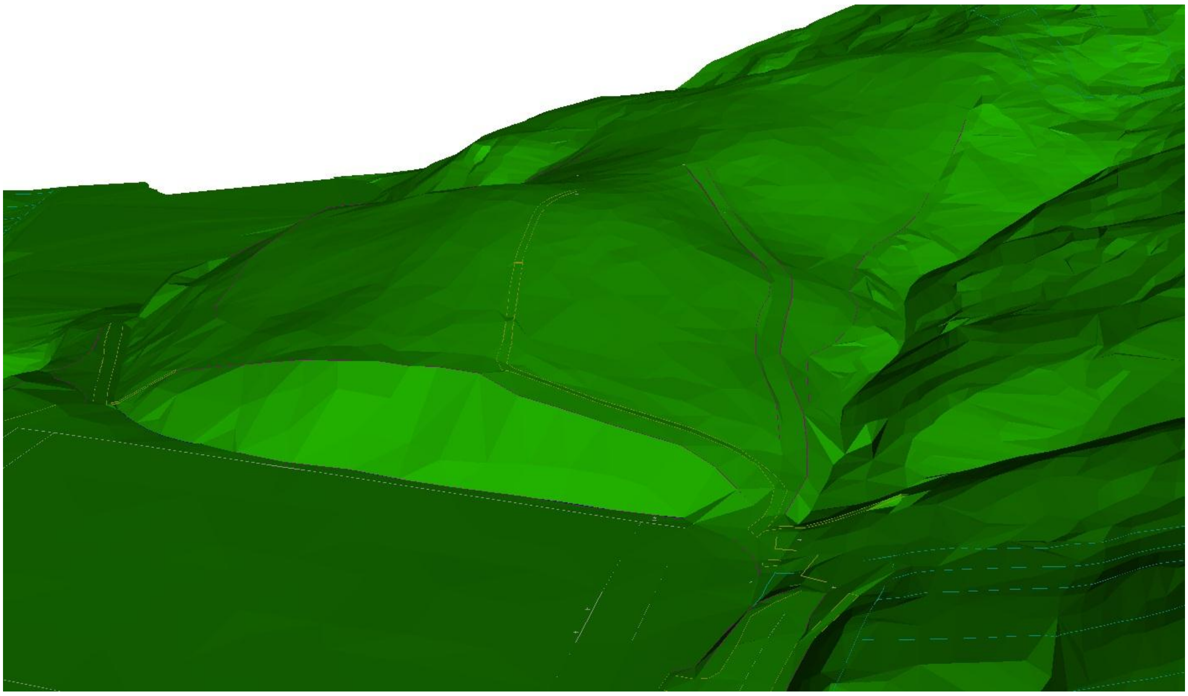
It is understood that the WCC programme is to design, construct and commission the new facility, including connections to the existing water reticulation system by the third quarter of 2014.

CH2M Beca Limited (Beca) has been commissioned by WCC under the Capacity Infrastructure Services Request for Tender for the Consultancy Services for the Hospital Prince of Wales Reservoir (WCC, 1 February 2012) (the brief).

Figure 1-A: Proposed Reservoir Site Location

Figure 1-B Aerial View of Prince of Wales Park

Figure 1-C: Existing Site Profile of Upper Prince of Wales Park Looking South Across the Proposed Reservoir and Pipe Tunnel Site



1.2 Purpose and Scope of Report

The purpose of this "Conceptual Design Options Report" is set out in the brief. A primary objective is to develop and evaluate a number of reservoir concept options with due consideration of maintenance, operation and cost factors and identify a preferred concept to be taken forward to preliminary and final design.

Also, in accordance with the requirements of the brief, the following technical assessments are included within this report

- Reservoir piping and pipe tunnel configurations.
- Site specific seismic design considerations for structure and slope stability for a range of potential earthquake return periods (by reference to other reports).
- Reservoir failure mode and probable secondary flow routes with suggested mitigation methods,
- Basis of Design Statements setting out the proposed basis of the future detailed design with respect to structural, mechanical and geotechnical aspects.
- The cost estimates for the total reservoir project (within the WCC defined scope) at the completion of Concept Design.
- The cost implications for providing seismic strength of the reservoir above and below a base case of 1,000 year return period earthquake for SLS2 operational continuity serviceability limit state. The 1,000 year SLS2 return period event matches the Wellington Regional Hospital design standard for operational continuity serviceability limit state.
- This report does not include assessment of options and definition for:
- Landscape design (which will be addressed in the Preliminary Design Report).
- Methodology for reservoir water circulation and mixing (which will be addressed in the Preliminary Design Report).
- Construction methodology and programme (which will be addressed in the Preliminary Design Report).

- Concrete durability greater than 100 years (Options for enhanced durability to be addressed in the Preliminary Design Report).
- Pipework Tunnel ventilation (which will be addressed in the Preliminary Design Report)
- Instrumentation and controls including telemetry systems (which are a Greater Wellington Regional Council (GW) responsibility).
- Power distribution for controls, equipment and lighting downstream of a switchboard and metering enclosure located inside the pipe tunnel (which is a GW responsibility).

The report does not consider or include in cost estimates elements related to the project which are outside the limit of the WCC defined scope, for example water reticulation piping past a connection point to the reservoir site from Hargreaves Street.

1.3 Project Components

The scope of the physical works for this project is as follows:

- A new 35,000m^3 completely buried reinforced/prestressed concrete reservoir together with inlet, outlet, overflow and scour pipework, and drainage pipework installed partially in a pipe tunnel and in-ground trenches;
- Potential raising the ground level of the Upper Prince of Wales park sports ground and associated works in order to dispose of excavated materials;
- Subsequent use of the Upper Prince of Wales Park sports ground area for temporary storage of material followed by construction of a properly drained and surfaced sports ground;
- Disposal of excess excavated materials off site;
- Landscaping resulting from vegetation disturbance over and around the reservoir including the adjacent sports ground.

The current WCC intention is that the project will be constructed under a competitively tendered construction contract or contracts depending on the outcome of the design process, with pipe, valve and flange procurement possibly undertaken directly by Capacity.

1.4 Associated and Referenced Reports

This report references and draws on information and conclusions from the following documents:

- Capacity Infrastructure Services Request for Tender for the Consultancy Services for the Hospital Prince of Wales Reservoir (WCC, February 2012)
- Hospital Prince of Wales Reservoir Park and Surplus Material Options Assessment Report (Beca, Rev B ‘Final’ dated 26 September 2012)
- Hospital Prince of Wales Reservoir Seismic Hazard Assessment (Beca, Rev B ‘Final’ dated 21 December 2012)
- Hospital Prince of Wales Reservoir Geotechnical Report (Beca, Rev B ‘Final’ dated 3 October 2012)
- Hospital Prince of Wales Reservoir Geotechnical Report – Addendum (Beca, Rev 1 dated 14 January 2013)
- Hospital Prince of Wales Reservoir (Stage 1 to 3) Risk Analysis (Beca August 2012)

1.5 Appendices

In addition to Appendices A and B which are referenced in the text, Appendices C – E (which are not referenced in the text) provide, as required by the Client brief, Basis of Design Reports for Structural, Mechanical and Geotechnical aspects of the future preliminary and detailed design.

1.6 Abbreviations

A list of abbreviations is provided immediately following the Table of Contents.

1.7 Acknowledgements

The project has been developed in close co-ordination with the GW who are responsible for the supply and delivery of bulk water to the proposed WCC owned HPOW Reservoir.

WCC provided a cost estimate for the design, construction, installation, and commissioning of the electrical instrumentation and controls, which are a GW responsibility. These costs have been included in the concept design estimates in this report.

WCC Parks and Garden have provided comments on landform and vegetation quality at the site.

2 Reservoir Configuration Options

2.1 Background

The previous design reports by others for the proposed reservoir have indicated that a single compartment circular reservoir provides the most technically suitable and economical solution within the assumed constraints.

The Parks and Surplus Material Options assessment required the development of reservoir configuration options, and it became clear that multiple reservoir options were not viable. This was due to site footprint extent preventing burial of the reservoirs without extensive contour alteration. Hence multiple reservoir options were discounted.

2.2 Site Constraints

The site constraints are:

- Top Water Level (TWL) = 92.0m to match operational top water levels of other reservoirs in the area.
- Maximum water depth = 15.0m. Beyond this depth the excavation volumes and reservoir wall thickness increases and the cost becomes uneconomic.
- Plan diameter/length = 75m. This is the maximum reservoir plan dimension that can reasonably be buried on the site given the above TWL.

2.3 Reservoir Arrangements Options

Readers are referred to Volume 2 – Drawings which requires reading in conjunction with the option descriptions below.

Drawing CE-K20 shows the location of all reservoir options in relation to each other and ground contours, all as proposed in the Park and Surplus Material Options report. The term “conventional roof” used in the description below means a concrete roof with minimal slope (1: 10 or flatter) to affect roof drainage but is not water retaining.

2.3.1 Reservoir Option – Circular, R1.0 Centrally Located, Conventional Roof

This option involves a circular and centrally located reservoir as show on Drawings CE-K02 and CE-K03. The reservoir has an external diameter of 67.7m and wall eaves height (floor level to top of roof) of 11.8m. The subgrade level for the excavation is assumed as 81.0m. Reservoir floor level is nominally 82.0m. Water depth is nominally 10.0m.

The front curved wall of the reservoir (facing the Upper Park) has generally been "centrally located" to approximately align with the shape of the existing contours. For comparison purposes, the cross sections shown on Drawings CE-K03 identify two fill slopes as follows:

- 1V : 2H which are dressed with 100mm of topsoil.
- 1V : 0.36H which is a reinforced slope using geotechnical fabric layers.

2.3.2 Reservoir Option - R1.1, Circular, Offset Located, Conventional Roof

Details of this option are as for Option - R1.0 except the location of the reservoir has been moved approximately 15.5m to the west towards the existing gully. This results in:

- The use of more favourable fill slopes around the reservoir.
- The use of the gully for the disposal of surplus excavated material,
- Some variation in subgrade conditions for the support of the reservoir foundations, and
- Removal of a large extent of rejuvenated bush, disturbance of an existing drainage path and sewer pipeline.

Details of this option are shown on Drawings CE-K04 and CE-K05.

2.3.3 Reservoir Option – R2.0, Rectangular with Circular Ends, Conventional Roof

This option involves a rectangular reservoir with circular ends (overall length of 100m x width of 40m) and of similar height to Reservoir Option R1.0.

The width of the reservoir has been reduced from that used for Reservoir Option R1.0 to allow the use of more favourable fill slopes (more stable and not reinforced) generally in the eastern and western directions. The front curved wall of the reservoir (facing the Upper Park) has generally been "centrally located" to approximately align with the shape of the existing contours. Details of this option are shown on Drawings CE-K06 and CE-K07.

2.3.4 Reservoir Option – R3.0, Circular, Centrally Located, Truncated Conical Roof

This option involves a circular reservoir as shown on Drawings CE-K08 and CE-K09. The reservoir has an external diameter of 68.0m and wall height of 7.0m and with a truncated conical roof.

Water depth is nominally 13.0m resulting in an overall reservoir height of 15m from the bottom subgrade level of 78.5m. This option has a similar diameter to Option R1.0. The roof profile is kept below the existing site contours and allows the reservoir to be covered with a soil profile which more closely follows the existing ground contours than Reservoir Options R1.0, R1.1 or R2.0. The profile of this option is also shown as a dashed outline on Drawing CE-K16.

2.3.5 Review by Parks and Gardens

The Council Parks and Gardens Business Unit was consulted on the above arrangement options and expressed concern that the existing landform would be noticeably modified by each option. A further option R3.1 was therefore developed which reduced modifications to existing ground contours to a "best fit" profile.

2.3.6 Reservoir Option – R3.1, Centrally Located, Truncated Conical Roof

This option is shown on Drawings CE-K15 and CE-K16. As noted above, the dashed outline indicates Option R3.0 and demonstrates the differences between Options R3.0 and R3.1. Note that the roof of both options is below top water level. The roof therefore experiences water pressure from the underside, a situation which is an unusual loading and water-tightness situation for a concrete water retaining structure.

2.3.7 Reservoir Option R1.2, Centrally Located, Conventional Roof

During a review of the Park and Surplus Materials Options Assessment report, Opus International Consultants Limited suggested an option which is Option R1.0 but with spoil disposed into the gully to the west of the site. This option is labelled Option R1.2 in this report but has not been identified on the drawings. From an arrangement perspective it is however identical to Option R1.0.

2.4 Cost Optimisation

Cost optimisation was carried out by assigning cost indices to the floor, wall and roof elements as follows;

- Reservoir floor index: 1.0 per m^2 (base cost)
- Reservoir walls: quadratic interpolation between indices values: (up to 10m high = 1.5 per m^2), (10m – 15m high = 2.5 per m^2, and 15.20m high = 4.0 per m^2)
- Reservoir roof index: 2.0 per m^2

The cost index reflects the *relative* cost of the reservoir elements based on construction costs per element. The optimisation was carried out for circular and rectangular reservoirs against a single variable termed the “Base Dimension”. This Base Dimension is the diameter in the case of a circular reservoir and the length of one side of the equivalent square plan area for a rectangular reservoir.

Figures 2.1 and 2.2 show the cost indices for rectangular and circular reservoirs. The summation of the cost indices for the three elements is plotted against the Base Dimension (diameter or length) and water depth respectively. Each reservoir option has been plotted in the figures for reference, and the optimum dimensions for rectangular and circular reservoirs shown.

The plots demonstrate there is very little difference in cost between the rectangular and circular reservoirs for this capacity of reservoir.

2.5 Configuration Options Summary

As demonstrated in section 2.4, the circular and rectangular reservoirs result in similar costs for the reservoir structure.

However circular reservoirs have a clear advantage in conforming to the existing landform, in particular Options R1.0 and R3.1.

The Option R1.0 reservoir is a conventional structural design, whereas Option R3.1 is non-conventional as the underside of the truncated roof experiences water pressure from within the reservoir. The Option R3.1 option therefore present greater risk in design, construction and operation than Option R1.0.

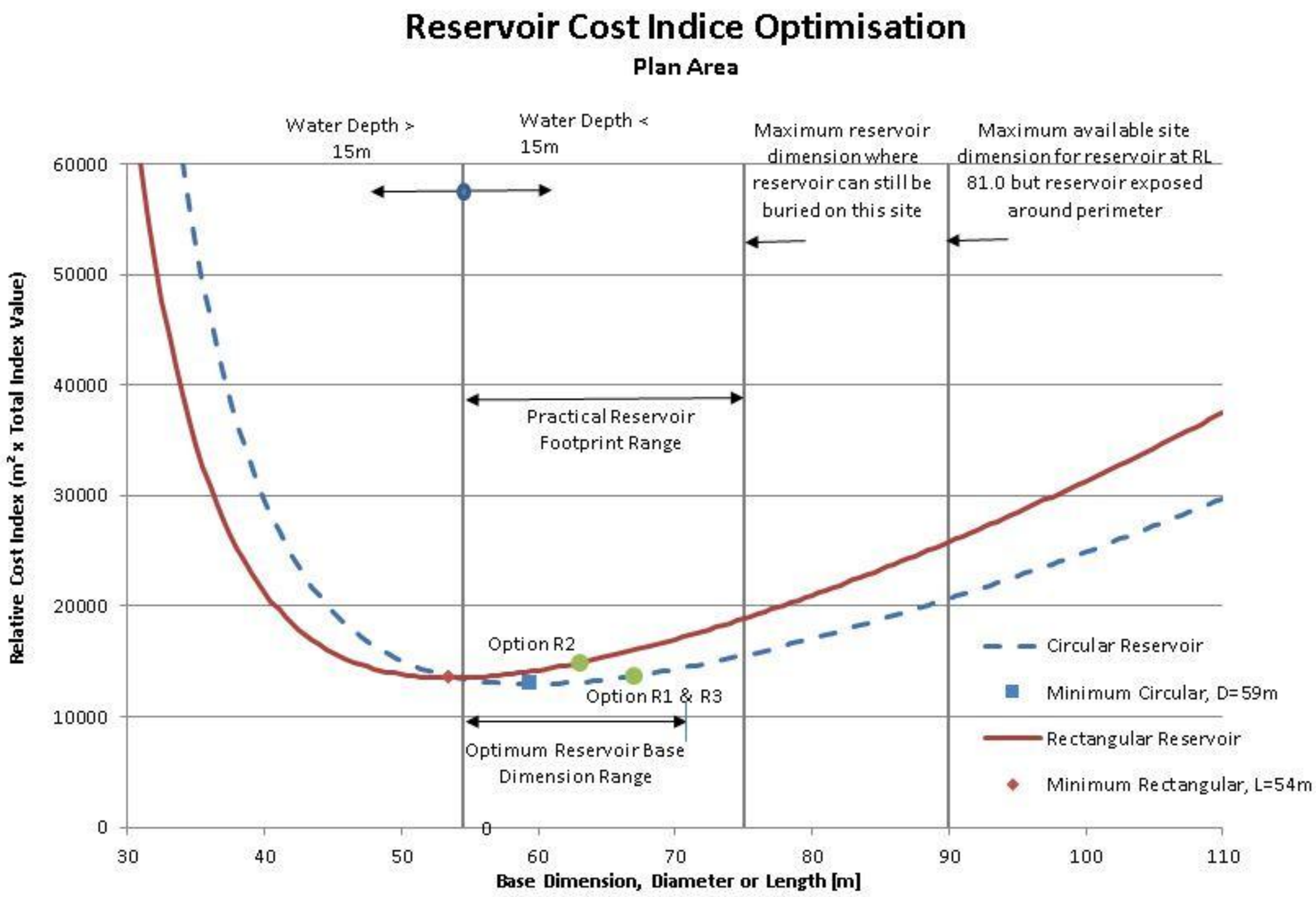


Figure 2-A Reservoir Cost Index Optimisation - Base Dimension

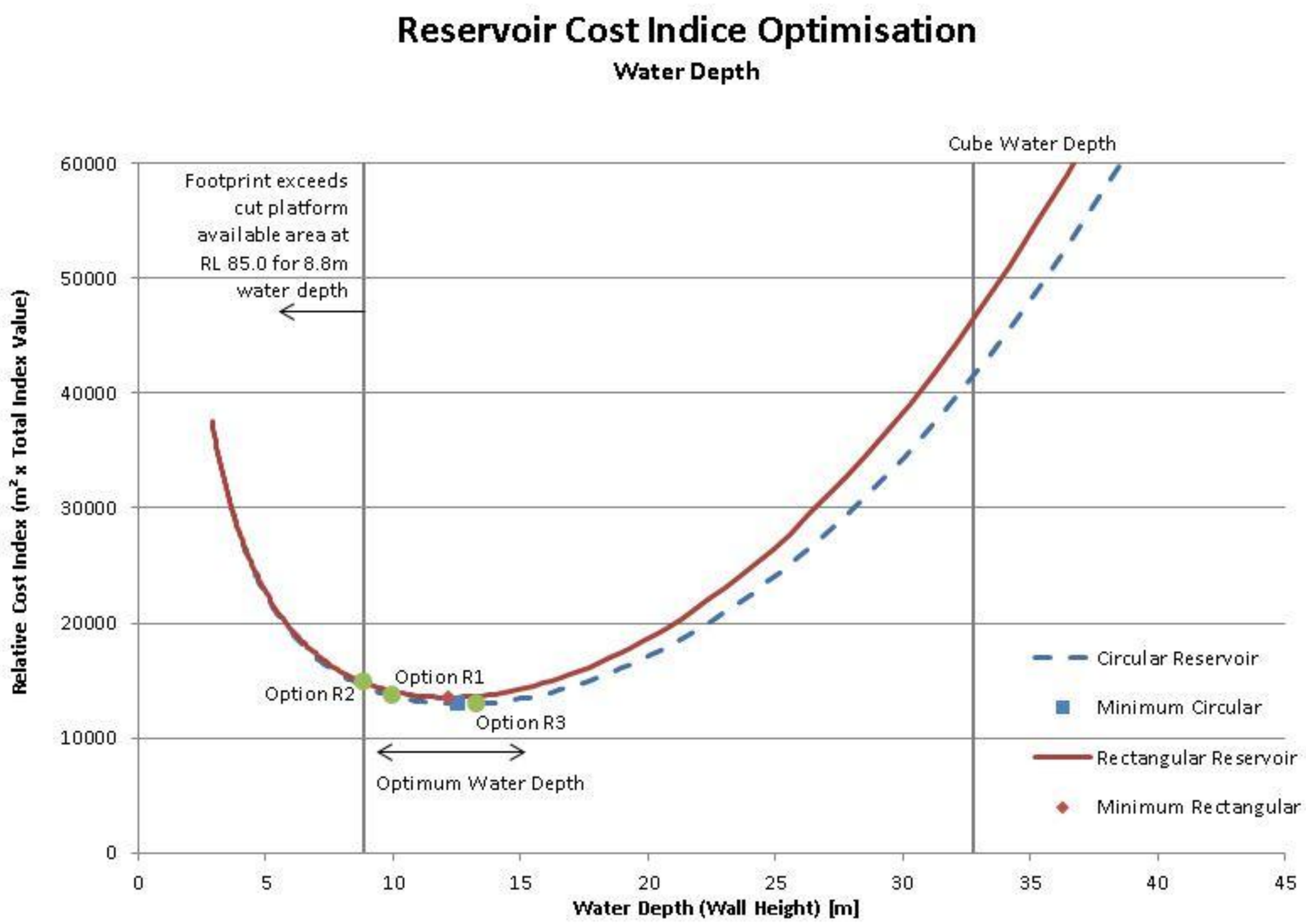


Figure 2-B Reservoir Cost Index Optimisation - Water Depth

3 Pipework Tunnel Options

3.1 Piping Configuration

Preliminary definition of the piping configuration for the reservoir that is to be included in the proposed pipe tunnel has been provided by WCC. Piping includes high and low pressure inlets, outlets, overflow, scour and a ducted air vent from inside the reservoir. The pipe tunnel will also include electrical switchboards and instrumentation and control equipment.

A ventilation system to the pipe tunnel is also required which will be addressed as part of the preliminary design stage.

A Piping and Instrumentation Diagram (P & ID) included in Volume 2 has been developed by WCC and Beca to define the preferred solution for the reservoir piping control and operational requirements.

Conceptual options for the layout of the pipe tunnel have been developed to determine optimum configuration.

3.2 Piping Tunnel Arrangement Options Considered

The following variables were considered to produce options for the pipe tunnel:

- Position of tunnel on site
- Dimensions of the tunnel for cross section and length
- Single or dual tunnels to match pipe routes for the inlet/outlet and scour/overflow pipes
- Please refer to drawings in Volume 2.

The pipe tunnel positions considered are shown on drawing NM-K01 These positions are approximately located North-East (A), North (B) and North-West (C) of the reservoir. In all cases the tunnel has been designed to extend under the floor of the reservoir for pipe penetrations to be made. This configuration avoids penetrations through the reservoir walls which are difficult with large bore piping and congested wall reinforcing. The underfloor penetration will also allow inspection of the pipe spools up to the floor penetration as they will remain accessible.

WCC may elect to provide a personnel access hatch through the floor of the reservoir at the end of the tunnel adjacent the pipe penetrations as shown on the Drawings. The provision of this low level reservoir access via either the wall or floor has been requested by WCC. This will allow easy personnel access to the interior of the reservoir when the tank is empty without the fall hazard of a 10m ladder from the reservoir roof. This hatch will be subjected to the head of water in the reservoir. Design, procurement and installation of this hatch is estimated at $85,000 (including estimating contingency, P and G, offsite overheads, profit, engineering and contract contingency). Reservoir roof access hatches and an access ladder will be provided as per WCC requirements.

The tunnel configurations considered were:

1. Option 1 - Single level: all inlet and outlet pipelines at a single level inside the pipework tunnel. This configuration is shown on drawings NM-K03, and K04. The advantage with a single level tunnel is that there are no falling hazards. The main disadvantage is that the width of the tunnel is with clear access space around the pipes is significant; the preliminary layout shown in NM-K03 is over 9 metres clear internal width.

2. Option 2 - Dual levels: half of the inlet and outlet pipelines are located on an upper level and the other half on a lower level. This configuration is shown on drawings NM-K05 and K06. This results in a narrower width and increased height for the pipework tunnel with a generally square cross section. From a structural point of view this has advantages for resisting earth pressures. It does require more access stairs and platforms between levels and creates potential maintenance difficulty and an inherent falling hazard at these access locations.

3. Option 3 – Separate tunnels: The pipework is located in two separate tunnels. This configuration is shown on drawings NM-K07 and K08. This replicates the narrower inset to model Options 1 and 2 structure of the dual level option but without a falling hazard and aligns the pipes to either Rolleston Street for overflow and scour and Hargreaves Street for inlet and outlet. The major disadvantage with this option is the requirement for an additional structure.

The details of the pipes, valves and fittings to be used in the pipework tunnel are defined in the Mechanical Basis of Design Report in Appendix D.

3.3 WCC Feedback on Preliminary Tunnel Arrangement Options

Preliminary feedback from WCC on the pipe tunnel Options 1 to 3 inclusive was a direction to reduce the width of the tunnel by removing the bypass valves (which were required by the brief) around each of the 600NB isolation valves. This alteration reduced the horizontal spacing requirements for the pipework for all options.

Further feedback from WCC received on 22 June 2012 included a mark-up of the P&ID (refer drawing NR-002 in Volume 2) These comments suggested moving a number of valves and instruments, and a review of the size of the scour line. Drawing NR-002 shows the repositioned valves, and scour line reduced from 600NB to 500NB. This has a minor but positive effect on the pipework tunnel width.

WCC also suggested that the modulating flow control plug valves could be reduced in diameter in order to allow more reliable flow control at lower flow rates. This would require reducers either side of the valves which increases the overall length of the pipe tunnel. WCC have now confirmed that plug valves are to be 600mm diameter. Hence reducers will not be required on the 600mm diameter low pressure inlet and outlet pipelines.

3.4 Options Selection

The dual tunnels option has been discounted due to the additional cost of building an additional structure.

Splitting the pipework between two floors in the pipework tunnel has also been discounted. Having two floors creates an inherent fall hazard and has a higher cost associated with it than a single floor.

Tunnel positions A and C on drawing NM-K01 have been discounted. These options ran approximately North-East and North-West respectively. Both options require either longer access roads across the upper playing field to the tunnel entrance or disturbance of the existing gully vegetation. The remaining tunnel alignment option, option B, runs North towards Rolleston St. Options A and C were considered to be more visible where they day lighted above ground than Option B. All tunnel alignment options are cost-neutral. The final orientation of the pipework tunnel is expected to be approximately as per Option B but could be rotated slightly to better fit with the proposed excavation plan.

It has been proposed that the entry to the pipework tunnel be on the side rather than the end of the tunnel. This allows the end to be covered and thus limits the angles from which the entrance can be seen.

3.5 Functional Requirements

Access is to be provided both for regular maintenance/inspection and in an emergency. Access is to include vehicle access to the door of the reservoir to allow delivery of heavy valves or other components.

The pipe tunnel is to allow for inspection, maintenance and replacement of the pipework up to the penetration of the pipes through the reservoir floor. WCC have indicated that they do want to proceed with a reservoir floor access hatch inside the pipe tunnel. This will allow inspection of internal pipework and the interior of the reservoir structure.

The reservoir will be fitted with an air vent discharging via the end of the pipework tunnel. The pipe tunnel shall have provision for a floor drainage system capable of removing water at the full inlet flow rate of the reservoir (1,200 l/s). This is to direct a potential uncontrolled flow into the pipe tunnel to Rolleston Street in the event that a pipeline rupture occurs inside the tunnel.

The piping in the tunnel is to incorporate flexibility to allow for movement of the reservoir structure during a seismic event without putting excessive load onto the reservoir floor pipe penetrations. This seismic isolation of the pipework to the reservoir structure is to be achieved by flexible couplings in the piping at either end of the tunnel. These couplings will be designed to allow movement of the pipework sections inside the tunnel relative to both the reservoir and the buried pipework outside the tunnel. There will be a seismically triggered isolation valve (flow control plug valves) and a manual valve on reservoir pipework immediately adjacent the reservoir to preserve reservoir contents in event of an earthquake.

The structure is a potential target for vandalism. This will largely be avoided by having the majority of the structure buried.

3.6 Pipe Tunnel Recommendations

The preferred option, based on the preceding analysis, is a single tunnel (configuration Option 1) located below the floor level of the reservoir orientated towards Rolleston St (alignment Option B) fitted with an access door on the side of the tunnel. This preferred option is shown on drawings NM–201 to 205 inclusive.

4 Seismic Design

Damage sustained by reservoirs in Christchurch is a timely reminder of the importance of resilience of reservoir infrastructure. The principal potable water storage facility servicing Christchurch, the 35,000 cubic metre Huntsbury No 1 reservoir, was damaged to the extent that all water stored prior to be earthquake was lost. The damage was due to a previously undetected “shear zone” beneath the reservoir, with the land portions each side of the zone moving horizontally and laterally relative to each other.

Other smaller Christchurch reservoirs were also damaged, predominantly at roof level, and particularly at the roof/wall junction. Unlike Huntsbury No 1 where damage was principally due to land displacement, at other locations reservoir damage was due to shaking of the structure itself.

The seismic design criteria discussed in this section are related to level of shaking. Potential land displacement issues are discussed in section 4.3 Geotechnical Considerations.

4.1 Seismic Hazard Scenarios

The brief required consideration of 100 and 150 year design life periods, in conjunction with 500, 1000 and 2500 year return period seismic events for the serviceability limit state, and a 2500 year return period for the ultimate limit state. The reservoir facility is an Importance Level 4 structure in terms of the New Zealand Loadings Standard (AS/NZS 1170.0:2002) due to WCC's requirement for it to be available for service immediately following a disaster.

Table 4-A below (extracted from the Seismic Hazard Assessment Report) shows the probability of exceedance prescribed in NZS 1170.0:2002 for an Importance Level 4 structure with a 50-year working life. Note that probability of exceedance is not specified in NZS 1170.0:2002 for Importance Level 4 structures with design working lives longer than 50 years.

Table 4-A 50-year working life structure probability of exceedance (NZS1170.0)

	Design working life (years)	Annual probability of exceedance	Probability of exceedance during design working life (per cent)
SLS2	50	1/500	10 %
ULS	50	1/2500	2 %

The probability of exceedance shown in Table 4-A represents the minimum design requirements for each limit state according to NZS 1170.0:2002. The return period of the seismic event for structures with 100 and 150-year design working lives to achieve the same probability of exceedance as for a 50-year design working life structure are shown in Table 4-B.

Table 4-B Required return period seismic event for 100 and 150-year design working life structure to obtain the same probability of exceedance as for a 50-year design working life structure

Case	Design working life (years)	Probability of exceedance (per cent)	Return period of seismic event (years)
SLS2	100	10 %	950 (1000)
ULS	100	2 %	4950 (5000)
SLS2	150	10 %	1425 (1500)
ULS	150	2 %	7425 (7500)

Note: The numbers in the parenthesis are rounded return period.

Therefore to address the large return period shaking required to achieve an equivalent probability of exceedance as in NZS1170.0:2002 the report presented recommended spectra for 5000 and 7500-year return periods as well as the 500, 1000 and 2500-year return period spectra specified by in the brief.

For this concept design report the following seismic design criterion have been adopted for concept design and the preparation of cost estimates.

- Design Working Life — 100 years
- Importance Level — 4
- Serviceability Limit State 1 (SLS1) Return Period Seismic Event — 25 years

(The structure and the non-structural components do not require repair after the SLS1 earthquake)

- Serviceability Limit State 2 (SLS2) 1000 years
  Return Period Seismic Event
  (The structure maintains operational continuity after the SLS2 earthquake)
- Ultimate Limit State (ULS)
  Return Period Seismic Event 5000 years

Note that the preliminary calculations indicate that SLS2 Limit State return period seismic event of 1000 years governs the structural design requirements, not the ULS seismic event. For comparison purposes, assessment of seismic loading for 500 year (50 year Design Working Life) and 1500 year (150 year Design Working Life) SLS2 Limit State return period seismic events has been carried out. Costs for design to the lower and higher seismic standard have been prepared are included in Section 10 Cost Estimates.

The selection of seismic design criterion to be used for the reservoir is to be made by Capacity.

4.2 Derivation of Total Seismic Load

The horizontal seismic design load (the base shear) applied to a reservoir and contents via its foundation is expressed as a proportion of the weight of the structure and contents. This proportion is defined as the horizontal design action coefficient - refer NZS 3106:2009, Design of concrete structure for the storage of liquids. The evaluation of the design action coefficient takes into account location specific criteria including seismic zone hazard factor for the region, proximity to faults (near fault factor), spectral shape factor for the site, and subsoil conditions.

The spectral shape factor varies according to the period of vibration of the structure, and the near fault factor varies due to structures period of vibration and distance to a fault. These are presented in the site specific Seismic Hazard Assessment Report.

Structure performance factor, ductility and damping are also considered in the evaluation of the coefficient.

The value of the design action coefficient increases for increasing return earthquake periods.

For a nominated site and assuming one period of vibration is under consideration, the coefficient varies in accordance with the return period factor.

Table 4-C extracted from the Seismic Hazard Assessment report shows the variation of return period for a range of annual probabilities of exceedance.

Table 4-C Return Period Factor, R

Required annual probability of exceedance	Equivalent return period (years)	R_s or R_u
1/250	250	0.75
1/500	500	1.00
1/1000	1000	1.30
1/1500	1500	1.45
1/2500	2500	1.75
1/5000	5000	2.10
1/7500	7500	2.30

As the loads the reservoir will experience vary in direct proportion to the return factors, the implication on selection of design life and the annual probability of exceedance become apparent.

Table 4-D below illustrates the variation in seismic horizontal accelerations for serviceability and limit states for design lives of 50, 100 and 150 years for a circular reservoir.

Table 4-D Seismic Horizontal Accelerations $C_d(T_i)$ for Different Limit States & Return Periods

Limit State	Design Working Life (years)	Equivalent return period (years)	Seismic Horizontal Acceleration for Circular Reservoirs	
			Impulsive $C_d(T_i)$	Convection $C_d(T_c)$
SLS2	50	500	0.91	0.034
SLS2	100	1000	1.19	0.04
SLS2	150	1500	1.33	0.05
ULS	50	2500	1.41	0.06
ULS	100	5000	1.69	0.07
ULS	150	7500	1.85	0.08

The main implication of increasing return period is increasing thickness of reservoir walls. The connection capacity required at the wall/floor and wall/roof interfaces will also increase.

In evaluating cost implications wall thickness has been adjusted in proportion to the horizontal acceleration Cd (Ti) values.

4.3 Geotechnical Considerations

The Hospital Prince of Wales Reservoir Geotechnical Report (Beca, Rev B “Final” dated 3 October 2012) concluded that;

- The risk of fault rupture affecting the reservoir is considered to be extremely low, with the nearby Lambton Fault being classified as Inactive and with no proven movement in the last 100 000 years, and
- Analyses indicated the reservoir platform would be stable under the predicted ground acceleration for a 7500 year return period which is the ULS event for a 150 year design working life.

Accordingly it is considered that geotechnical factors will not influence the seismic performance of the reservoir.

5 Geotechnical Considerations – Non Seismic

A summary of the geotechnical parameters for the civil and structural design of the reservoir have been incorporated in the Geotechnical Basis of Design in Appendix E.

6 Stormwater & Overflow Drainage & Potential Overflow/Failure Secondary Flow Routes

The stormwater concept design covers the following areas:

- Drainage of the reservoir site around the existing spur catchment
- Upper Park sports field drainage,

- Upgrade of Rolleston Street drainage; and,
- Overflow/failure related secondary flow routes.

6.1 Drainage of Reservoir Site around the Existing Spur Catchment

The drainage of the land area above and around the reservoir itself is based on Option R1.0 and is considered to be relatively straight forward as the reservoir will be buried and the land above it reinstated with landscaping similar to the existing vegetation. This will mean that there is effectively little or no change from the existing surface characteristics to increase runoff and therefore require formal drainage.

Any stormwater drainage that is required is expected to be in the form of shaping and grading the surface above the reservoir to shed runoff in a dispersed fashion and effectively match existing runoff quantities and overall flow paths. It is proposed therefore, that no specific drainage is required.

However, it is understood surface flows that occur in flood events from the catchment around the reservoir upstream of Rolleston Street currently do not flow to Rolleston Street but flow across the upper park. It may therefore be a change in secondary flow path for these flows to be directed by landform and landscaping to Rolleston Street as is required in the brief. Similarly, upper park surface runoff is required in the brief to be directed to Rolleston Street which may also be an alteration to existing flow paths.

Such a change is normally an issue that requires inclusion in a resource consent process. We anticipate that quantification of such changes, possibly by modelling, may be a requirement for resource consenting. It is noted that the resultant flow changes to Rolleston Street may, however, be similar to the flows that might be expected in a maximum reservoir overflow scenario which is also to be directed down Rolleston Street.

6.2 Upper Park Sports Field Drainage

The proposed modifications to the sports field surface levels and draining it to Rolleston Street (as required in the brief) will result in some relatively minor changes to existing flow paths and catchments. The eastern corners of the field currently drain off to the east as opposed to Rolleston Street to the west.

It is expected that the effects of this change will be relatively minor due to the small change in areas involved. The drainage assumes Options P1 or P2 (refer Park and Surplus Options Assessment Report) and so consists of a grass (or concrete lined) swale along north and south edges of the field. These swales will be drained by sumps into collector drains. A manhole at each end will be provided for maintenance access. If these cannot be located sufficiently clear of the playing areas then the manholes will be buried so as not to impact on the use of the field. In order to minimise the number of manholes in or around the sports field, uPVC pipes will be used allowing two of the sumps to be connected by junctions.

The sports field drainage has been sized to a 1 in 2 year return period in accordance with WCC standards. For the concept stage the depths and sizes of manholes have been estimated as 1350mm in diameter and 2.4m deep (unless noted otherwise) for pricing purposes. This will be refined during detailed design.

6.3 Upgrade of Rolleston Street Drainage

The existing drainage along Rolleston Street from the inlet of the catchment upstream of Rolleston Street to manhole M27 006 will be upgraded. Design flows for the catchment and the design scour discharge of 400 L/s will be considered for the design of this upgrade. At this concept stage a 600mm diameter RCRRJ pipe has been included; however this will be reviewed at the subsequent preliminary design stage. This drain has been assumed to be laid adjacent to the existing pipe and the existing pipe abandoned/blocked off where necessary. A standard precast headwall with a galvanised steel debris grill will be provided on the inlet from the catchment upstream of Rolleston Street. Rock rip rap to provide a stabilised inlet for protection from scour and erosion will be provided.

6.4 Capacity of Rolleston Street Drainage

We have assessed the capacity of the stormwater system in upper Rolleston Street (immediately downstream of manhole M27 006 being the current limit of works for the project) to be approximately 550L/s. The design is based on the assumption that this pipe performs to this capacity and note that any further works to increase capacity of this drain are not currently within the scope of the project.

Our assessment of the existing catchment hydrology suggests that 550L/s falls between a 1 in 5yr and 1 in 10yr ARI storm (being 483L/s and 567L/s respectively). The changes made as a result of the project will increase these peak flows by a relatively minor amount to 496L/s and 582L/s, these increases are a result of a small changes in catchment area as a consequence of modifying the playing field.

We note our assessment of the 100yr peak flow is 840L/s currently and increasing to 862L/s as a result of the works.

6.5 Secondary/Overflow Down Rolleston Street

The overflow/scour drain from the reservoir itself will connect into the upgraded Rolleston Street drain. The overflow from the reservoir is sized to allow 1200L/s which exceeds the capacity of the Rolleston Street storm water drain. It is proposed that the resulting overflow would spill to ground in a controlled manner via an overflow manhole as secondary flow down Rolleston Street.

As Rolleston Street is steep, relatively wide and valley like with clear topography, the catchment and flowpaths are relatively easily defined and understood. We have therefore approached assessing the effects of an overflow (whether by reservoir overflow or by storm overflow) by making a preliminary assessment of the flow capacity down the road. This included a site walk over to identify critical locations down the street.

At the upper end the street the road is up to 8.65m wide (kerb to kerb) with footpaths 1.5m to 1.75m wide on each side. The flow capacity down this corridor is approximately 1500L/s with at a depth of 50mm. This figure is substantially higher than the maximum expected overflow from the reservoir of 1200L/s or a 100yr storm. And even if the primary stormwater drain is blocked or constricted (i.e. 550L/s is not deducted from the overflow amounts) the there is sufficient capacity in the road corridor for these flows.

Further down the street it bends and the road crossfall would act to concentrate flow to the inside of the bend. However, the geometry of the road, footpath and a boundary wall means the flow can be much deeper resulting in no lessening of flow capacity and so is not expected to be a constriction.

After the bend in the road the cross section becomes more even and has a similar capacity to that noted above. Ultimately, the overflow will pond down on Wallace Street and from there the only available flow path away is through a Massey Campus building that sits across a gulley below

Wallace Street. We understand the options to upgrade the downstream drainage network is under the jurisdiction of another project.

6.6 Reservoir Failure Scenarios

The effects of the overflow or a seismic failure at the reservoir will also result in secondary flow down Rolleston Street from an overflow manhole. An assessment of the likely failure modes of the reservoir and piping after a significant seismic event has been carried out as part of the concept design.

The reservoir structure is to be designed to be robust. The most likely cause of failure of the structure would be as a result of fault rupture adjacent or under the reservoir or slope failure under the reservoir.

The risk of fault rupture affecting the reservoir is considered to be extremely low, with the nearby Lambton Fault being classified as Inactive and with no proven movement in the last 100,000 years. The risk of slope failure due to seismic shaking is also considered extremely low with the reservoir platform achieving a factor of safety greater than 1.0 for seismic events with a return period of 2500 years and close to 1.0 for seismic events with return periods of 5000 and 7500 years.

Pipe failure, particularly at a flanged joint position or flexible bellows within the pipe tunnel, is considered the most likely scenario to generate an uncontrolled discharge of any significance. Provisions to provide flexibility for pipe rotation or isolation of pipeline thrust will be included in the pipe design. These have finite capacity to absorb rotation and thrust however, and will likely rupture before failure of the steel pipe sections. The likelihood of a full pipe bore failure (900mm open cross section) is also considered very unlikely with partial bore failure with significant less cross sectional area being a more probable scenario.

Mitigating measures to prevent uncontrolled discharge from the piping in the tunnel include:

- Providing an actuated valve as the second fitting on the reservoir pipelines within the tunnel with seismic trigger to automatically close at a SLS1 level earthquake acceleration or greater.
- Additional manually operated valves upstream of the actuated valves to provide double block and bleed (tell tale indication) functionality of failure to close of the actuated valves.
- Locating pipe bellows or other flexible joints downstream of the pipeline valves.
- The new pipeline will be fabricated from cement lined steel with fully welded joints.
- The pipe tunnel floor drainage system will be designed to channel the equivalent overflow discharge of 1200 l/s via a separate pipeline to the overflow manhole located in the access way from Rolleston Street to the playing field. A grated cover will allow controlled overflow to the street. Personnel access should be able to the tunnel if a pipe rupture has occurred to check the closure of valves and manually operate the secondary shut off valves.

The probability of a seismic event to cause such a failure is low. The mitigating measures noted above would further reduce the risk of uncontrolled discharge if a pipe failure occurred.

7 RMA Compliance

7.1 Planning Considerations

A number of consent(s) will be required in order to undertake the physical works. The preparation of an Assessment of Environmental Effects (AEE) and the consent applications required for the project are to undertaken during Stage 2 after completion of the preliminary design report. Planning input to Stage 1 has been limited to consultation with WCC Parks and Gardens and this process will not commence until Stage 2 of the project.

Initial consultation between WCC Parks and Gardens and Beca has been carried out to outline the extent of disturbance for each reservoir option and obtain input on the key vegetated areas on site to assist with the attribute scoring of the various options.

This consultation has identified clear preferences from WCC Parks and Gardens for reservoir options where the final landform is as close as practicable to the existing contours of the site and that minimise the area of vegetation disturbance. In response to these preferences, reservoir option R3.1 was developed to provide a 'best fit' landform option for the site that resembles the existing ground profile much more closely than R3.0 (or R1 0 and R2.0).

To identify and define the key vegetated areas on the site a survey plan of the site has been marked up showing the current areas of vegetation refer to drawing CE-K30 in Volume 2 - Drawings. These areas have classified the vegetation as high-value, moderate value and low-value based on discussions on site between WCC Parks and Gardens and Beca. Option assessment has reflected the importance of protecting the areas of high-value vegetation.

Table 7-A outlines the likely consents required for the reservoir options included in the Park & Surplus Material Options Assessment Report. The additional options R1.2 and R3.1 included in this report are expected to be similar to the R1.1 and R3 options respectively included in this table.

A preliminary soil contamination assessment of the upper playing field at the Prince of Wales Park has been carried out by CH2M Beca Ltd. The investigation identified the presence of low levels of organochlorine pesticide, DDT, and heavy metals (cadmium, lead and nickel) in shallow surface samples collected from hand auger locations within the sports turf area. Low levels of PAH were also detected in these samples.

Results of the sampling were compared against 'commercial/industrial outdoor worker' and 'recreational' assessment criteria selected from the Resource Management (National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health) regulations 2011 (NES (Soil)) and PAH above background levels.

A resource consent will be required to undertake soil disturbance under either Regulation 9, 10 or 11 of the NES (Soil). An intrusive Detailed Site Investigation (DSI) is recommended as a next stage of assessment. Where a DSI report exists which states that the soil contamination does not exceed the standards in Regulation 7, then the activity can proceed as a Controlled Activity. If the soil contamination exceeds the standards in Regulation 7, then the activity proceeds as a Discretionary Activity.

The Regulations apply regardless of the level of contamination and control certain types of activities on contaminated land including soil disturbance.

Table 7-A Consent Requirements

Consents Required	Options			
	R1.0 Circular Reservoir, Central	R1-1 Circular Reservoir, Offset Towards West	R2 .0 Rectangular Reservoir	R3.0 Circular Reservoir, Central Truncated Roof.
Greater Wellington Regional Council				
Bulk earthwork – over 10,000m^2 on slopes over 28^o	x	✓	✓	x
Discharge permit to discharge sediment to water.[1]	×	×	×	×
Discharge permit to discharge sediment and chemical flocculant to water, and to land where it may enter water[1]	×	×	×	×
Wellington City Council				
Earthworks	✓	✓	✓	✓
Planting in the town belt	×	×	×	×
Carparking & access routes	NA	NA	NA	NA
Vegetation removal over 100m^2	✓	✓	✓	✓
Structures in Open Space C	✓	✓	✓	✓
Hazardous substances	Organochlorine pesticide, DDT heavy metasl and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metasl and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metasl and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metasl and low levels of PAH detected in preliminary sampling across sportsfield
Utility Structures in Open Space C land.	✓	✓	✓	✓

Note 1: Assumes that all stormwater is treated to remove sediment to meet total suspended solids limits before being discharged into the stormwater system. If not, consent may be required for this discharge.

8 Reservoir Structural Option For Multi Criteria Analyses

Capacity have defined the reservoir requirements in the following statement:

"The chosen design solution must be fit for purpose, proven, low risk, seismic resilient for a significant structure to provide best engineering solution to the Hospital and residents particularly after a post-seismic event."

The reservoir ***structural*** options identified in this concept study have been assessed against this definition as a filter to screen options to be taken forward to a Multi Criteria Analyse (MCA) process.

Reservoir Options R1.0, R1.1 and R1.2 have a structural form that is both proven and considered viable from both an engineering and asset owner's position. These options have been taken forward to the MCA process.

Both reservoir Options R3.0 and R3.1 have permanent hydrostatic water load on the underside of the roof which does not meet the viability criteria. The absence of proven track record means the asset owner may be exposed to design and performance risks for such a reservoir roof form. For these reasons and in agreement with Capacity the R3.0 and R3.1 options are not considered to adequately meet the defined reservoir requirements to warrant taking them forward to the MCA process.

Option R2.0 has some identified risks for watertightness and seismic resilience but does have a proven record of similar reservoir structural form. This option is therefore considered acceptable to take forward to the MCA process.

9 Reservoir Option Multi-Criteria Analyses

Multi-Criteria Analyses (MCA) have been carried out based on a high level engineering assessment of key attributes, excluding cost, in order to rank the viable reservoir options identified. The analyses have been carried out by Beca and not through a workshop or similar stakeholder meetings. Input received from participants at the project risk workshop session including Greater Wellington Regional Council, WCC Parks & Gardens and Capacity has been taken into account in the scoring of the attributes. Comments received from Opus International Consultants Ltd, WCC Parks & Gardens and Capacity on earlier Beca reports (Parks & Surplus Materials and Initial Geotechnical reports) have also been considered in the scoring.

9.1 Attributes

Attributes considered and their definition were:

- Earthworks/Geotechnical
  Extent of earthwork activities and geotechnical design implications.
- Seismic Resilience
  Dependability of the seismic design and proven performance of the structural system.
- Water Tightness
  Proven performance of the structural system adopted which enables the number of water retaining joints to be minimised and/or the likelihood of cracking that allows leakage to be minimised.
- Consenting/Landform
  Consenting difficulty and extent of disturbance to existing vegetation and modification of landform.

Each attribute was assigned an equal weighting. The attribute scoring system ranged from 1 (unfavourable) to 3 (neutral) to 5 (favourable). Note that cost was excluded as an attribute. Cost estimates are presented with attribute scores in Section 10. The attribute scoring and commentary is included in Appendix A.

Table 9-A - Multi-Criteria Analyse, Reservoir Option Attribute Scoring

	Attributes & Scoring				
Reservoir Options	**Earthworks/ Geotechnical**	**Seismic Resilience**	**Water Tightness**	**Consenting/ Landform**	**Total**
R1.0 – Circular, Centrally Located	4	5	5	3	17
R1.1 – Circular, Offset, Gully Fill	1	4	4	1	10
R1.2 – Circular, Centrally Located, Gully Fill	3	5	5	1	14
R2.0 – Rectangular Circular Ends	3	3	3	2	11

Referring to Table 9-A, Option R1.0 (circular reservoir which is centrally located) is the highest scoring option.

Option R1.2 (circular reservoir which is centrally located with gully fill) is the second highest scoring option. It has been penalised with the use of the gully for filling resulting in significant disturbance to the existing vegetation and modification to the landform.

Option R1.1 (circular reservoir which is offset with gully fill) is the lowest scoring option. It has been penalised for use of the gully as for Option R1.2 as well as for geotechnical implications of part of the reservoir not being founded on a uniform subgrade stiffness.

The outcome of the MCA, without consideration of cost, is to select Reservoir Option R1.0 as the preferred option.

10 Cost Estimates

10.1 Base Estimates

Cost estimates with breakdown have been prepared for Option R1.0. Summary cost estimates have been prepared for all other options and are included in Table 11-A. These estimates include design, consenting and construction costs. The estimates are inclusive of estimating contingency, P and G, offsite overheads, profit, engineering and contract contingency.

Cost estimate details for Option R1.0 are included in Appendix B.

Table 10-A Cost Estimates

Option	Description	Estimated Cost (excluding GST
R1.0	Circular, central to site, conventional roof	$17.9M (-5% $17.0M; +10% $19.7M)

The cost estimates assume a 1,000 year return period for earthquake serviceability limit state SLS2 (refer Table 4-D) with a design working life of 100 years.

The additional cost to adopt a 1,500 year return period for earthquake serviceability limit state SLS2 is estimated to be $500,000 (-5%; +10%).

The reduction in cost to adopt a 500 year return period for serviceability limit state SLS2 is estimated to be $500,000 (-5%; +10%).

Although not estimated in detail, comments on other options included in this report are:

- Option R1.1, Circular and Offset Located
  - This option was identified as the most cost effective earthworks option in the Park and Surplus Material Options Assessment Report. Based on the earthworks cost saving, the estimated cost for Option R1.1 is $17.7M (-5%; +10%).

- Option R1.2, as Option R1.0 but with gully disposal.
  - This option was suggested by Opus in their review process. It involves using the gully to the west of the site for disposing of cut material. It is estimated to have a similar cost as Option R1.1 of $17.7M (-5%; +10%).

- Option R2.0, Rectangular with Circular Ends
  - The earthworks costs were estimated in the Parks and Surplus Material Options Assessment Report as $1.1M more expensive than for Option R1.0, and overall construction costs are expected to be greater. No detail cost estimate has been prepared for Option R2.0, but is expected to be in the order of $20.1M (-5%; +10%).

11 Summary

Table 11-A Attribute Scoring and Estimated CostA below identifies attribute scoring and estimated costs for each reservoir option.

Table 11-A Attribute Scoring and Estimated Cost

Option	Attribute Total	Estimated Cost (-5%; +10%) (Excluding GST)
R1.0	17	$17.9M
R1.1	10	$17.7M
R1.2	14	$17.7M
R2.0	11	$20.1M

The cost estimates are based on a design for a SLS2 1000 year return period earthquake. Reducing this to a SLS2 500 year return period earthquake would reduce the estimated cost by $0.5M(-5%; +10%).

The site is assessed to be suitable geotechnically for a 35,000 cubic metre capacity reservoir. Reservoir failure through disruption of the structure as a result of a major earthquake event is minimised by WCC’s decision that it is a facility required to be available for operation immediately following a major earthquake event. Pipework detailing will be designed to accommodate some ground displacement, and isolation valves actuated by a seismic sensing trigger device which will close to prevent escape of water.

Based on the attribute scoring and estimated costs, the preferred Option is R1.0.

While R1.1 and R1.2 are slightly less expensive than R1.0, they are both subject to likely Resource Consent stage objections due to use of the gully for disposal of excess material.

We note that the estimated costs exceed previous client estimates undertaken for this reservoir facility. In our opinion significant cost reduction (if required) could be achieved primarily through storage capacity reduction. We would be pleased to develop any reduced storage option as an extension to the brief.