

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



© CH2M Beca 2013 (unless CH2M Beca has expressly agreed otherwise with the Client in writing). This report has been prepared by CH2M Beca on the specific instructions of our Client. It is solely for our Client's use for the purpose for which it is intended in accordance with the agreed scope of work. Any use or reliance by any person contrary to the above, to which CH2M Beca has not given its prior written consent, is at that person's own risk.

Revision History

Revision N°	Prepared By	Description	Date
A	Simon Edmonds/Mike Yarrall/Darren Hewett/Iain Smith/Wade Robertson/Anna Lewis	Draft For Client Review	09/2012
B	Simon Edmonds/Mike Yarrall/Darren Hewett/Iain Smith/Wade Robertson/Anna Lewis	Final	01/02/2013

Document Acceptance

Action	Name	Signed	Date
Prepared by	Simon Edmonds/Mike Yarrall/Darren Hewett/Iain Smith/Wade Robertson/Anna Lewis		19 February 2013
Reviewed by	Dennis Hunt/Brian Smith/Graham Ramsay		19 February 2013
Approved by	Mark Sneddon		19 February 2013
on behalf of	CH2M Beca Limited		

Volume 1 of 2 – Report Text and Appendices

Table of Contents

Executive Summary.....	1
1 Introduction.....	5
1.1 Background	5
1.2 Purpose and Scope of Report	7
1.3 Project Components	8
1.4 Associated and Referenced Reports.....	8
1.5 Appendices	8
1.6 Abbreviations	9
1.7 Acknowledgements	9
2 Reservoir Configuration Options.....	9
2.1 Background	9
2.2 Site Constraints	9
2.3 Reservoir Arrangements Options	9
2.4 Cost Optimisation	11
2.5 Configuration Options Summary	11
3 Pipework Tunnel Options	14
3.1 Piping Configuration	14
3.2 Piping Tunnel Arrangement Options Considered	14
3.3 WCC Feedback on Preliminary Tunnel Arrangement Options.....	15
3.4 Options Selection	15
3.5 Functional Requirements.....	16
3.6 Pipe Tunnel Recommendations	16
4 Seismic Design	16
4.1 Seismic Hazard Scenarios	17
4.2 Derivation of Total Seismic Load.....	18
4.3 Geotechnical Considerations.....	19
5 Geotechnical Considerations – Non Seismic	19
6 Stormwater & Overflow Drainage & Potential Overflow/Failure Secondary Flow Routes	19
6.1 Drainage of Reservoir Site around the Existing Spur Catchment	20
6.2 Upper Park Sports Field Drainage	20
6.3 Upgrade of Rolleston Street Drainage	21
6.4 Capacity of Rolleston Street Drainage	21
6.5 Secondary/Overflow Down Rolleston Street	21
6.6 Reservoir Failure Scenarios	22
7 RMA Compliance	22
7.1 Planning Considerations.....	22
8 Reservoir Structural Option For Multi Criteria Analyses	25

9	Reservoir Option Multi-Criteria Analyses	25
9.1	Attributes.....	25
10	Cost Estimates.....	26
10.1	Base Estimates.....	26
11	Summary	27

List of Tables

Table 4-A	50-year working life structure probability of exceedance (NZS1170.0).....	17
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.....	17
Table 4-C	Return Period Factor, R.....	18
Table 4-D	Seismic Horizontal Accelerations $C_d(T_i)$ for Different Limit States & Return Periods.....	19
Table 7-A	Consent Requirements	24
Table 9-A	- Multi-Criteria Analyse, Reservoir Option Attribute Scoring	26
Table 10-A	Cost Estimates.....	26
Table 11-A	Attribute Scoring and Estimated Cost.....	27

List of Figures

Figure 1-A:	Proposed Reservoir Site Location	5
Figure 1-B	Aerial View of Prince of Wales Park.....	6
Figure 1-C:	Existing Site Profile of Upper Prince of Wales Park Looking South Across the Proposed Reservoir and Pipe Tunnel Site.....	7
Figure 2-A	Reservoir Cost Index Optimisation - Base Dimension	12
Figure 2-B	Reservoir Cost Index Optimisation - Water Depth	13

Appendices

Appendix A - Option Attribute Criteria Scoring Table

Appendix B - Cost Estimate

Appendix C - Structural Basis of Design

Appendix D - Mechanical Basis of Design

Appendix E - Geotechnical Basis of Design

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).

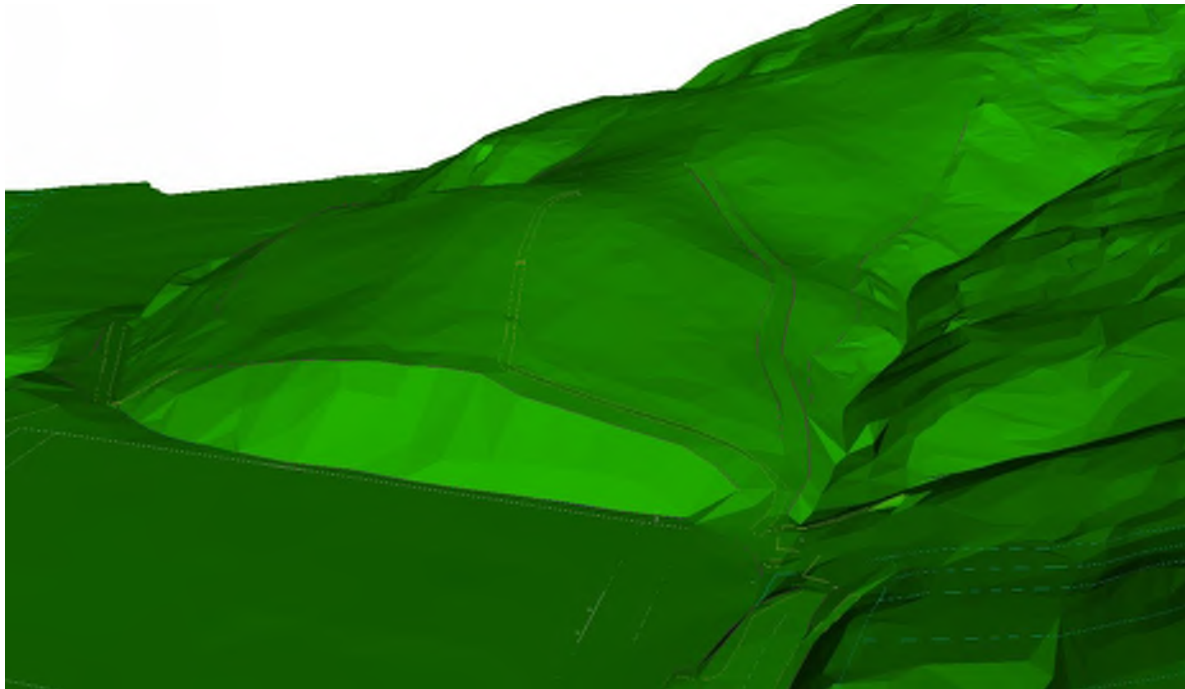
Figure 1-A: Proposed Reservoir Site Location



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 35,000m³ 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 shown 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 m² (base cost)
- Reservoir walls: quadratic interpolation between indices values: (up to 10m high = 1.5 per m²), (10m – 15m high = 2.5 per m², and 15.20m high = 4.0 per m²)
- Reservoir roof index: 2.0 per m²

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.

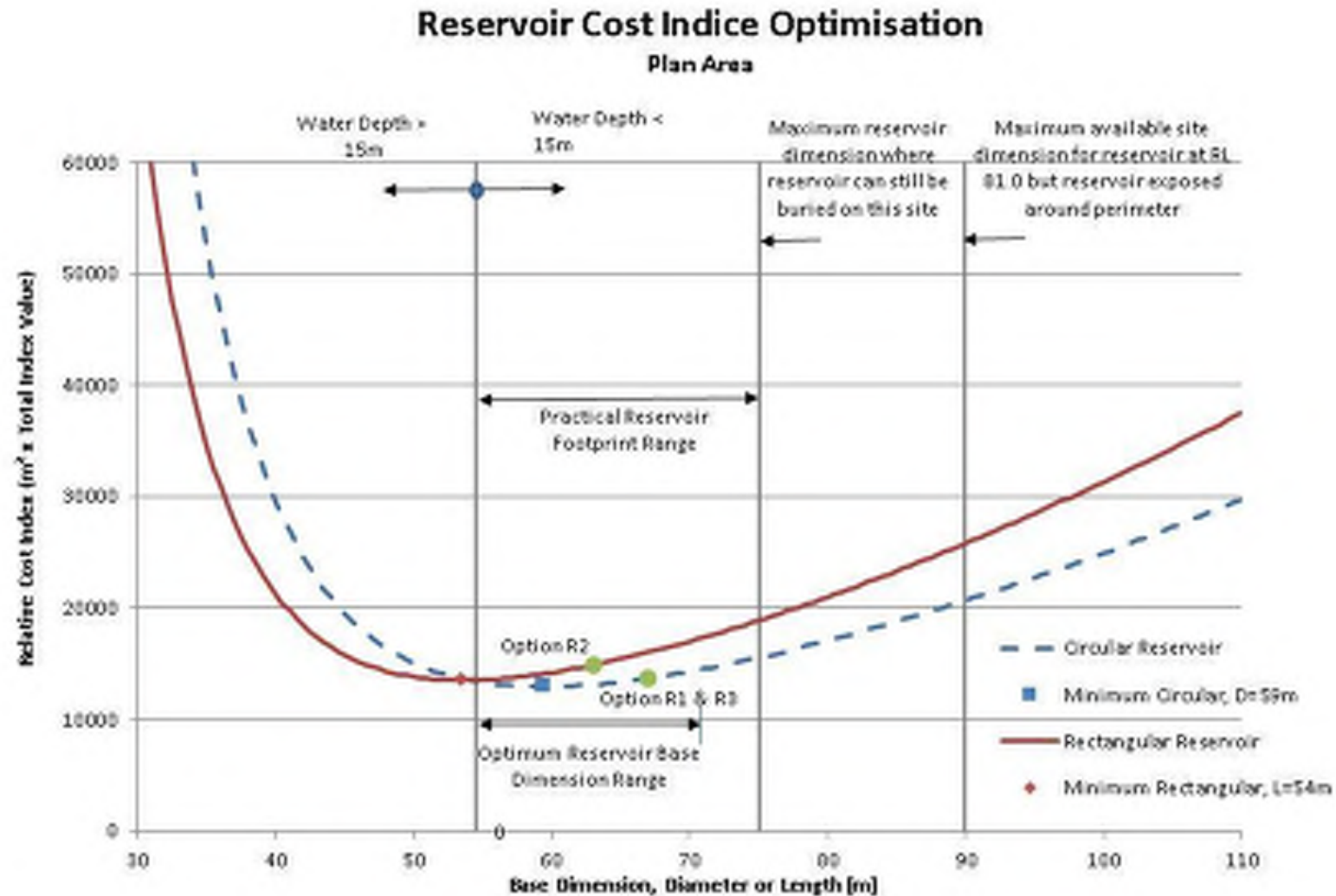


Figure 2-A Reservoir Cost Index Optimisation - Base Dimension

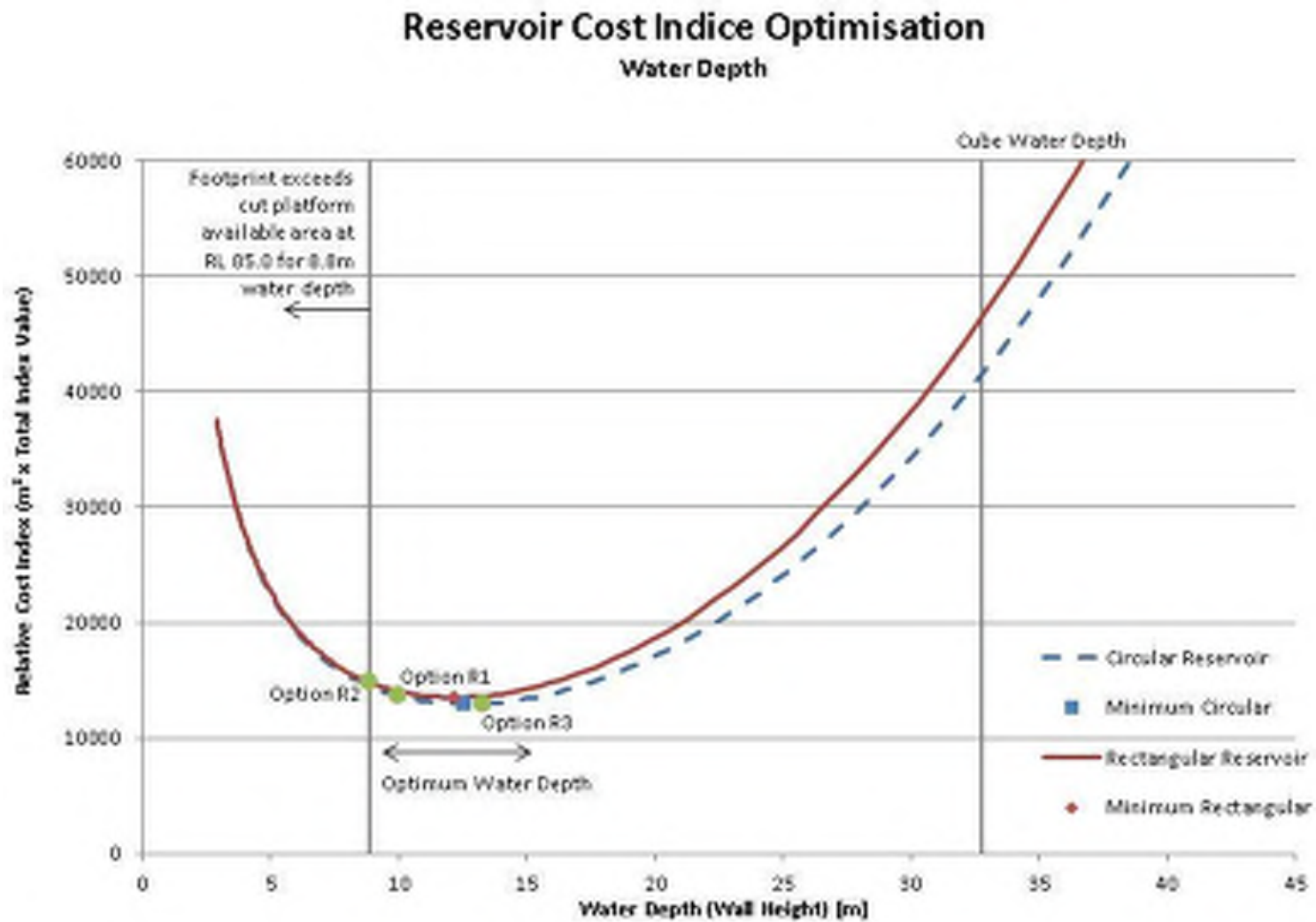


Figure 2-B Reservoir Cost Index Optimisation - Water Depth

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 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 the 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) 25 years
- Return Period Seismic Event

(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 $C_d(T_i)$ 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) 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 gully 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°	x	✓	✓	x
Discharge permit to discharge sediment to water. ¹	x	x	x	x
Discharge permit to discharge sediment and chemical flocculant to water, and to land where it may enter water ¹	x	x	x	x
Wellington City Council				
Earthworks	✓	✓	✓	✓
Planting in the town belt	x	x	x	x
Carparking & access routes	NA	NA	NA	NA
Vegetation removal over 100m ²	✓	✓	✓	✓
Structures in Open Space C	✓	✓	✓	✓
Hazardous substances	Organochlorine pesticide, DDT heavy metals and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metals and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metals and low levels of PAH detected in preliminary sampling across sportsfield	Organochlorine pesticide, DDT heavy metals 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

Reservoir Options	Attributes & Scoring				
	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.

Appendix A

Option Attribute Scoring Table A1

Table A1 – Option Attributes Scoring

Option	Commentary	Attributes and Scoring				
		Earthworks/ Geotechnical	Seismic Resilience	Water Tightness	Consenting/ Landform	Total
R1.0	■ Circular reservoir; 67m ID and 10m water depth ■ Centrally positioned within the existing ridge spur ■ 'Flat' roof (0.6m fall approximately) ■ Backfill slopes 1V to 2H generally ■ Moderate extent of disturbed ground in spur ■ Shape of buried reservoir will be evident at the site ■ Shape of reservoir does not relate to spur ■ Moderate amount of vegetation removed ■ Favourable structural (typical design concept) ■ Seismic Resilience of reservoir form is favourable ■ Neutral for pipe tunnel ■ Neutral geotechnical outcome ■ Average quantity of surplus excavated material to dispose of ■ Modest amount of excavated material storage off site required ■ Low to moderate degree of 'end use' landscape and visual effect ■ High degree of landscape and visual effects during construction	4	5	5	3	17
R1.1	Same as for R1.0 with the following changes: ■ Positioned at the western edge of the ridge spur ■ Some backfill slopes flatter than 1V to 2H ■ Substantial (Very high) extent of disturbed ground in spur and gully ■ Shape of buried reservoir will be less evident than R1.0	1	4	4	1	10

Table A1 – Option Attributes Scoring

Option	Commentary	Attributes and Scoring				
		Earthworks/ Geotechnical	Seismic Resilience	Water Tightness	Consenting/ Landform	Total
	■ Gully habitat destruction is balanced by avoiding cost for disposal of surplus material and reduced road construction traffic ■ Substantial (Very high) extent of vegetation removed ■ Moderate to high degree of 'end use' landscape and visual effect ■ Very high degree of landscape and visual effects during construction ■ Unfavourable geotechnical conditions for foundations ■ Gully used for disposal of surplus excavated material ■ Best cut/fill balance for earthworks ■ No excavated material storage off-site required					
R1.2	Same as for R1.0 with the following changes: ■ Gully used for excess fill disposal and some storage ■ Substantial (Very high) extent of disturbed ground in spur and gully ■ Gully habitat destruction is balanced by avoiding cost for disposal of surplus material and reduced road construction traffic ■ Shape of buried reservoir will be less evident than R1.0 ■ Substantial (Very high) extent of vegetation removed ■ Very high degree of landscape and visual effects during construction ■ Moderate to high degree of 'end use' landscape and visual effect ■ Neutral geotechnical outcome ■ Best cut/fill balance for earthworks ■ No excavated material storage off site required	3	5	5	1	14

Table A1 – Option Attributes Scoring

Option	Commentary	Attributes and Scoring				
		Earthworks/ Geotechnical	Seismic Resilience	Water Tightness	Consenting/ Landform	Total
R2.0	■ Rectangular reservoir (rounded ends) 100m x 40m plan dimensions with water depth of 9.0m (equivalent 63m by 63m tank) ■ Positioned along a SW-NE alignment within the existing ridge spur ■ Backfill slopes 1V:2H generally ■ Flat roof (0.6m fall approximately) ■ Substantial (Very high) extent of disturbed ground in ridge line ■ Shape of buried reservoir will be evident at the site ■ Shape of reservoir reflects alignment of spur but results in significant cut face at southern end ■ High amount of vegetation removed ■ Very high degree of landscape and visual effects during construction ■ Moderate degree of 'end use' landscape and visual effect ■ Seismic resilience of reservoir form less favourable than R1.0 ■ Neutral for pipe tunnel ■ Neutral geotechnical outcome ■ Large quantity of surplus excavated material to dispose of ■ High amount of excavated material storage off-site required	3	3	3	2	11

Scoring System:

Unfavourable

Neutral

Favourable

1**2****3****4****5**

Notes:

1. RMA compliance has been addressed for options R1.0; R1.1; and R2 in the parks and surplus materials report. Option R1.2 is a subset option of R1.0 and has similar consenting attributes.
2. The overall degree of landscape and visual effect (i.e. construction and end use scenarios) will be subject to public consultation and additional site evaluation.
3. The degree of landscape effect has been considered using a 5 point scale, including: Very Low, Low, Moderate, High and Very High. Generally, very low and low denote a 'minor' (or less than) effect whereas high and very high denote 'significant'.
4. Landscape and visual criteria has been assessed on
  - (a) Size and shape of reservoir
  - (b) Location of reservoir with spur
  - (c) Extent of disturbed ground (overall)
  - (d) Final form of reservoir and relationship to spur
  - (e) Reservoir backfill batter slope profile
  - (f) Extent of vegetation removal
  - (g) Overall physical effects
  - (h) Overall visual effects

Appendix B

Cost Estimates

Hospital Prince of Wales Reservoir Cost Estimate Summary Table Option R1.0

Item	Description	Lower Bound	Estimated Cost	Upper Bound
1.0	Services diversions and demolition	\$ 190,000	\$ 190,000	\$ 190,000
2.0	Inlet and outlet pipework (in trench)	\$ 380,000	\$ 380,000	\$ 380,000
3.0	Topsoil removal (upper park) and disposal	\$ 12,000	\$ 12,000	\$ 12,000
4.0	Bulk excavation to stockpile and off site disposal	\$ 970,000	\$ 970,000	\$ 970,000
5.0	Subsoil drainage	\$ 225,000	\$ 225,000	\$ 225,000
6.0	Pipe tunnel structure	\$ 615,000	\$ 615,000	\$ 615,000
7.0	Reservoir structure	\$ 7,000,000	\$ 7,000,000	\$ 7,000,000
8.0	Pipework and valves	\$ 1,050,000	\$ 1,050,000	\$ 1,050,000
9.0	Electrical and Control System (Capacity Estimate)	\$ 340,000	\$ 340,000	\$ 340,000
10.0	Backfill and Access Driveway	\$ 300,000	\$ 300,000	\$ 300,000
11.0	Topsoiling and Landscaping	\$ 285,000	\$ 285,000	\$ 285,000
12.0	Overflow/Stormwater Drainage	\$ 230,000	\$ 230,000	\$ 230,000
13.0	Upper Sports Ground Resurfacing	\$ 130,000	\$ 130,000	\$ 130,000
14.0	Rolleston Street Pavement Repairs	\$ 150,000	\$ 150,000	\$ 150,000
	Estimating Contingency (approx.9.5, 12 & 17%)	\$ 1,130,000	\$ 1,450,000	\$ 2,030,000
	Subtotal	\$ 13,007,000	\$ 13,327,000	\$ 13,907,000
	Add Preliminary and General (12%)	\$ 1,560,840	\$ 1,599,240	\$ 1,668,840
	Subtotal	\$ 14,567,840	\$ 14,926,240	\$ 15,575,840
	Add Off-site Overheads and Profit (8%)	\$ 1,165,427	\$ 1,194,099	\$ 1,246,067
	Total Base Estimate	\$ 15,733,268	\$ 16,120,339	\$ 16,821,907
15.0	Consenting (preparation of AEE; Consultation and preparation of consent applications)	\$ 172,000	\$ 172,000	\$ 172,000
16.0	Engineering (Concept, Preliminary and Detail Design and Construction Management & Observation)	\$ 700,000	\$ 700,000	\$ 700,000
	Total Expected Estimate	\$ 16,605,268	\$ 16,992,339	\$ 17,693,907
	Contract Contingency (approx. 2.5%, 5.5% & 11.5%)	\$ 394,733	\$ 907,661	\$ 2,006,093
	Total	\$ 17,000,000	\$ 17,900,000	\$ 19,700,000
	Comparison	-5%	0%	+10%

Note 1: Main Contractor Preliminary & General (P&G) costs covers the cost of on-site overheads such as site supervision / management, site offices, stores, hoardings, amenities, plant cranes , temporary works, etc.

Note 2: Main Contractor Off-site overheads and Profit (OH&P) Margins covers the cost of contributions to the Main Contractor's Profit and off-site overhead costs.

Note 3: The Estimating Contingency sum is integral to the estimate total and is a general allowance for residual cost risk including design development, omissions, sundry measured items and assumptions made for construction details not shown. This is not a project contingency which is expected to be held in addition to this estimating contingency. Typically the estimating Contingency decreases throughout the design development process.

Note 4: The Recommended Contract Contingency is a sum proposed to be held by the Client available to cover post contract variation works to the contract, such as unforeseen ground conditions and construction risk.

Note 5: The lower and upper bound estimates represent probable variances in assumed adjustments to the estimated value of the current design, they do NOT represent a guaranteed or maximum / minimum price.

Note 6: The estimate is based upon rates and prices current as at October 2012 and no allowance has been included for increases (escalation) in labour, materials or plant beyond this date.

Note 7: All values within this report and included in the attached Estimate Details are GST exclusive.

Note 8: Current market assumed rates and sums based on a traditional procurement route, ie. fully designed and competitively tendering for lump sum tenders from at least three suitable selected tenderers.

Note 9: Items 15.0 and 16.0 records the values included in the accepted consultant's commission.

Note 10: The percentage adjustments noted as "approx." have been adjusted to give rounded number totals to reflect the high level nature of the estimating at this time

Item 1 - Services Diversions and Demolition

ITEM	DESCRIPTION	QTY (m)	Rate (\$/unit)	Cost
1.1	Site establishment of new permanent vehicle access to upper park site across road reserve			\$ 10,000
1.2	New diversion of City to Sea walkway. Gavel path 900mm wide, 120m long from the end of Rolleston St up the existing gulley to a point above the reservoir.			\$ 10,000
1.3	Site temporary fencing around construction site (allows for use of existing upper park fence as construction site perimeter fence)	300	\$ 120.00	\$ 36,000
1.4	Removal of car parking spaces and no parking line marking on Rolleston Street for construction vehicle access			\$ 10,000
1.5	Site water/power connections for construction phase that are converted to final connections as part of contract works			\$ 20,000
1.6	Relocation of 375 CI watermain across upper park (1934 CI replacement) with 400NB CLS plus tie ins adjacent existing 450 CLS and 375 CI watermain	100	\$ 810.00	\$ 81,000
1.7	Demolition of flowmeter structures and removal of redundant piping			\$ 20,000
Total				<u>\$ 187,000</u>

Item 2 - Inlet and Outlet Pipework (in trench)

ITEM	DESCRIPTION	QTY (m)	Rate (\$/unit)	Cost
2.1	800NB Inlet Pipeline	130	\$ 1,332.18	\$ 173,183
2.2	900NB Outlet Pipeline	130	\$ 1,500.06	\$ 195,007
2.3	Connection allowance at Hargreaves Street	1	\$ 10,000.00	\$ 10,000
			Total	<u>\$ 378,190</u>

Notes:

- 1 Scope is in-trench pipework from exit of pipe tunnel to connection points in Hargreaves Street.
- 2 Assume rate includes excavation, disposal, concrete lined mild steel pipe with external wrapping, bedding and haunching material, backfill with imported material to finish ground level and testing and commissioning.

Item 3 - Topsoil Removal (Upper Park) and Disposal

ITEM	DESCRIPTION	QTY (m ³)	Rate (\$/unit)	Cost
3.1	Upper park topsoil removal and disposal	1,245	\$ 9.35	\$ 11,641
Total				<u>\$ 11,641</u>

Notes:

- 1 Southern landfill, no charge for topsoil disposal based on discussions with landfill

Item 4 - Bulk Excavation to Stockpile And Off Site Disposal

ITEM	DESCRIPTION	QTY (m ² , m ³)	Rate (\$/unit)		Cost
4.1	Formation of access road to reservoir site (included in bulk excavation quantity, item 4.5)			\$	-
4.2	Excavation of pipe tunnel to reservoir platform (included in bulk excavation quantity, item 4.5, 1,150m ³ allowance)			\$	-
4.3	Removal and disposal off site of vegetation across reservoir disturbed ground area	5,000	\$	10.00	\$ 50,000
4.4	Stockpiling top soil across reservoir disturbed ground area	500	\$	3.26	\$ 1,630
4.5	Bulk excavation to upper park stockpile	25,000	\$	6.70	\$ 167,500
4.6	Bulk excavation to offsite stockpile including storage. This rate includes an allowance of \$14.13/m ³ for the landfill fee per tonne of \$5.65 and 2.5 tonnes/m ³	5,500	\$	24.54	\$ 134,970
4.7	Bulk excavation to waste	25,000	\$	24.54	\$ 613,500
Total				\$	<u>967,600</u>

Item 5 - Subsoil Drainage

ITEM	DESCRIPTION	QTY (m)	Rate (\$/unit)		Cost
5.1	Excavation of 1.0m deep by 0.5m wide trenches across reservoir footprint for subsoil drainage along radial and circumferential floor construction joints, providing perforated 110mm diameter subsoil drains with filter sock in filter fabric wrapped pea metal drain. Drains under reservoir discharge to telltales in pipe tunnel.	650	\$	300.00	\$ 195,000
5.2	Drains around perimeter of reservoir and pipe tunnel, discharge to overflow pipe to Rolleston Street via new manhole.	100	\$	300.00	\$ 30,000
Total				\$	<u>225,000</u>

Item 6 - Pipe Tunnel Structure

ITEM	DESCRIPTION	QTY (m ³)	Rate (\$/unit)	Cost
6.1	Concrete material and construction cost	220	\$ 1,500.00	\$ 330,000
6.2	External Doors			\$ 20,000
6.3	Reservoir Floor Access Hatch (Watertight)			\$ 60,000
6.4	Access Stairs and Platforms			\$ 80,000
6.5	Floor Drain, Sump and Grate Covers			\$ 35,000
6.6	Waterproofing			\$ 50,000
6.7	Ventilation System			\$ 40,000
Total				<u>\$ 615,000</u>

Item 7.0 Reservoir Structure

ITEM	DESCRIPTION	QTY (m ² , m ³)	Rate (\$/unit)	Cost
7.1	Walls (11.8m high, 425mm thick)	1,056	\$ 3,000.00	\$ 3,166,773
7.2	Floor Slab (54m diameter, 250mm thick)	573	\$ 1,000.00	\$ 573,000
7.3	Wall Footing (0.5m deep x 2m wide)	212	\$ 1,000.00	\$ 212,000
7.4	Annular floor slab ring (54m to 67m diameter, 250mm thick)	309	\$ 1,200.00	\$ 370,800
7.5	Roof Slab (250mm thick)	881	\$ 950.00	\$ 837,342
7.6	Roof Support Beam (9 straight 700mm wide by 700mm deep, average 50m long) or as annular roof support beams (750mm wide x 950mm deep) at 10m, 34m and 57m diameters)	233	\$ 3,000.00	\$ 699,000
7.7	Roof to wall connection beam (0.5m by 0.5m)	53	\$ 3,000.00	\$ 157,865
7.8	Columns, 55 off 650x650mm cross section, 11m high	256	\$ 3,000.00	\$ 768,000
7.9	Column footing (55 no. 3m square 250mm thick plus slab thickness)	124	\$ 1,000.00	\$ 124,000
7.10	Wall Membrane	2,515	\$ 7.50	\$ 18,863
7.11	Roof Membrane	3,670	\$ 22.50	\$ 82,575
Total				<u>\$ 7,010,217</u>

Structural Description

Includes: Reservoir floor detail excavation for column and wall foundations and site concrete to base of excavations
150mm basecourse placement under floor slab
Roof support column foundation pads and floor slab pour for central 10m diameter section of reservoir and 34m diameter column pads
Column construction 26 No. 650mm square to 13m high poured insitu
Erection of falsework around columns to 12m high
Placement of annular precast reinforced ring beams 950mm deep x 750mm wide on falsework each side of columns
Site insitu joints between annular beam segments and post tensioning of complete annular ring beam at 34m diameter
Placement of radial precast roof infill beams (Single Tees) between central column ring and annular ring beam at 34m diameter
Topping slab 150mm thick over central 34m diameter section of reservoir
Construction of column foundations and wall propping deadmans in circumferential lines from at 57m diameter 29 No.
Column construction 29 No. 650 square from 9m to 11m high poured insitu
Wall perimeter strip foundation construction 2000mm wide x 500mm deep
Placement and propping of external wall precast, prestressed wall panels 425mm thick to foundation deadmans (note pilasters are 750mm thick)
Pouring vertical infill joints between wall panels
Incrementally stressing wall panel post tensioning in horizontal wall ducts between pilaster positions to 10% of capacity of circumferential post-tensioning
Erection of falsework around 29 No. columns to 10m high average
Placement of annular precast/post tensioned ring beam segments 950mm deep x 700mm wide on falsework each side of columns
Site insitu joints between annular beam segments and post tensioning of complete annular ring beam
Placement of 400 thick radial precast roof infill beams (1200mm wide) between annular ring beams at 57m diameter and top of wall panels (including PTFE bearings on top of wall panels)
Topping slab 150mm thick from 34m to 57m diameter
Completion of wall panel infills around reservoir perimeter (close up construction access through missing wall panels)
Pouring of floor slabs from central 10m diameter section of reservoir to 54m diameter
Circumferential post tensioning of reservoir walls between eight pilaster positions.
Pouring of final 6.5m wide annular floor slab section in 10m lengths around internal base of reservoir walls
Pouring of wall/roof connection beam around perimeter of reservoir incorporating roof topping slab from 57m to 67m diameter
Post tensioning wall/roof connection beam onto wall and circumferentially post tensioning segments together.
Construction of two roof hatches – 1 x personnel access only with ladder below and 1 x major opening (2.5m x 4m) incorporating personnel access.
Reservoir water test
External wall mulseal/bitumen emulsion coating
Application of roof membrane and protection slab
Backfill of reservoir walls incorporating drainage layer against walls full height

Item 8.0 Pipework and Valves

ITEM	DESCRIPTION	QTY (m ² , m ³)	Rate (\$/unit)	Cost
8.1	Pipe and valve supply and installation in pipe tunnel and reservoir.	1	\$ 1,050,000.00	\$ 1,050,000
Total				<u>\$ 1,050,000</u>

Notes:

- 1 Refer attached schedules for breakdown.

Hospital Prince of Wales Reservoir Pipe Tunnel Mechanical Costing Schedule

ITEM	QTY	NOM. O.D. (mm)	PIPE O.D. (mm)	SPOOL LENGTH (m)	MATERIAL	DESCRIPTION	MODEL	Rate (\$/item/length)	Cost
003	1	300			AIR VENT MESH			200 \$	700
007	1	300	324	21.00	SCH10 S/S 316	PIPE SPOOL, ONE END FLANGED		600 \$	12,830
003	1	300	324	-	SCH10 S/S 316	90° ELBOW, BOTH ENDS FLANGED		1,700 \$	1,660
004	1	300	324	10.00	SCH10 S/S 316	PIPE SPOOL FLANGED ONE END		600 \$	6,730
005	1	300	324	-	SCH10 S/S 316	45° ELBOW, BOTH ENDS PLAIN		1,200 \$	1,200
006	1	300	324	25.00	SCH10 S/S 316	PIPE SPOOL, BOTH ENDS PLAIN		600 \$	15,000
061	10				GALV. STEEL	HANGING PIPE SUPPORT		750 \$	7,500
101	1	800	813	0.80	CONC. LINED STEEL	PIPE SPOOL, PLAIN ENDS	(COVERED ELSEWHERE IN RUDDLE I)		
102	1	600 / 900	610 / 813	-	CONC. LINED STEEL	ECCENTRIC REDUCER, PLAIN ENDS		9,435 \$	9,435
103	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		3,968 \$	3,968
104	1	600	610	2.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		1,620 \$	3,240
105	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		3,968 \$	3,968
106	1	600	610	0.25	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		1,620 \$	405
107	1	600	-	-	COATED STEEL	COMPRESSION COUPLING	VIKING JOHNSON COUPLING	7,194 \$	7,194
108	1	600	610	0.25	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		1,620 \$	405
109	1	600	610	0.85	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		8,100 \$	7,705
110	1	600	610	3.40	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	5,918
111	1	600	-	-	ELASTOMER LINED STEEL	MAGNETIC FLOWMETER	ABB WATERMASTER 24V DC	20,000 \$	20,000
112	1	600	610	3.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		1,620 \$	5,680
113	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	6,000 \$	6,000
114	1	600	610	1.75	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	3,245
115	1	600	610	0.85	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		8,100 \$	7,705
116	1	600	610	0.60	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	1,382
117	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	6,000 \$	6,000
118	1	600	610	0.50	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		1,620 \$	1,630
119	1	600	-	-	RUBBER	SINGLE SPHERE BELLOWS	JAMES WALKER TOWNSON	6,000 \$	6,000
120	1	600	610	0.50	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		1,620 \$	1,630
121	1	600	-	-	CAST IRON	ACTUATED RESILIENT SEATED PLUG VALVE	VAL-MATIC 5800R W/ IP68 ROTORK IQ	13,000 \$	13,000
122	1	600	610	1.10	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		1,620 \$	2,602
123	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	6,000 \$	6,000
124	1	600	610	0.70	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		1,620 \$	1,954
125	1	600	610	0.70	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	1,541
126	1	600	610	-	CONC. LINED STEEL	90° ELBOW, BOTH ENDS PLAIN		7,935 \$	7,935
127	1	600	610	2.80	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED WITH PUDDLE FLANGE		1,620 \$	4,946
128	1	600	610	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		7,935 \$	8,345
129	1	600	610	53.50	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	87,080
130	1	600	610	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		7,935 \$	8,345
131	1	600	610	6.90	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		1,620 \$	11,178
151	1	80	-	-	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED, OTHER END WELDED TO BLIND 600NB FLANGE		960 \$	960
152	1	80	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	1,000 \$	1,000
153	1	80	-	-	DUCTILE IRON	COMBINATION AIR RELEASE VALVE	VAL-MATIC VM-200 C-S	1,600 \$	1,600
161	10	600	-	-	GALV. STEEL	VERTICAL ACTING PIPE SUPPORT		4,500 \$	45,000
162	1	600	-	-	GALV. STEEL	ELBOW PIPE SUPPORT		4,500 \$	4,500
163	1	600	-	-	GALV. STEEL	FLANGE ANCHOR PIPE SUPPORT		4,500 \$	4,500
164	1	600	-	-	GALV. STEEL	HORIZONTAL ACTING PIPE SUPPORT		3,000 \$	3,000
171	1	600	610	-	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	410
172	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	6,000 \$	6,000
173	1	600	-	-	DUCTILE IRON	DISMANTLING JOINT	TYCO THRUST TYPE DISMANTLING JOINT	2,200 \$	2,200
174	1	600	610	0.50	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		1,620 \$	896
175	1	600 / 300	610 / 324	0.85	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		8,100 \$	6,885

Hospital Prince of Wales Reservoir Pipe Tunnel Mechanical Coating Schedule

ITEM	QTY	NOM. DIA. (mm)	PIPE OD (mm)	SPOOL LENGTH (m)	MATERIAL	DESCRIPTION	MODEL	Rate (\$/item/length)	Cost
176	1	600	610	0.30	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 1,620	\$ 486
201	1	300	324	30	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 24,300
202	1	300	324	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 2,018	\$ 2,018
203	1	300	324	2.55	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 2,066
204	1	300	324	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 2,018	\$ 2,018
205	1	300	324	0.2	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 162
206	1	300	-	-	COATED STEEL	COMPRESSION COUPLING	VIKING JOHNSON COUPLING	\$ 2,023	\$ 2,023
207	1	300	324	0.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 243
208	1	300	324	0.5	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		\$ 3,300	\$ 2,110
209	1	300	324	2.25	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 2,063
210	1	300	-	-	ELASTOMER LINED	MAGNETIC FLOWMETER	ABB WATERMASTER 24V DC	\$ 14,000	\$ 14,000
211	1	300	324	3.5	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 3,065
212	1	300	324	0.5	CONC. LINED STEEL	EQUAL TEE, ALL ENDS PLAIN		\$ 3,300	\$ 1,650
213	1	300	324	-	CONC. LINED STEEL	90° ELBOW, BOTH ENDS PLAIN		\$ 4,035	\$ 4,035
214	1	300	324	0.5	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 405
215	1	300	324	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 2,018	\$ 2,018
216	1	300	324	5.7	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 4,847
217	1	300	-	-	RUBBER	SINGLE SPHERE BELLOW	JAMES WALKER TOWNSON	\$ 2,700	\$ 2,700
218	1	300	324	0.8	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$ 810	\$ 1,108
219	1	300	-	-	CAST IRON	ACTUATED RESILIENT SEATED PLUG VALVE	VAL-MATIC 5800R W/ IP68 ROTORK IQ	\$ 8,000	\$ 8,000
220	1	300	324	0.6	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$ 810	\$ 946
221	1	300	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$ 2,400	\$ 2,400
222	1	300	324	3.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 2,660
223	1	300	324	-	CONC. LINED STEEL	90° ELBOW, BOTH ENDS PLAIN		\$ 4,035	\$ 4,035
224	1	300	324	2.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED WITH PUDDLE FLANGE		\$ 810	\$ 1,850
225	1	300	324	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 4,035	\$ 4,265
226	1	300	324	20.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 16,430
227	1	300	324	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 4,035	\$ 4,265
228	3	300	324	32.0	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 9,770
229	2	300	324	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 4,035	\$ 8,300
230	1	300	324	1.0	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 810
241	1	300	324	1.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 3,040
242	1	300	-	-	DUCTILE IRON	DISMANTLING JOINT	TYCO THRUST TYPE DISMANTLING JOINT	\$ 1,200	\$ 1,200
243	1	500	-	-	DUCTILE IRON	PRESSURE REDUCING VALVE	CIA-VAL 90-07/690-01 OR BERHAD MODEL 720	\$ 8,000	\$ 8,000
244	1	300	324	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 4,035	\$ 4,265
251	1	80	-	0.25	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED, OTHER END WELDED TO BLIND 300NB FLANGE		\$ 660	\$ 165
252	1	80	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	\$ 1,000	\$ 1,000
253	1	80	-	-	DUCTILE IRON	COMBINATION AIR RELEASE VALVE	VAL-MATIC VM-200-C-S	\$ 1,600	\$ 1,600
261	8	300	-	-	GALV. STEEL	VERTICAL ACTING PIPE SUPPORT		\$ 3,000	\$ 24,000
262	1	300	-	-	GALV. STEEL	ELBOW PIPE SUPPORT		\$ 3,000	\$ 3,000
263	1	300	-	-	GALV. STEEL	FLANGE ANCHOR PIPE SUPPORT		\$ 3,000	\$ 3,000
264	1	300	-	-	GALV. STEEL	HORIZONTAL ACTING PIPE SUPPORT		\$ 2,250	\$ 2,250
301	1	300	324	30	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 24,300
302	1	300	324	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 2,018	\$ 2,018
303	1	300	324	2.55	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 2,066
304	1	300	324	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 2,018	\$ 2,018
305	1	300	324	0.2	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 810	\$ 162
306	1	300	-	-	COATED STEEL	COMPRESSION COUPLING		\$ 2,023	\$ 2,023
307	1	300	324	0.4	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 810	\$ 554
308	1	300	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$ 2,400	\$ 2,400

Hospital Prince of Wales Reservoir Pipe Tunnel Mechanical Costing Schedule

ITEM	QTY	NOM. DIA. {mm}	PIPE OD {mm}	SPOOL LENGTH (m)	MATERIAL	DESCRIPTION	MODEL	Rate (\$/item/length)	Cost
309	1	300	324	0.5	CONC. LINED STEEL	EQUAL TEE, ONE END FLANGED, BRANCH FLANGED		\$	230
310	1	300	324	0.5	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	63s
311	1	300	-	-	DUCTILE IRON	SWING CHECK VALVE	VAL-MATIC SWING FLEX 500	\$	1,600
312	1	300	324	2.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	2,323
313	1	300	-	-	ELASTOMER LINED	MAGNETIC FLOWMETER	ABB WATERMASTER 24V DC	\$	14,000
314	1	300	324	4.1	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	3,553
315	1	300	324	0.5	CONC. LINED STEEL	EQUAL TEE, ALL ENDS PLAIN		\$	3,300
316	1	300	324	0.3	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	473
317	1	300	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$	2,400
318	1	300	324	0.3	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	473
351	1	80	-	0.25	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED, OTHER END WELDED TO BLIND 300NB FLANGE		\$	660
352	1	80	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	\$	1,000
353	1	80	-	-	DUCTILE IRON	COMBINATION AIR RELEASE VALVE	VAL-MATIC VM-200-C-5	\$	1,600
351	2	300	-	-	GALV. STEEL	VERTICAL ACTING PIPE SUPPORT	(COVERED ELSEWHERE IN BUDGET)	\$	6,000
401	1	900	914	0.00	CONC. LINED STEEL	PIPE SPOOL, PLAIN ENDS		\$	
402	1	600 / 900	610 / 914	-	CONC. LINED STEEL	ECCENTRIC REDUCER, PLAIN ENDS		\$	10,455
403	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$	3,968
404	1	600	610	2.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$	3,240
405	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$	3,968
406	1	600	610	0.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$	486
407	1	600	-	-	COATED STEEL	COMPRESSION COUPLING	VIKING JOHNSON COUPLING	\$	7,194
408	1	600	610	0.30	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$	486
409	1	600	610	0.85	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		\$	8,100
410	1	600	610	0.70	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	1,544
411	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$	6,000
412	1	600	610	3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	5,680
413	1	600	-	-	ELASTOMER LINED	MAGNETIC FLOWMETER	ABB WATERMASTER 24V DC	\$	20,000
414	1	600	610	3.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	3,620
415	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$	6,000
416	1	600	610	0.75	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	1,620
417	1	600	610	0.85	CONC. LINED STEEL	EQUAL TEE, ALL ENDS PLAIN		\$	8,100
418	1	600	610	1.00	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	2,030
419	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$	6,000
420	1	600	610	0.4	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	1,620
421	1	600	-	-	RUBBER	SINGLE SPHERE BELLOW	JAMES WALKER TOWNSON	\$	6,000
422	1	600	610	1.0	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	1,620
423	1	600	-	-	CAST IRON	ACTUATED RESILIENT SEATED PLUG VALVE	VAL-MATIC 5800R W/ IP68 ROTORK IQ	\$	13,000
424	1	600	610	1.0	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	2,440
425	1	600	-	-	CAST IRON	LUGGED BUTTERFLY VALVE	VAL-MATIC SERIES 2000	\$	6,000
426	1	600	610	0.6	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$	1,620
427	1	600	610	3.5	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$	6,080
428	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$	3,968
429	1	600	610	3.5	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED WITH PUDDLE FLANGE		\$	1,620
430	1	600	-	-	-	VORTEX PREVENTER WITH GRILL		\$	2,600
451	1	80	-	0.25	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED, OTHER END WELDED TO BLIND 600NB FLANGE		\$	960
452	1	80	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	\$	1,000
453	3	80	-	-	DUCTILE IRON	COMBINATION AIR RELEASE VALVE	VAL-MATIC VM-200-C-5	\$	1,600
461	6	600	-	-	GALV. STEEL	VERTICAL ACTING PIPE SUPPORT		\$	4,500

Hospital Prince of Wales Reservoir Pipe Tunnel Mechanical Costing Schedule

ITEM	QTY	NOM. DIA. (mm)	PIPE OD (mm)	SPOOL LENGTH (m)	MATERIAL	DESCRIPTION	MODEL	Rate (\$/mcm/length)	Cost
501	1	600	610	0.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN	(COVERED ELSEWHERE IN BUDGET)	\$ 1,620	\$ -
502	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 3,968	\$ 3,968
503	1	600	610	2.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 1,620	\$ 3,240
504	1	600	610	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 3,968	\$ 3,968
505	1	600	610	0.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 1,620	\$ 486
506	1	600	610	-	COATED STEEL	COMPRESSION COUPLING		\$ 7,194	\$ 7,194
507	1	600	610	0.30	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 1,620	\$ 486
508	1	600	610	0.85	CONC. LINED STEEL	EQUAL TEE, BRANCH FLANGED, BOTH ENDS PLAIN		\$ 8,100	\$ 7,705
509	1	600	610	11.30	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 1,620	\$ 18,716
510	1	600	-	-	RUBBER	SINGLE SPHERE BELLOW	JAMES WALKER TOWNSON	\$ 6,000	\$ 6,000
511	1	600	610	0.8	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 1,620	\$ 1,706
512	1	600 / 500	610 / 508	0.85	CONC. LINED STEEL	BRANCH TEE, ALL ENDS PLAIN		\$ 7,250	\$ 6,197
513	1	600	610	3.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 1,620	\$ 5,270
514	1	600	610	3.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 1,620	\$ 5,270
515	1	600	610	-	CONC. LINED STEEL	90° ELBOW, BOTH ENDS PLAIN		\$ 7,935	\$ 7,935
516	1	600	610	2.8	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED WITH PUDDLE FLANGE		\$ 1,620	\$ 4,946
517	1	600	610	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 7,935	\$ 8,345
518	1	600	610	20.0	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 3,620	\$ 32,810
519	1	600	610	-	CONC. LINED STEEL	90° ELBOW, ONE END FLANGED		\$ 7,935	\$ 8,345
520	1	600	610	10.0	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 1,620	\$ 16,200
521	1	600	1800	2.4	CONC. LINED STEEL	BELLMOUTH	1.8m DIAMETER FABRICATED CONE	\$ 6,000	\$ 14,400
530	1	500	508	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 3,030	\$ 3,030
531	1	500	508	0.3	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED	VAL-MATIC SERIES 2000	\$ 1,200	\$ 740
532	1	500	508	-	CAST IRON	LUGGED BUTTERFLY VALVE		\$ 5,000	\$ 5,000
533	1	500	508	1.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED	VAL-MATIC SERIES 2000	\$ 1,200	\$ 2,320
534	1	500	508	-	CAST IRON	LUGGED BUTTERFLY VALVE		\$ 5,000	\$ 5,000
535	1	500	508	0.3	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS FLANGED		\$ 1,200	\$ 1,120
536	1	500	508	3	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 1,200	\$ 3,980
537	1	500	508	-	CONC. LINED STEEL	45° ELBOW, BOTH ENDS PLAIN		\$ 3,030	\$ 3,030
538	1	500	508	7.8	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN WITH PUDDLE FLANGE		\$ 1,200	\$ 9,360
539	1	500	-	-	CONC. LINED STEEL	VORTEX PREVENTER WITH GRILL		\$ 2,200	\$ 2,200
551	1	80	-	0.25	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED, OTHER END WELDED TO BLIND 600MM FLANGE		\$ 960	\$ 240
552	1	80	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	\$ 1,000	\$ 1,000
553	1	80	-	-	DUCTILE IRON	COMBINATION AIR RELEASE VALVE	VAL-MATIC VM-200 C-5	\$ 1,600	\$ 1,600
561	5	500	-	-	GALV. STEEL	VERTICAL ACTING PIPE SUPPORT		\$ 4,500	\$ 22,500
601	4	100	114	0.50	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 300	\$ 675
602	4	100	-	-	DUCTILE IRON	RESILIENT SEATED GATE VALVE	TYCO FIGURE 500	\$ 1,200	\$ 4,800
603	4	100	114	2.00	CONC. LINED STEEL	PIPE SPOOL, ONE END FLANGED		\$ 300	\$ 2,475
604	3	100	114	0.20	CONC. LINED STEEL	EQUAL TEE, ALL ENDS PLAIN		\$ 500	\$ 540
605	2	100	114	-	CONC. LINED STEEL	90° ELBOW, BOTH ENDS PLAIN		\$ 450	\$ 900
606	2	100	114	1.00	CONC. LINED STEEL	PIPE SPOOL, BOTH ENDS PLAIN		\$ 300	\$ 600
Subtotal \$ 1,050,000									
Contingency (20%) \$ 210,000.0									
Total \$ 1,260,000									

Item 9.0 - Electrical and Controls

ITEM	DESCRIPTION	Cost
9.1	Greater Wellington estimate including 15% contingency and accuracy of +/- 15%	\$ 339,130
Total		<u>\$ 340,000</u>

Note: Above pricing and modification is from email from Keith Woolley to Simon Edmonds on 14 September 2012
Allowance for P&G, margin, estimating contingency and contract contingency will increase this by a factor of approximately 1.6

Includes: Power supply from Rolleston St to the pipe tunnel
Switchboard and metering inside the tunnel
Power from the main switchboard to all actuators and instruments
Telemetry and SCADA panel and control cabling to instruments inside tunnel, reservoir, door and access hatch.

Item 10.0 - Backfill and Access Driveway

ITEM	DESCRIPTION	QTY (m ³ or m ²)	Rate (\$/unit)		Cost
10.1	Engineered backfill from stockpile	25,000	\$	4.13	\$ 103,250
10.2	Engineered backfill from offsite (Horokiwi Quarry at \$8.84/m ³)	5,500	\$	24.44	\$ 134,420
10.3	Permanent access road to pipe tunnel entrance 120m long, 4.0m wide surfaced with gobi blocks. Includes formation and bedding.	480	\$	130.00	\$ 62,400
Total					<u>\$ 300,070</u>

Item 11.0 - Topsoiling and Landscaping

ITEM	DESCRIPTION	QTY (m ³ or m ²)	Rate (\$/unit)	Cost
11.1	Surface landscaping	10,000	\$ 15.00	\$ 150,000
11.2	Allowance for local temporary fencing, additional planting and plant replacement over the first 2 to 4 years	1	\$ 50,000.00	\$ 50,000
11.3	Clay material topsoil subbase from stockpile	1,500	\$ 4.13	\$ 6,195
11.4	Top soiling from stockpile	316	\$ 4.82	\$ 1,523
11.5	Top soiling imported	1,184	\$ 62.86	\$ 74,426
Total				<u>\$ 282,144</u>

Notes:

- 1 Scope includes topsoiling over reservoir and pipe tunnel and associated landscaping.

Item 12.0 - Overflow/Stormwater Drainage

ITEM	DESCRIPTION	QTY (m)	Rate (\$/unit)	Cost
12.1	300mm dia. uPVC pipe, 2-2.5m deep	240	\$ 300.00	\$ 72,000
12.2	600mm dia. RCRRJ pipe, 2-3.0m deep	115	\$ 580.00	\$ 66,700
12.3	Connect 600mm dia. RCRRJ pipe to existing manhole	1	\$ 2,000.00	\$ 2,000
12.4	Remove existing 375mm dia. RCRRJ pipe, approx 105m long	1	\$ 3,480.00	\$ 3,480
12.5	Remove existing 300mm dia. RCRRJ pipe approx. 50m long and 1 No. manhole and backfill with excavated material	1	\$ 4,060.00	\$ 4,060
12.6	Remove existing 100mm dia. EW pipe approx. 10m long and 1 No. manhole and backfill with excavated material.	1	\$ 2,900.00	\$ 2,900
12.7	1050mm dia. precast concrete manhole, 2m deep	2	\$ 3,955.60	\$ 7,911
12.8	1350mm dia. precast concrete manhole, 2.4m deep	2	\$ 5,568.00	\$ 11,136
12.9	1350mm dia. precast concrete manhole, 2.4m deep, on road verge	3	\$ 5,800.00	\$ 17,400
12.10	1500mm dia. precast concrete manhole with scruffy dome, 2.4m deep	1	\$ 7,540.00	\$ 7,540
12.11	Remove 1 No. manhole	2	\$ 2,320.00	\$ 4,640
12.12	WCC standard yard sump with 200mm thick concrete apron at surface, including sump lead and saddle connection to PVC pipe	8	\$ 2,204.00	\$ 17,632
12.13	Precast concrete headwall for 600mm dia. RCRRJ pipe with galvanised steel grate	1	\$ 4,060.00	\$ 4,060
12.14	2500mm wide swale including grassing and 100mm thick topsoil	220	\$ 30.00	\$ 6,600
12.15	0.3m thick, 2m wide rock rip rap protection all around 1500mm dia. manhole and Bidum A34 geotextile	1	\$ 1,740.00	\$ 1,740
12.16	5m x 3m x 0.3m rock rip rap protection formed to suit ground levels including excavation, cart away surplus materials and Bidum A34 geotextile layer	1	\$ 1,044.00	\$ 1,044
			Total	<u>\$ 230,843</u>

Notes:

- 1 Scope includes drainage upgrades to upper park sports ground.

Item 13.0 - Upper Sports Ground Resurfacing

ITEM	DESCRIPTION	QTY (units)	Rate (\$/unit)	Cost
13.1	Imported topsoil (field reinstatement)	1245	\$ 62.86	\$ 78,261
13.2	Travelling irrigation unit (Note - Not preferred)	1	\$ 18,750.00	\$ 18,750
13.3	Irrigation water supply 40mm HDPE, 150m long connection to Rolleston St with two hose taps and backflow preventer			\$ 31,250
			Total	<u>\$ 128,261</u>

Notes:

- 1 Upper park sports ground drainage is included in Item 12.0
- 2 No allowance for concrete swale drains along sides of sports field.
- 3 In field irrigation is preferred by WCC but the costs have not be determined at this stage. A cost allowance is included for a travelling irrigation unit for this concept estimate.

Item 14.0 - Rolleston Street Pavement Repairs

ITEM	DESCRIPTION	QTY (units)	Rate (\$/unit)	Cost
14.1	Allowance for post construction pavement repairs in Rolleston Streets.	1	\$150,000.00	\$ 150,000
			Total	<u>\$ 150,000</u>

Appendix C

Structural Basis of Design

Basis of Structural Design

Hospital Prince of Wales Reservoir Structural Basis of Design

Prepared for Wellington City Council (WCC)

By CH2M Beca Limited (Beca)

1 February 2013



Revision History

Revision N°	Prepared By	Description	Date
A	Juvenal Verano	For Client Review	05.10.12
B	Juvenal Verano	Updated for CDOR	01/02/2013

Document Acceptance

Action	Name	Signed	Date
Prepared by	Juvenal Verano		01/02/2013
Reviewed by	Simon Edmonds		01/02/2013
Approved by	Dennis Hunt/Mark Sneddon		01/02/2013
on behalf of	CH2M Beca Limited		

Table of Contents

1	Introduction.....	1
2	Objectives	1
3	Definitions	1
4	Scope of Structural Design	3
5	Reference Documents	3
6	Specific WCC Requirements	3
6.1	General Requirements	4
6.2	Structural Requirements.....	4
7	Reservoir Structural Description	4
7.1	Structural System	4
7.2	Roof	4
7.3	Floor Slab	5
7.4	Walls.....	5
7.5	Foundations.....	5
8	Design Standards & Guidelines.....	5
9	Material Properties	6
9.1	Reinforced Concrete.....	6
9.2	Prestressed Concrete.....	7
9.3	Structural Steel	7
10	Design Criteria and Loads	7
10.1	General	8
10.2	Permanent Loads, G	8
10.3	Imposed Loads, Q	8
10.4	Earth Pressure, F_e	8
10.5	Hydrostatic Pressure, Fl	8
10.6	Ground Water, Gw.....	9
10.7	Earthquake Loads, E	9
10.8	Swelling, Sw	9
10.9	Shrinkage, Sh	9
10.10	Temperature, T	10
10.11	Construction Load	10
11	Load Combinations	10
11.1	Roof slab, beams and columns	10
11.2	Floor slab, Wall and Wall Joints	10
11.3	Foundation.....	11
11.4	Associated Structures.....	11
11.5	Stability	11

12 Allowable Deflections..... 12

13 Durability Requirements..... 12

14 Water Tightness Requirements..... 12

    14.1 Floor Slab Waterstops 13

    14.2 Wall Section Waterstops 13

    14.3 Roof Slab Waterstops..... 13

15 Geotechnical Information..... 13

1 Introduction

This document describes the basis of structural design for **The Hospital Prince of Wales Reservoir Project** located in upper Prince of Wales Park at Mount Cook, Wellington City.

The project includes a reservoir and the associated earthworks and pipelines. The reservoir is fully buried below ground level and will store 35,000m³ volume of water. It will supply water to Wellington Hospital and to the Central Business District (CBD). CH2M Beca (Beca) has been commissioned by Wellington City Council (WCC) to supply engineering services for the development of the new reservoir.

2 Objectives

The principal objective of this document is to establish a basis that shall be used for the structural design and documentation works of the reservoir and pipe tunnel and to:

- Comply with client requirements
- Comply with statutory requirements
- Adopt a sound design philosophy
- Utilise the relevant experience and skills of the design team members
- Adopt the latest engineering technology
- Enable close coordination with other design disciplines
- Permit construction sequencing to be undertaken in accordance with the client's agreed programme

3 Definitions

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

B2	Exposure Classification
CBD	Central Business District
Capacity	Capacity Infrastructure
Council	Wellington City Council
CCANZ	Cement and Concrete Association of New Zealand
CoG	Centre of Gravity
DL	Dead load
EL _U	Ultimate limit state earthquake load
EL _S	Serviceability limit state earthquake load
Eu	Ultimate limit state earthquake load.

F_l	Fluid pressure loading
F_x	Force in x direction
F_y	Force in y direction
F_z	Force in z direction
G	Dead load
GWRC	Greater Wellington Regional Council
HERA	NZ Heavy Engineering Research Association
HPOW	Hospital Prince of Wales
N.A.	Not applicable
NZBC	New Zealand Building Code
NBS	New Building Standard
LL	Live Load
POW	Prince of Wales Park
Q	Live Load
RFP	Request for Proposal
OSH	Occupational Health and Safety Department of Labour
Sh	Shrinkage effects loading
SLS1	Serviceability limit state for structures of Importance Levels 1, 2 or 3
SLS2	Serviceability limit state for Importance Level 4 – Operational Continuity
SW	Swelling effects loading
TBC	To Be Confirmed
TWL	Top Water Level
UDL	Uniformly Distributed Load
ULS	Ultimate limit state
WCC	Wellington City Council
WE	Wellington Electricity
W_{L_u}	Ultimate limit state wind load
W_{L_s}	Serviceability limit state wind load
γ_c	Live load combination factor
μ	Ductility factor

ε

Damping factor

4 Scope of Structural Design

Beca will prepare structural design calculation, detailed drawings and specifications to demonstrate compliance with the relevant codes, standards and guidelines outlined in this brief. The scope of structural design work shall cover the reservoir and pipe tunnel including but not limited to the following:

- Foundations
- Floor slabs
- Walls
- Roof support columns
- Roof support beams
- Roof slabs
- Columns
- Reservoir pipe tunnel roof, walls and floor
- Access hatches into the reservoir (roof and floor)
- Pipe supports inside the reservoir and pipe tunnel
- Access stairs and landings
- Access ladders and platforms
- Guard rails and handrails

5 Reference Documents

The following documents are to be referred to in the design of the Hospital Prince of Wales Reservoir.

This document shall be read in conjunction with the following documents:

- Hospital Prince of Wales Reservoir Geotechnical Basis of Design (Beca, Rev B dated 1 February 2013)
- Hospital Prince of Wales Reservoir Mechanical Basis of Design (Beca, Rev 1 dated 1 February 2013)
- 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)
- Capacity Infrastructure Services Request for Tender for the Consultancy Services for the Hospital Prince of Wales Reservoir (2012).

6 Specific WCC Requirements

The following WCC requirements are specified in the RFP document and are listed below for the purpose of outlining the general and structural requirements which are deemed critical for the structural design.

6.1 General Requirements

- Reservoir is to be located in the Upper Prince of Wales Park, Mt Cook, Wellington.
- The required reservoir capacity is 35,000m³ to be stored below the TWL of 92.00m (New City Datum).
- Reservoir to be fully buried.
- Have roof hatches (non-venting) in all four quadrants of the reservoir adjacent pipework discharge points (overflow, primary inlet, high pressure inlet and the fourth quadrant) with an access ladder and platforms below at least one of the roof hatches.
- Incorporate a continuous membrane across the reservoir roof.
- Incorporate a bitumen emulsion or equivalent membrane product on the external face of the reservoir walls.
- Completed a successful reservoir water tightness test prior to backfilling the reservoir or sealing the external face of the reservoir walls.

6.2 Structural Requirements

- Reservoir Design Life to be either 100 or 150 years (WCC to advise selection at preliminary design stage).
- The reservoir shall be designed for SLS2 seismic loads equivalent to a 1000 year return period event (HOLD Capacity to confirm seismic design standard). This is to provide equivalence or exceed the seismic design standards for the Wellington Regional Hospital to provide operational continuity within six hours of a 1000 year return period earthquake.
- Allowance for excavation around any point of the perimeter of the reservoir in the future.
- A minimum of four sumps shall be positioned in the floor located opposite to inlets. Sumps to have grates and connect to the sewer pipe system.
- The reservoir structure shall be designed to withstand a water pressure of at least one meter above the top level of the reservoir walls.
- The roof shall be designed to take the required loading from landscaping activities, tractor mowing and use of an excavator (at least 10 tonnes) for future maintenance.
- A minimum of 300mm of turf including top soil shall be provided over the roof the reservoir on top of any site concrete layer protecting the waterproofing membrane layer below.
- The reservoir walls shall be either cast in-situ, pre-stressed or precast concrete panels.
- The lowest roof beam must be 300mm above TWL or 50mm above the maximum water level when the reservoir is discharging via the overflow system at a flow rate of 1200L/s (250mm above TWL).
- The minimum level of the roof slab shall be at least 450mm above the TWL.
- Pipe tunnel/gallery to have at least 2.2m clear internal height.

7 Reservoir Structural Description

7.1 Structural System

7.2 Roof

- Minimum reservoir floor and roof grades are to be 1 in 100.

Either an orthogonal column and beam layout or a circumferential column and beam layout depending on selected option. Reservoir Option R1.0 could have either an orthogonal or

circumferential roof structure layout. Reservoir Option R3.1 would dictate a circumferential roof structure layout.

7.3 Floor Slab

- Nominally 250 thick slab. Either conventionally reinforced or post tensioned construction.

7.4 Walls

Nominally 425 thick (based on SLS2 1000 year earthquake return period), prestressed vertically and post tensioned horizontally. 8 No. Pilasters; circumferential stressing in four overlapping tendons around reservoir.

7.5 Foundations

- Continuous strip foundations normally two metres wide below reservoir wall. Local slab thickenings 500mm deep at column positions.

8 Design Standards & Guidelines

The following standards (and manual) shall be used in the design of the structures stated in this document.

AS/NZS1170.0	General Principles
AS/NZS1170.1	Permanent, Imposed and other Actions
AS/NZS1170.2	Wind Actions
AS/NZS1170.3	Snow and Ice Actions
NZS1170.5	Earthquake Actions - New Zealand
NZS3106	Design of Liquid Retaining Structures
AS4678:2002	Earth-retaining Structures
NZS3101.1	Concrete Structures Standard
NZS3404.1	Steel Structures Standard
NZS 3109	Concrete Construction
NZS 3121	Water and aggregate for concrete
NZS 3114	Concrete surface finishes
NZS 3104	Specification for concrete production
NZS 3122	Specification for Portland and blended cements
AS/NZS 4671	Steel Reinforcing Material
AS/NZS 4672	Steel prestressing materials
AS 1310	Steel wire for tendons in prestressed concrete
AS1311	Steel tendons for prestressed concrete

AS 3678	Hot-rolled structural steel plates, floorplates and slabs
AS 3679.1	Hot-rolled structural steel bars and sections
AS 3679.2	Welded Sections
AS/NZS 1252	High Strength Bolts for Structural Engineering
AS/NZS 2451	Bolts, screws and nuts
AS/NZS 1553.1	Covered Electrodes for Welding
AS/NZS 1554.1	Welding of Steel Structures
AS/NZS 1554.2	Welding of Stainless Steel Structures
CCANZ Publication TM 34 Tilt-up Technical Manual	
Transit New Zealand (TNZ) Bridge Manual	
RRU 83 – Seismic Design of Bridge Abutments	
New Zealand Building Act 2004	
New Zealand Standard Act 1988	

9 Material Properties

The following material properties shall be used in the design of the structural elements including components and connections.

9.1 Reinforced Concrete

9.1.1 Concrete

The minimum compressive strength of concrete to be used in design of ordinary reinforced concrete element shall be:

- Reservoir (roof, beam, wall, base-slab, column & foundation) $f_c' = 40\text{MPa}$
- Reservoir access hatch upstands $f_c' = 40\text{MPa}$
- Pipe tunnel $f_c' = 40\text{MPa}$
- Valve chambers $f_c' = 30\text{MPa}$
- Pipe encasement $f_c' = 30\text{MPa}$
- Stair and platform foundation $f_c' = 30\text{MPa}$
- Pipe supports and thrust blocks $f_c' = 30\text{MPa}$
- Retaining walls around entrance to pipe tunnel $f_c' = 30\text{MPa}$

9.1.2 Reinforcing Steel

The minimum yield strength of reinforcing bar to be used in the design shall be:

- Deformed bar designated 'DH' shall be grade 500E $f_y = 500\text{MPa}$
- Plain round bar designated 'RH' shall be grade 500E $f_y = 500\text{MPa}$
- Plain round bar designated "R" shall be grade 300E $f_y = 300\text{MPa}$

9.2 Prestressed Concrete

If prestressed concrete shall be used in the design of reservoir the following material properties shall be used.

9.2.1 Concrete

The maximum compressive strength of prestressed concrete shall be determined in conjunction with the concrete supplier and precaster but is likely to be no more than 55 MPa. The minimum compressive strength of concrete to be used in design of prestressed concrete element shall be:

- Roof slab, floor slab, roof support beams, walls $f_c' = 40\text{MPa}$

9.2.2 Prestressing Strands

The minimum tensile strength of prestressing strand (tendons) to be used in the design of prestressed concrete shall be:

- VSL Super Grade (or equivalent) $F_{p_u} = 1840\text{MPa}$
- Strand diameters shall be either 12.7mm or 15.9mm with appropriate proprietary wedge anchorage systems.

9.3 Structural Steel

The minimum yield strength of steel (mild and stainless) structures, components and connections to be used in the designed shall be:

9.3.1 Steel Sections

- Hot rolled plates $f_y = 250\text{MPa}$
- Hot rolled structural sections (UB, UC, PFC, EA) $f_y = 300\text{MPa}$
- Hot rolled structural sections (RHS, SHS & CHS) $f_y = 350\text{MPa}$

9.3.2 Stainless Steel Sections

- Plates (TBC)
- Sections (TBC)

9.3.3 Bolts & Nuts & Washers

- High strength bolts grade 8.8 $f_u = 830\text{MPa}$
- Mild steel bolts grade 4.6 $f_u = 460\text{MPa}$
- Stainless steel bolts grade 316 (TBC)

9.3.4 Welding Consumables

- E48XX electrodes shall be specified for all welds $f_{uw} = 410\text{MPa}$

10 Design Criteria and Loads

The following criteria and loadings shall be used in the design of the reservoir structure and the associated structures stated in this brief.

10.1 General

As stated in Clause 3.4.4e of RFP, WCC requires that for design purposes the reservoir shall be considered as a significant post disaster storage facility which structural performance during and after major earthquake event are expected to exceed the requirements of Importance Level (IL) 4 structures with a special post disaster function. Associated structures are also expected to perform similar to the reservoir structure during and after a major seismic event.

10.1.1 Design Life

WCC's requirement for the new reservoir including the associated structures shall be designed for a minimum design life of 100 years. (HOLD – Capacity to confirm design working life as 100 or 150 years).

10.1.2 Importance Level

For seismic loading calculation, as specified in the AS/NZS 1170.0 Table 3.3, for 100 years design life the corresponding IL to which a structure must be designed is 4. The reservoir shall be designed assuming importance level (IL) 4 and with due consideration that its performance will meet the requirement mentioned in Clause 10.1 above.

10.2 Permanent Loads, G

Dead loads include the self-weight of all structural elements including fixed equipment. The following material densities were used throughout the design process.

- Concrete 25kN/m^3
- Steel 79kN/m^3
- Fresh Water 10kN/m^3
- Soil 20kN/m^3

10.3 Imposed Loads, Q

Generally, the reservoir shall be designed for the loading specified in NZS 3106:2009 and NZS 1170.1. During the normal operating condition the roof shall be designed assuming the following imposed load:

- Uniform roof live load 3.0kPa
- Excavator or mower 10.0 Tonnes (TBC)

10.4 Earth Pressure, F_e

The reservoir shall be fully buried below ground level. The earth pressure due to retained soil to be used in the design shall be:

- Unit weight of soil (Refer Geotechnical Basis of Design)
- Coefficient of friction angle (Refer Geotechnical Basis of Design)

The increase in soil pressure due to seismic loading shall be calculated using either NZS 3106; Mononobe-Okabe principle or equivalent principle or RRU 83 Seismic Design of Bridge Abutments.

10.5 Hydrostatic Pressure, Fl

The hydrostatic pressure due to retained fresh water to be used in design shall be based on the maximum height of retained water assuming the water level to be a minimum of one meter above the top of the reservoir wall. Refer Clause 6.2 of this brief for the specific client requirements. If a

permanently submerged roof design is adopted then the maximum height of retained water above the top of the walls will be used.

10.6 Ground Water, Gw

Hydrostatic pressure due to ground water shall be considered in the design of the reservoir including associated underground structures. The water table gradient shall be based on the highest piezometer water level recorded on site reducing to an effective residual pressure at the reservoir wall. This is based on the assumption that a drainage system is installed around the external perimeter of the reservoir.

10.7 Earthquake Loads, E

The following seismic parameters shall be used for the calculation of earthquake loads. The height of retained water to be used in the designed shall be one meter above the top of reservoir wall. Refer Clause 6.2 of this brief for the specific client requirements.

■ Hazard factor	Z = 0.4 For Wellington City
■ Site sub-soil class	Soil Class A/B (Refer SSSHA)
■ Annual probability of exceedance	P = 1/2500; 1/5000 or 1/7500 for ULS (TBC)
■ Annual probability of exceedance	P = 1/1000 for SLS ₂ (Operational continuity) (HOLD TBC)
■ Annual probability of exceedance	P = 1/500 for SLS ₂ (Code requirement)
■ Return period factor	Ru for ULS (Refer SSSHA recommendations)
■ Return period factor	Rs = 1.3 For SLS ₂ (Operational continuity)
■ Return period factor	Rs = 1.0 For SLS ₂ (Code requirement)
■ Near fault factor	N(Ti,D) = (Refer SSSHA recommendations)
■ Structural performance factor	Sp = 1.0 for ULS & SLS ₂ (for reservoir)
■ Structural performance factor	Sp = 0.925 for ULS (for pipe tunnel structure)
■ Structural performance factor	Sp = 0.7 for SLS (for pipe tunnel structure)
■ Ductility factor	μ = 1.25 for ULS
■ Ductility factor	μ = 1.0 for SLS
■ Damping factor	ε = 5% for ULS
■ Damping factor	ε = 0.5% for SLS

The increase in liquid pressure (hydrodynamic) due to seismic accelerations shall be calculated as per Appendix A of NZS3106:2009.

10.8 Swelling, Sw

The effect due to swelling shall be considered in the design of the reservoir structure. In the absence of a rational analysis, the minimum effects due to moisture variation shall be determined considering the swelling strains specified in Table 2, Clause 4.2.4 of NZS 3106:2009

10.9 Shrinkage, Sh

The effect due to shrinkage shall be considered in the design of the reservoir structure. In the absence of a rational analysis, the minimum effects due to moisture variation shall be determined considering the shrinkage strains specified in Table 2, Clause 4.2.4 of NZS 3106:2009

10.10 Temperature, T

The effect due to increase or decrease in temperature and due to differential temperature gradient shall be considered in the design of the reservoir for the walls for load cases prior to backfilling. The following changes in temperature and temperature gradient as per Clause 4.2.3 of NZS 3106:2009 shall be used in the design:

- Roof $\pm 20^{\circ}$ Celsius
- Roof temperature gradient is 5° Celsius per 100mm
- Wall $\pm 30^{\circ}$ Celsius
- Wall temperature gradient as per in NZS 3106:2009

The coefficient of thermal expansion of concrete used in design was $11 \times 10^{-6} / ^{\circ}$ Celsius

10.11 Construction Load

Allowance for load during construction shall be considered in the design. The roof and its immediate supports shall be designed assuming a minimum construction live load of 2.0kPa (TBC).

11 Load Combinations

In general, the load combinations specified in NZS3106:2009 shall be used in the design of reservoir structure. The load combinations specified in AS/NZS 1170.1 shall be used in the design of the associated structures (pipe tunnel). The following load combinations shall be used:

11.1 Roof slab, beams and columns

The roof slab of the reservoir including its immediate support shall be designed considering the load combinations specified Transit New Zealand (TNZ) Bridge Manual.

11.1.1 Maintenance Condition

Design Standard: TNZ Bridge Design Manual

- $1.35G + 1.35(1.67LLxI)$ At ULS Where: $I = 1.3$ Dynamic amplification factor
- $1.0G + 1.35LLxI$ At SLS

11.1.2 Operating Condition

Design Standard: NZS 3106:2009

- $1.35G$ At ULS
- $1.2G + 1.5Q$ At ULS
- $G + Flp + Sh$ SLS – Group A
- $G + T + 0.7Sh$ SLS – Group B1
- $G + Flp + Es_1$ SLS – Group B2

11.2 Floor slab, Wall and Wall Joints

The following load combinations shall be used in the design of floor slabs, walls and wall joints.

11.2.1 Serviceability

- $G + Flp + 0.5Sw$ Group A

- $G + Flp + T$ Group B
- $G + Flp + Es1$ Group B
- $G + T + 0.7Sh$ Group B

11.2.2 Ultimate

- $1.35G$
- $1.2G + 1.5Q$
- $1.0G + 1.0Eu$

11.3 Foundation

The following load combinations shall be used in the design of foundations.

11.3.1 Serviceability

- $G + Flp + 0.5Sw$
- $G + Flp + T$
- $G + Flp + Es1$
- $G + T + 0.7Sh$
- $G + Q + \gamma_c Q$

11.3.2 Ultimate

- $1.35G$
- $1.2G + 1.5Q$
- $1.0G + 1.0Eu$

11.4 Associated Structures

The following load combinations shall be used in the design of other structures.

11.4.1 Serviceability

- $G + Q$
- $G + Ws + \gamma_c Q$
- $G + Es$

11.4.2 Ultimate

- $1.35G$
- $1.2G + 1.5Q$
- $1.0G + 1.0Eu$

11.5 Stability

The load combinations specified in AS/NZS 1170.1 shall be used for the global stability checked against overturning and sliding of the reservoir.

- $0.9G + 1.0Eu$

12 Allowable Deflections

The following deflection criteria shall be used in the design of structural elements:

■ Roof slab	L/360
■ Roof beams	L/360
■ Cantilever retaining wall	H/150
■ Column lateral deflection	H/400

13 Durability Requirements

The following design criteria for concrete durability shall be used in the design of reservoir.

■ Minimum intended life span	100 or 150 years (HOLD – TBC by Capacity)
■ External exposure classification	B2
■ Internal exposure classification	B2
■ Minimum concrete cover for walls	50mm for outside surface
■ Minimum concrete cover for walls	50mm for inside surface
■ Minimum concrete cover for roof slab	40mm for top bars (covered by membrane)
■ Minimum concrete cover for roof slab	25mm for bottom bars
■ Minimum concrete cover for ground slab	50mm for top bars
■ Minimum concrete cover for ground slab	40mm for bottom bars (with site concrete)
■ Minimum concrete cover for foundation	40mm for bottom bars (with site concrete)
■ Minimum concrete cover for foundation	50mm for bottom bars
■ Other structure not part of reservoir	50mm

Note the above specification is applicable for a 100 year durability in accordance with NZS3101:2006. For a design life greater than 100 years specific design and assessment will be required for concrete mix design and cover requirements.

14 Water Tightness Requirements

The reservoir walls shall be classified as water-tight liquid retaining structure having a Tightness Class 3 as per Table 3 Clause 5.1.1 of NZS3106:2009. The reservoir floor shall be classified as having a Tightness Class or 1 or 2 (HOLD - TBC)

Post-tensioning of the concrete water retaining elements of the reservoir shall be used to achieve this Tightness Class as required for the selected reservoir form.

The floor slab shall be designed to either; limit the width of cracks due to combined flexural stresses and secondary stresses (temperature, swelling and shrinkage) by limiting the tensile stress of the reinforcing steel to 240 MPa. This requirement is as per clause 5.2.5 of NZS3106:2009 or by the application of post tensioning to the slab in segments and between segments to resist combined tensile flexural and secondary stresses by the application of compression stress across the full concrete section.

14.1 Floor Slab Waterstops

At specified locations joints in the floor slab will be constructed. These will be to limit concrete pour size and optimise the geometry of slab segments. Waterstops to be provided at floor slab joints include synthetic rubber hydrophilic waterstops within the depth of the slab section and PVC external waterstops at the underside of the slab. Formed recesses will be provided for the hydrophilic sealants in the surface of the precast panels. A bullnose finish to the slab edge at each side of the joint is proposed to provide a cleanable surface finish. In general surface applied polyurethane sealants are not proposed for floor joints but will be provided around pipe penetrations, floor access hatches and at the junction of the wall to the floor slab.

Column foundations are to be constructed integrally with the floor slab with floor slab joints located away from column positions. Reinforcing starter bars will be provided through the floor for columns above. Waterstops are not proposed at the construction joint positions between the top of floor slab and base of column sections.

14.2 Wall Section Waterstops

Only vertical construction joints shall be used for the reservoir walls. Waterstops at the wall infill sections between precast concrete panels will be synthetic rubber hydrophilic waterstops within the depth of the wall section. Formed recesses will be provided for the hydrophilic waterstops in the surface of the precast panels. Roughened concrete surface finish will be provided for the balance of the precast panel construction joint surface.

Continuity will be provided between the vertical wall hydrophilic waterstops in the wall joints and the horizontal hydrophilic waterstops between the wall and floor slab.

14.3 Roof Slab Waterstops

TBC. The roof slab shall be covered with a waterproof membrane.

15 Geotechnical Information

Applicable geotechnical information for the structural design of the reservoir is included in the Beca document Hospital Prince of Wales Reservoir – Geotechnical Basis of Design (Beca 2012).

Appendix D

Mechanical Basis of Design

Basis of Design

Hospital Prince of Wales Reservoir Mechanical Basis of Design

Prepared for Wellington City Council (Client)

By CH2M Beca Limited

1 February 2013



Revision History

Revision N°	Prepared By	Description	Date
A	Mike Yarrall	For Client Review	17/07/2012
0	Mike Yarrall	For Use	12/09/2012
1	Mike Yarrall	Updated for CDOR	01/02/2013

Document Acceptance

Action	Name	Signed	Date
Prepared by	Mike Yarrall		01/02/2013
Reviewed by	Darren Hewett		01/02/2013
Approved by	Simon Edmonds		01/02/2013
on behalf of	CH2M Beca Limited		

Table of Contents

1	Introduction.....	1
2	Objectives	1
3	Definitions	1
4	Process Description	2
5	Scope of Mechanical Design.....	2
6	Reference Documents	2
7	Mechanical Design Criteria	3
7.1	Flows and Hydraulics	3
7.2	Valves	5
7.3	Flexible Couplings	6
7.4	Pipe.....	6
7.5	Flanges	7
7.6	Instrumentation	7

Appendices

Appendix A - Process and Instrumentation Diagram and Legend Sheet

1 Introduction

This document describes the basis of design for the mechanical components for the **Hospital Prince of Wales Reservoir Project** located in upper Prince of Wales Park at Mount Cook, Wellington City.

The project includes a reservoir and the associated earthworks and pipelines. The reservoir is fully buried below ground level and will store 35,000m³ volume of fresh water. It will supply water to Wellington Hospital and to the Central Business District (CBD). CH2M Beca (Beca) has been commissioned by Wellington City Council (WCC) to supply engineering services for the development of the new reservoir.

2 Objectives

The principal objective of this document is to establish a basis that shall be used for the design and documentation works of all aspects of the reservoir mechanical and piping components and to:

- Comply with client requirements
- Comply with statutory requirements
- Address whole of life operation, inspection and component replacement requirements
- Utilise the relevant experience and skills of the design team members
- Incorporate consideration of recent engineering technology developments
- Enable close coordination with other design disciplines
- Permit construction sequencing to be undertaken in accordance with the client's agreed programme

3 Definitions

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

CBD	Central Business District
Capacity	Capacity Infrastructure
Council	Wellington City Council
GW	Greater Wellington regional Council
HPOW	Hospital Prince of Wales
RFT	Request for Tender
TBC	To Be Confirmed
TWL	Top Water Level
WCC	Wellington City Council
WE	Wellington Electricity

4 Process Description

The HPOW reservoir will be part of the 'Low Level Zone'. This includes the following reservoirs: Macalister, Carmichael and Mt Albert. The reservoirs in this zone are hydraulically connected through the reticulation system with the larger reservoirs having a Top Water Level (TWL) of 92m. The HPOW reservoir is filled from the new 800 NB inlet main on Hargreaves St and the Bell Rd Reservoir. The outlets are both to the Carmichael Reservoir and the new 900 NB outlet main on Hargreaves St.

In a seismic event the reservoir will automatically be isolated from the reticulation network. Flow control valves also serve to allow control over the filling rate of the reservoir as well as controlling the flow out of the reservoir.

5 Scope of Mechanical Design

The following areas are covered by the mechanical design:

- Above ground pipework, valves and fittings inside the tunnel and reservoir.
- Underground pipelines, valves and fittings from the tunnel to the connection to the new streetworks pipelines.
- Underground pipework, valves and fittings from the tunnel to the existing mains.
- Underground overflow and scour pipework, valves and fittings from the tunnel to the stormwater system.

Pipelines associated with the reservoir:

- Outlet pipeline 900 NB to Hargreaves Street.
- Primary inlet pipeline 800 NB from Hargreaves Street.
- Scour and overflow pipeline.
- Secondary inlet pipeline 300NB that connects to the existing high pressure inlet pipeline, 450 NB existing installed in 1954 from Bell Road reservoir
- Secondary outlet pipeline 300 NB that feeds the existing 375 NB that supplies Carmichael Reservoir.

Pipeline design will include, where necessary: manual valves, automated actuated valves, air valves, scour valves and pressure reducing valves.

6 Reference Documents

The following documents are to be referred to in the design of the Hospital Prince of Wales Reservoir:

- Capacity Infrastructure Services Request for Tender for the Consultancy Services for the Hospital Prince of Wales Reservoir 1 February 2012
- Capacity Infrastructure Services Approved Products Register 2011.04.01 (2011)
- Regional Standard for Water Services. Capacity Infrastructure Services. Rev A [DRAFT] – May 2012.
- Wellington City Council Pipe Networks requirements checklist for as-built drawings (2011)
- Wellington City Council Water Supply Specification 7th Edition (2004)

The following standards shall be used in the design of the Hospital Prince of Wales Reservoir:

- AS/NZS2280:2004 Ductile iron pipes and fittings
- AS/NZS2451:1998 Bolts, screws and nuts with British Standard Whitworth threads (rationalized series)
- AS/NZS2566.1:1998 Buried flexible pipelines Part 1: Structural Design
- AS/NZS2566.2:2002 Buried flexible pipelines Part 2: Installation
- AS/NZS2638.2:2003 Gate valves for waterworks purposes – Resilient seated
- AS/NZS4020:2005 Testing of products for use in contact with drinking water
- AS/NZS4058:2007 Precast concrete pipes (pressure and non-pressure)
- AS/NZS4087:2011 Metallic flanges for waterworks purposes
- AS/NZS4130:2009 Polyethylene (PE) Pipes for Pressure Applications
- AS/NZS4158:2003 Thermal-bonded polymeric coatings on valves and fittings for water industry purposes
- AS/NZS4331:1995 Metallic Flanges (HOLD)
- AS/NZS4442:1998 Welded steel pipes and fittings for water, sewage, and medium pressure gas
- AS/NZS:4998:2009 Bolted unrestrained mechanical couplings for waterworks purposes
- AS2129:2000 Flanges for pipes, valve and fittings
- AS3897.3:2002 Site Testing of Protective Coatings
- AS4041:2006 Pressure Piping
- BS EN 1092-1:2007 Flanges and their joints – Circular flanges for pipes, valves, fittings and accessories, PN designated – Part 1: steel flanges
- NZS1170.5:2004 Earthquake Actions - New Zealand
- Seismic Hazard Assessment for the Hospital Prince of Wales Park Reservoir (Beca, 2012)

7 Mechanical Design Criteria

The pipework arrangement will be designed to allow the same functionality as shown in the “Proposed Pipe Work and Fittings” drawing (approved 13/10/2011) issued by Capacity Infrastructure Services. This functionality is to be confirmed by preparation of a P&ID by Beca and subsequent review by Capacity and amendment by Beca to an agreed process flow.

Pipework and valves within the pipe tunnel are to be configured to allow the inspection (internal and external) and removal and replacement of pipe spools and fittings.

The design life for the pipelines is to be at least 100 years with maintenance.

7.1 Flows and Hydraulics

The inlet and outlet pipework will separately connect the reservoir to the Central Zone via the streetworks pipelines (primary connection) and to the Kaitoke main via Bell Road reservoir (secondary connection). There will be two separate inlets into the reservoir, and one outlet that will split to the two lines. A cross connection will link the secondary inlet to the outlets via a PRV. An overflow and scour will also be provided.

7.1.1 Primary Inlet

Design Flowrate: 700 L/s (maximum peak value)

Proposed diameter: 600 NB inside tunnel, 800 NB below ground

Nominal velocity: 2.5 m/s (at maximum peak flowrate)

Level of rim: 90.000 NCD

7.1.2 Secondary Inlet

Design Flowrate: 120 L/s (maximum peak value)

Proposed diameter: 300 NB inside tunnel and below ground

Nominal velocity: 1.7 m/s (at maximum peak flowrate)

Level of rim: Bent down to below normal water level; inlet pipe high point at 92.000 NCD

7.1.3 Outlet

Design Flowrate: 800 L/s (maximum peak value)

Proposed diameter: 600 NB inside tunnel, 900 NB below ground

Nominal velocity: 2.8 m/s (at maximum peak flowrate)

Below floor level at 45°, with grill (stainless steel handrailing to be considered in subsequent design stages) and vortex preventer.

7.1.4 Secondary Outlet

Design Flowrate: Not provided

Proposed diameter: 300 NB inside tunnel and below ground

Branches from primary outlet in tunnel.

7.1.5 Overflow Pipework

Design Flowrate: 1,200 L/s (maximum peak value)

Level of rim: 92.075 NCD

Allowable head: 250mm above rim (92.325 NCD)

Pipe provided to be 600 NB, this would give a velocity of 4.2 m/s at the maximum peak design flowrate. Although this velocity is high the discharge at this rate would be for short periods only hence CLMS could be considered suitable. As an alternative epoxy lined pipe could be used subject to agreement from Capacity. The headloss in the overflow pipe is expected to be about 5m. As the overflow is expected to be more than 20m above the discharge point, there is plenty of head available. A vortex inhibitor is to be provided at the bellmouth to prevent vortices in the event that the overflow inlet becomes submerged.

7.1.6 Scour Pipework

Design Flowrate: 400 L/s (nominal over a period of 24 hours)

The minimum pipe diameter will be approximately 500mm prior to connection into the overflow line.

7.1.7 Test Pressure

The test pressure for all pipework shall be 100m, which is the minimum specified in the RFT. This will be significantly in excess of the expected operating pressure. However, it will be less than the 1.3 x minimum pipe pressure rating as specified in the WCC Water Supply Specification.

7.2 Valves

7.2.1 Below Ground Valves

Below ground valves up to and including 300mm diameter will be flanged resilient seated gate valves.

Below ground valves larger than 300mm diameter will be resilient seated double flanged butterfly valves. Although the WCC Code of Practice for Land Development requires butterfly valves to be approved by the Council, the RFT indicates that most valves will be butterfly valves. For larger diameters, butterfly valves will offer a significant space, weight and cost saving over gate valves.

7.2.2 Above Ground Valves

Isolating:

All valves isolating lines of 50mm diameter or less will be ball valves.

All above-ground manual valves isolating lines of greater than 50mm diameter will be butterfly valves. These valves will be double flanged which enables them to still isolate when pipework on one side is removed.

Although the RFT shows bypass valves on large diameter valves Capacity has advised that no bypass valves should be provided as there is no requirement to maintain water supply while operating valves (e.g. for testing purposes).

Control:

As specified in the RFT, all control valves will be plug valves and the type is to be agreed with Capacity. Manufacturers such as Valmatic and Dezurik will be considered. Rotork actuators with battery backup will be used for the shut-off valves. Limit switches will be required.

Flow control on the reservoir inlet and outlet valves is required to balance operation of the reservoir with the other low level zone reservoirs and ensure turnover of the reservoir contents.

The valves will be specified to be controlled so that the final part of the closure happens slowly enough to mitigate the risk of problematic surge pressures in the pipeline.

Pressure Reducing Valve:

A hydraulically actuated control valve is proposed. This will be sourced from Capacity approved suppliers Claval or Bermad. These valves generally require at least 10m upstream pressure to operate, but we understand this will be available from the Kaitoke main.

Non Return Valves:

A non-return valve will be installed on the 300mm diameter outlet. This could be a swing-check type of valve or a wafer type twin plate sprung non-return valve. Capacity to confirm their preference.

Flap valves will be provided where the open channel tunnel drain discharges to the stormwater manhole.

Air Valves:

Air valves will be required to vent air that accumulates at high points and bends in the pipeline, and to assist when filling the pipe. They will also be required to let air in to the pipe to prevent negative pressure from occurring in the event of sudden valve closure.

The exact location of air valves will be confirmed once the pipework arrangement is finalised.

The recommended air valve type is the Vent-O-Mat RBX. Capacity to confirm their preference.

Standpipes will not be considered as they are not preferred by Capacity.

7.3 Flexible Couplings

Bellows will be provided as the first flexible coupling outside of the reservoir after the isolation valves hard piped to the reservoir structure. A flexible coupling will also be provided at the end of the tunnel arrangement. Where the pipework exits the tunnel the pipe penetration will be configured to allow a degree of movement to accommodate seismic movement and differential settlement.

All pipework in the tunnel will be restrained, with thrust taken up outside the tunnel. However pipe supports could also provide thrust restraint if required. Flexible couplings/bellows may need to have tie bolts installed to allow partial dismantling of piping which is in service to deal with thrust.

Viking Johnson dismantling joints or tied gibaults/flange adaptors will be used to provide the ability to dismantle valves and fittings and to correct alignments during construction.

Earthquake loads on the pipes will be determined as detailed in NZS 1170.5:2004 modified in accordance with the Site Specific Seismic Hazard Assessment (Beca, 2012). The design acceleration recommended by the site specific assessment is 1.56g horizontal.

7.4 Pipe

7.4.1 Below Ground Pipework

Pipe will be installed with a minimum of 1m cover. The design will aim to maintain a maximum cover of 2m. However, the raising of the existing park level will mean the cover for existing pipework may exceed this with agreement from Capacity. A minimum of 0.5m clearance needs to be provided between the 33kV cable and the pipeline, but the design will aim for a larger clearance.

The large diameter below ground pipework (i.e. the 800mm and 900mm diameter pipes) will be concrete lined mild steel to NZS 4442. The pipe wall thickness will be as detailed in column (b) of Table 2 in NZS 4442:1988. The pipe outside diameters will be 345, 426, 508, 610, 813 or 914 mm.

Depending on the pipe supplier, the pipes will be coated with either a Polyken Synergy or Sintakote coating. Hemispherical slip-in joints are proposed.

The structural pipeline design shall allow for HN-HO-72 external loading in areas where heavy traffic loads are expected.

The material for smaller diameter below ground pipework will be determined during subsequent design stages, but could be steel or ductile iron.

Pipework 600mm diameter and above will have 500mm diameter access hatches to provide personnel access to the interior of the pipe. These will be spaced at 200m intervals. We understand Capacity have preferred details for these access points.

7.4.2 Above Ground Pipework

Concrete lined, epoxy coated steel pipework to NZS 4442:1988 is preferred for all pipework in the tunnel. Epoxy lined pipe may be used for short pipe lengths with agreement from Capacity.

All pipework will either be welded, flanged or use gibault/flange adaptor flexible couplings.

7.5 Flanges

All flanges will be tested to a minimum of 10 bar. Subject to suitable valves, bellows and other fittings, the flange drilling will be to AS2129 Table E (which complies with the WCC Water Supply Specification, this is the metric equivalent of BS10 specified in the RFT). Table D meets the test and operation pressure requirements but is thinner and lighter than Table E but will only be used with approval from Capacity.

Other flange patterns will be considered if a preferred valve or other fitting cannot be obtained in a Table E drilling. AS/NZS 4331:1995 (HOLD to be determined by Capacity at detail design stage) / BS EN 1092 PN10 or PN16 drilling would be the next choice. This drilling pattern is not the first choice as it would not be consistent with most of the fittings in Capacity's network (HOLD to be determined by Capacity at detail design stage). The design will aim to have a consistent flange drilling for the same size of flange throughout the tunnel pipework.

An insulated flanged joint is required on the inlet pipe immediately outside the structure to allow cathodic protection to be applied to the inlet pipe without loss of the impressed current into the structure.

7.6 Instrumentation

7.6.1 Flowmeters

All flowmeters will be ABB Watermaster 24 V DC as specified in the RFT.

Flowmeters will be full bore as the pipework in the tunnel is already a reduced diameter.

As bidirectional flow could occur, at least 5 x diameter straight length will be provided upstream and downstream of the flowmeters.

7.6.2 Other Instrumentation

Tappings for pressure indicators and transmitters will be provided.

Appendix E

Geotechnical Basis of Design

Report

Hospital Prince of Wales Reservoir Geotechnical Basis of Design

Prepared for Wellington City Council (Client)

By CH2M Beca Limited

1 February 2013



Revision History

Revision N°	Prepared By	Description	Date
A	Jerry Spinks	For Client Review	05/10/12
B	Simon Edmonds	Updated for CDOR	01/02/2013

Document Acceptance

Action	Name	Signed	Date
Prepared by	Jerry Spinks	<i>J Spinks</i>	01/02/2013
Reviewed by	Andrew Law	<i>A Law</i>	01/02/2013
Approved by	Brian Smith	<i>B Smith</i>	01/02/2013
on behalf of	CH2M Beca Limited		

Table of Contents

1	Introduction.....	1
2	Objectives	1
3	Definitions	1
4	Scope of Geotechnical Basis of Design.....	2
5	Reference Documents	2
6	Geotechnical Design Criteria	3
6.1	Geotechnical Profile	3
6.2	Groundwater	3
6.3	Soil Parameters	3
6.4	Lateral Seismic Earth Pressures	4
6.5	Subgrade Stiffness	4
6.6	Temporary Cut Slopes.....	4
6.7	Engineered Fill Slopes.....	4
6.8	Foundations	4
6.9	Earthworks.....	5

1 Introduction

This document describes the basis of design for the geotechnical criteria for the **Hospital Prince of Wales Reservoir Project** located in upper Prince of Wales Park at Mount Cook, Wellington City.

The project includes a reservoir and the associated earthworks and pipelines. The reservoir is fully buried below ground level and will store 35,000m³ volume of fresh water. It will supply water to Wellington Hospital and to Central Business District (CBD). CH2M Beca (Beca) has been commissioned by Wellington City Council (WCC) to supply engineering services for the development of the new reservoir.

2 Objectives

The principal objective of this document is to establish a basis that shall be used for the design and documentation works of all aspects of the reservoir earthworks and foundations and earth pressure loading and to:

- Comply with client requirements
- Comply with statutory requirements
- Utilise the relevant experience and skills of the design team members
- Enable close coordination with other design disciplines
- Permit construction sequencing to be undertaken in accordance with the client's agreed programme

3 Definitions

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

CBD	Central Business District
Capacity	Capacity Infrastructure
Council	Wellington City Council
GW	Greater Wellington regional Council
HPOW	Hospital Prince of Wales
IL	Importance Level
RFT	Request for Tender
SLS	Serviceability Limit State
TBC	To Be Confirmed
TWL	Top Water Level
WCC	Wellington City Council

WE

Wellington Electricity

4 Scope of Geotechnical Basis of Design

The following items are addressed in the geotechnical basis of design:

- Geotechnical profile
- Groundwater
- Soil parameters
- Lateral seismic earth pressures
- Temporary cut slopes
- Engineered fill slopes
- Foundation bearing pressures
- Earthworks material classification

5 Reference Documents

The following documents are to be referred to in the design of the Hospital Prince of Wales Reservoir:

- Capacity Infrastructure Services Request for Tender for the Consultancy Services for the Hospital Prince of Wales Reservoir 1 February 2012
- NZS1170.5:2004 Earthquake Actions - New Zealand
- 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)
- O'Riley et al, Seismic Performance of the Terrace Tunnel Approach Walls, Wellington. 2006, NZSEE Conference
- MJ Pender, Some Properties of Weathered Greywacke, 1971 Aust NZ Conference in Geomechanics
- MJ Pender, Friction and Cohesion Parameters for Highly and Completely Weathered Wellington Greywacke, 1980, University of Auckland
- Read, S.A.L., Richards, L., and Perrin N.D.(2000): Assessment of New Zealand Greywacke Rock Masses with the Hoek-Brown Failure Criterion. In: C. Haberfield et al., (ed.) Proceedings GeoEng2000, International Conference on Geotechnical & Geological Engineering, Melbourne, 19-24 November 2000, paper SNES0868: Technomic Publishing Company, Lancaster.
- CH2M Beca Ltd, 2012: Seismic Hazard Assessment for the Hospital Prince of Wales Reservoir. Prepared for Capacity Infrastructure Services Limited.
- Duncan C. Wyllie, Foundations on Rock, First Edition (1992)
- BS8004:1986 Code of Practice for Foundations, 1986, British Standards Institute

6 Geotechnical Design Criteria

The new reservoir is expected to provide water for the Wellington Regional Hospital in Newton. This structure has a base isolation system and a design requirement that the building is fully operational within 6 hours after a major earthquake. The return period for this major earthquake has been selected as 1000 years. The proposed HPOW reservoir is to match this design requirement by having a design SLS 2 (Operational Continuity) earthquake return period of 1000 years (HOLD – Seismic design standard to be confirmed by Capacity). The design is to meet or exceed the requirements of Importance level 4 (IL4) structures for facilities with a special post disaster function. The backfill around the reservoir does not have to meet the same SLS2 criteria provided slope failure does not affect the integrity of the reservoir.

6.1 Geotechnical Profile

Unit	Description	Depth to top (m)	Thickness (m)
Topsoil	Soft to firm clayey organic SILT; dark brown; medium to high plasticity	0	0.2
Residual soil/ completely weathered GREYWACKE/ ARGILLITE*	Stiff, clayey SILT; orange-brown; low to high plasticity.	0.2	0.2-1.6
	Extremely weak to very weak GREYWACKE/ ARGILLITE.	0.4-1.8	0.3-1.0
Highly weathered GREYWACKE/ ARGILLITE	Weak to very weak GREYWACKE/ ARGILLITE.	0.7-2.5	0.5-5.4
Moderately weathered GREYWACKE/ ARGILLITE	Weak to moderately strong GREYWACKE/ ARGILLITE.	1.2-8.0	1m+

*GREYWACKE/ARGILLITE is subsequently referred to as Greywacke.

6.2 Groundwater

Groundwater monitoring was undertaken within boreholes BH01 and BH02, recording average groundwater RLs of 86.5m and RL 85.8m respectively.

6.3 Soil Parameters

Unit	Bulk Density γ (kN/m ³)	Effective cohesion c' (kPa)	Effective Friction Φ (°)
Engineered Fill (reworked Greywacke)	19	0	36
Residual Soil/ Completely Weathered Greywacke	18	10	28
Highly Weathered Greywacke	22	100	45
Moderately Weathered Greywacke	24	100	45

6.4 Lateral Seismic Earth Pressures

Lateral earth pressures will be prepared during detailed design. We recommend that RRU 83 – ‘Seismic Design of Bridge Abutments’ is used for determination of seismic earth pressures on the reservoir wall. We anticipate that ‘rigid’ wall loading will be applicable; however this should be confirmed by the structural designers. This method is to be compared with the NZS 3106:2009 method for embedded tanks. Seismic accelerations will be as per the recommendations from the Hospital Prince of Wales Reservoir – Site Specific Seismic Hazard Assessment (Beca, Rev B ‘Final’ dated 21 December 2012).

6.5 Subgrade Stiffness

During detailed design, the interaction of the reservoir foundations and underlying subgrade will be modelled by providing ‘spring stiffness’ values for the weathered Greywacke. Representative values will be determined by calculating displacements across a range of foundations.

6.6 Temporary Cut Slopes

Temporary cuts in rock are recommended to be no steeper than 60°, with a 2m to 3m wide intermediate catch benches every 8-10m height, achieving an equivalent overall slope of 1:1V(45°), max slope height in the order of 12m. Should persistent unfavourable defects be identified during the logging of the cut as it progresses, this angle may need to be reduced.

Temporary cuts in the overlying residual soils and completely weathered rock should be cut no steeper than 2H:1V.

For temporary cuts in rock an acceptable alternative to 60° cut slopes is to use either a vertical tied back soldier pile wall or near vertical reinforces shot crete and soil nail wall. This approach is recommended for the pipe tunnel excavation below the reservoir. Backfill between the excavation and pipe tunnel structure should be low strength concrete.

For vertical earth pressure from backfill on top of the pipe tunnel assessment should be in accordance with AS/NZS 3725:2007 Design for Installation of Buried Concrete Pipes, assuming the arrangement acts as ‘embankment fill’, adopting the scenario of positive projection and a backfill unit weight of 19 kN/m³ for saturated fill.

6.7 Engineered Fill Slopes

For long term stability a fill slope no steeper than 2H:1V, formed of Engineered Fill comprising re-worked Greywacke, is expected to achieve a factor of safety under static conditions of 1.4. With a maximum slope height in the order of 8m.

6.8 Foundations

We recommend a geotechnical ultimate bearing capacity of 1MPa within the moderately weathered Greywacke rock. For limit state structural design (with code factored structural loads), a strength reduction factor of 0.5 shall be applied to this value.

The settlement of shallow footings supported on the weathered Greywacke under sustained loads (e.g. unfactored dead plus live loads) are expected to be in the order of 10mm to 25mm.

6.9 Earthworks

In accordance with TNZ/F1: 1997, Specification for Earthworks Construction, we recommend the majority of the cut material is classified as Type A. This is interpreted as material that can be won by a 16T machine excavator, fitted with a toothed bucket.

The excavation will likely intercept the groundwater table (from below around RL 86m). Therefore, drainage of the reservoir platform will be required during construction, and also in the longer term, including the backfill. Drainage provisions for groundwater around the outside of the reservoir will need to be separate from the reservoir under floor drainage system as per the client requirements.

The excavated material will predominantly comprise highly to moderately weathered, highly fractured greywacke rock. The re-worked material is anticipated to be favourable in terms of its drainage characteristics and resistance to breaking down during re-working and compaction. Compaction criteria, including insitu and laboratory verification testing, will be set during the subsequent phase of design.

Report

Hospital Prince of Wales Reservoir Conceptual Design Options

Prepared for Wellington City Council (Client)

By CH2M Beca Limited

01 February 2013 Final

Volume 2 of 2 – Drawings

Drawing List of Reservoir Options

6517439-CE-K20-RB – Plan of Reservoir Options R1.0, R1.1, R1.2 R2.0, R3.0 & R3.1

6517439-CE-K02-RD – Plan of Reservoir Option R1.0

6517439-CE-K03-RC – Sections of Reservoir Option R1.0

6517439-CE-K04-RC – Plan of Reservoir Option R1.1

6517439-CE-K05-RC – Sections of Reservoir Option R1.1

6517439-CE-K06-RD – Plan of Reservoir Option R2.0

6517439-CE-K07-RC – Sections of Reservoir Option R2.0

6517439-CE-K08-RD – Plan of Reservoir Option R3.0

6517439-CE-K09-RC – Sections of Reservoir Option R3.0

6517439-CE-K15-RB – Plan of Reservoir Option R3.1 Earthworks

6517439-CE-K16-RC – Sections of Reservoir Option R3.1

6517439-CE-K17-RB – Pipe Tunnel Cross Sections – Option R1.0

6517439-CE-K18-RB – Sections of Reservoir – Option R1.0

6517439-CE-K25-RB – Plan of Reservoir Option R1.0 Earthworks

6517439-CE-K30-RA – Existing Site Features

Drawing List of Pipe Tunnel and Piping

6517439-NM-K01-RA – Plan Tunnel Location Options

6517439-NM-K02-RA – Sections Tunnel Layout Options

6517439-NM-K03-RA – Tunnel Pipework Layout Option 1 Plan

6517439-NM-K04-RA – Tunnel Pipework Layout Option 1 Sections

6517439-NM-K05-RA – Tunnel Pipework Layout Option 2 Plans

6517439-NM-K06-RA – Tunnel Pipework Layout Option 2 Sections

6517439-NM-K07-RA – Tunnel Pipework Layout Option 3 Plan

6517439-NM-K08-RA – Tunnel Pipework Layout Option 3 Sections

6517439-NM-201-RB – Mechanical Site Layout Plan SHT1

6517439-NM-202-RB – Mechanical Site Layout Plan SHT2

6517439-NM-203-RB – Pipe Tunnel Layout Plan

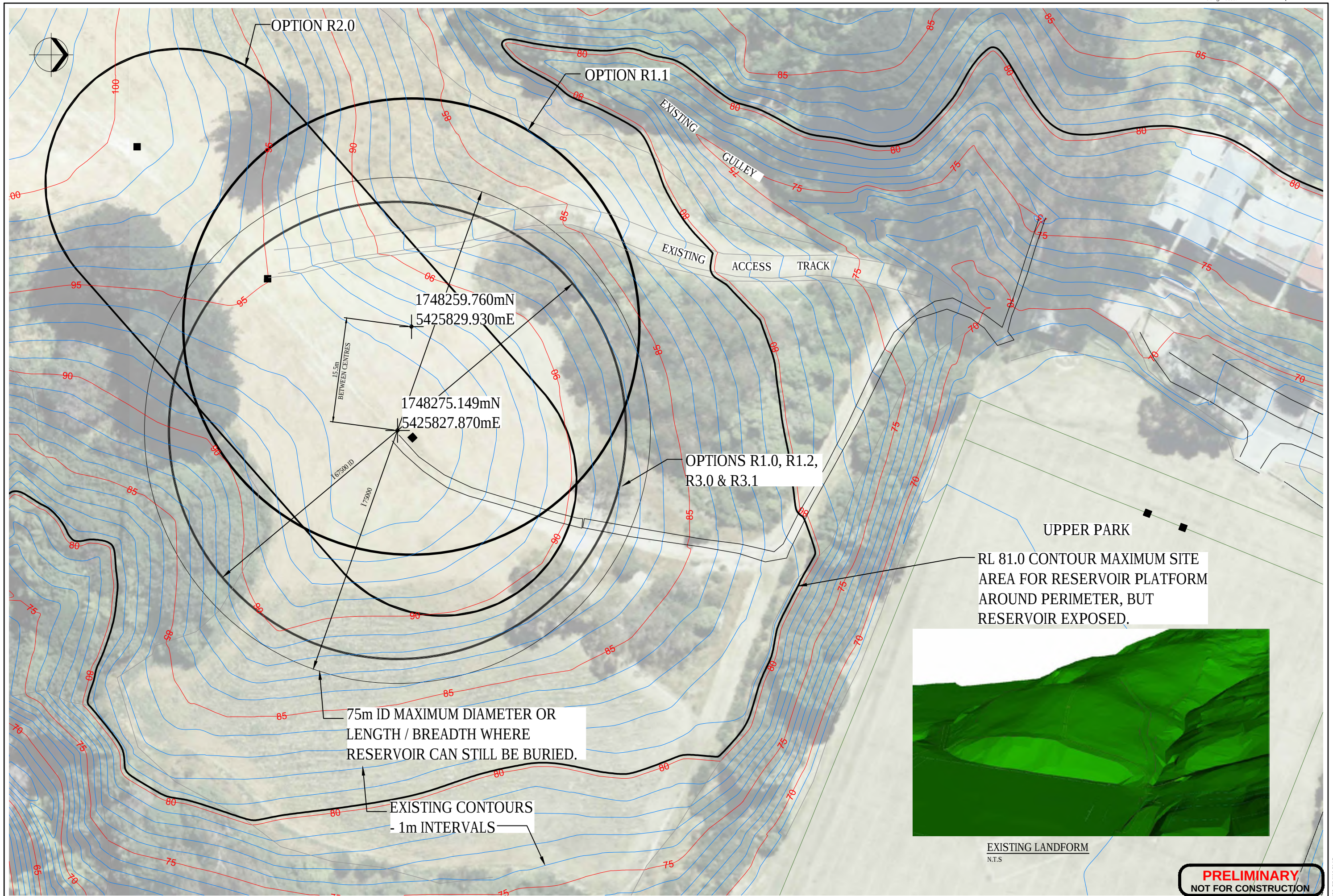
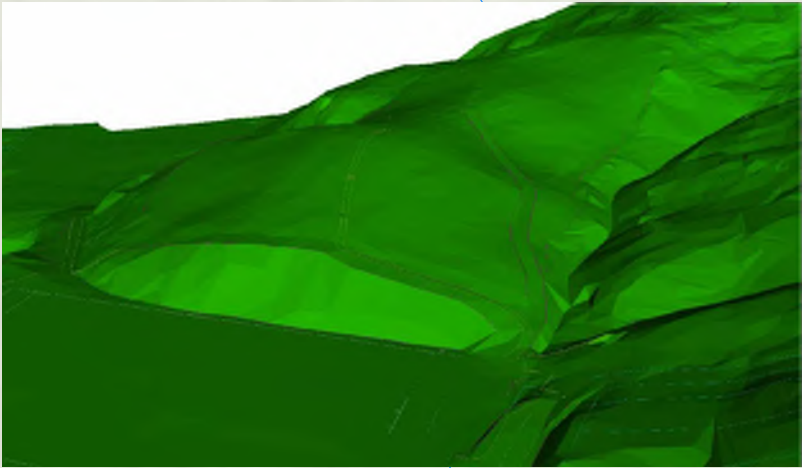
6517439-NM-204-RB – Pipe Tunnel Sections SHT1

6517439-NM-205-RB – Pipe Tunnel Sections SHT2

P and ID

6517439-NR-001-RC – Piping and Instrumentation Legend

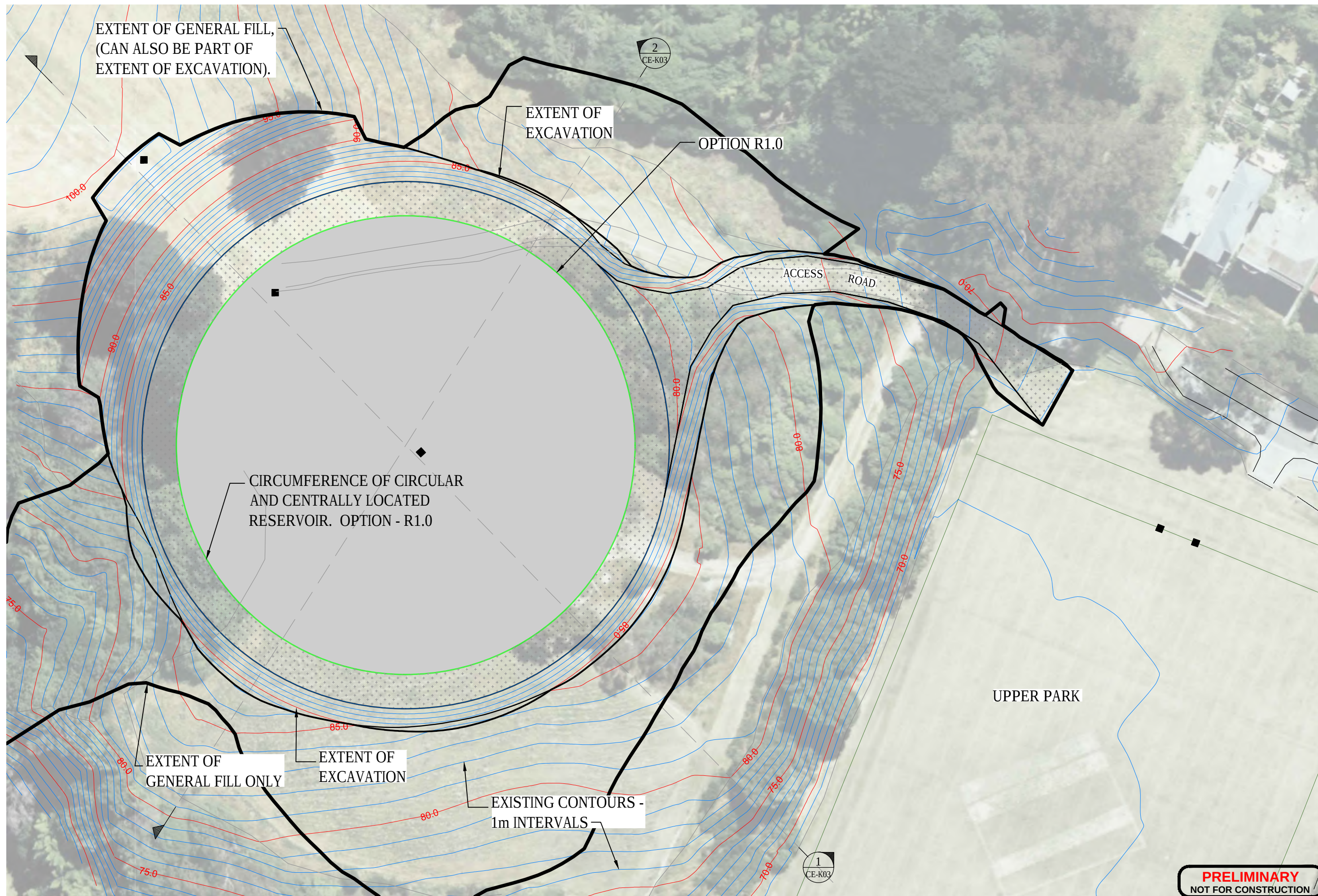
6517439-NR-002-RD – Piping and Instrumentation Diagram



EXISTING LANDFORM
N.T.S

PRELIMINARY
NOT FOR CONSTRUCTION

						Drawing Originator: 		Original Scale (A1) 1:250(A1) Drawn AJP 21.09.12 Reduced Scale (A3) 1:500(A3) Dsg Verifier BS 21.09.12 Dsg Check Date * Refer to Revision 1 for Original Signature		Client:  ALL WORK IN ACCORDANCE WITH THE WELLINGTON NEW ZEALAND OFFSHORE ENGINEERING		Project: HOSPITAL PRINCE OF WALES RESERVOIR		Title: PLAN OF RESERVOIR OPTIONS - R1.0, R1.1, R1.2, R2.0, R3.0 & R3.1		Discipline CIVIL	
B UPDATE FINAL ISSUE CDOR						AJP	CJM	SAE	01.02.13								
A FOR CONCEPTUAL DESIGN OPTIONS REPORT						AJP	CJM	SAE	21.09.12								
No. Revision						By	Chk	Appd	Date								



D	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
C	NOTES ADDED - FINAL P & S REPORT	AJP	CJM	SAE	14.09.12
B	DISTURBED GROUND EXTENTS ADDED	AJP	CJM	SAE	21.08.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:



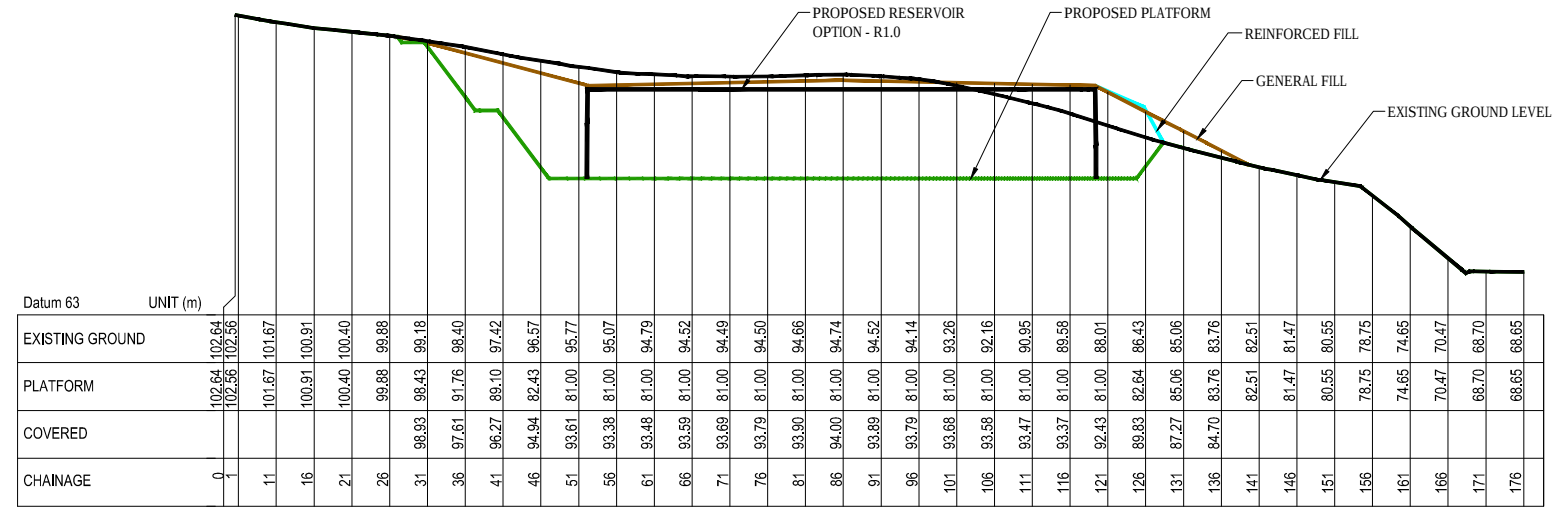
Original Scale (A1)	Design	CJM	31.07.12	Approved For Construction*
1:250(A1)	Drawn	AJP	31.07.12	
	Dsg Verifier	BS	31.07.12	
Reduced Scale (A3)	Dsg Check			
1:500(A3)	* Refer to Revision 1 for Original Signature			



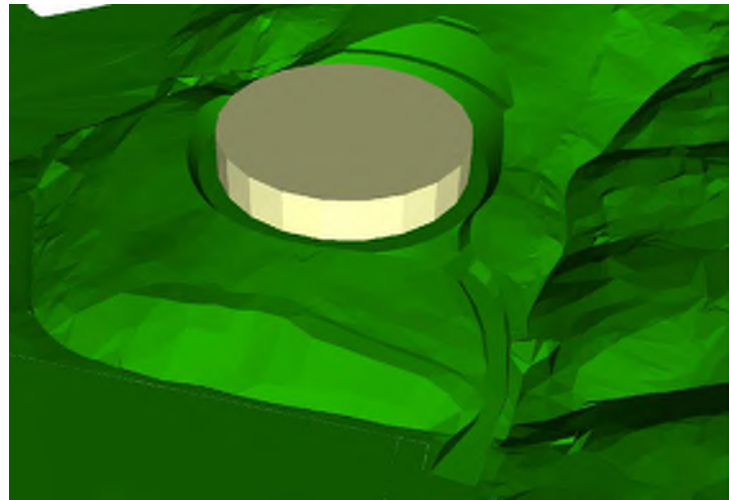
Client: HOSPITAL PRINCE OF WALES RESERVOIR

Title: PLAN OF RESERVOIR OPTION - R1.0

Discipline	CIVIL
Drawing No.	6517439-CE-K02
Rev.	D



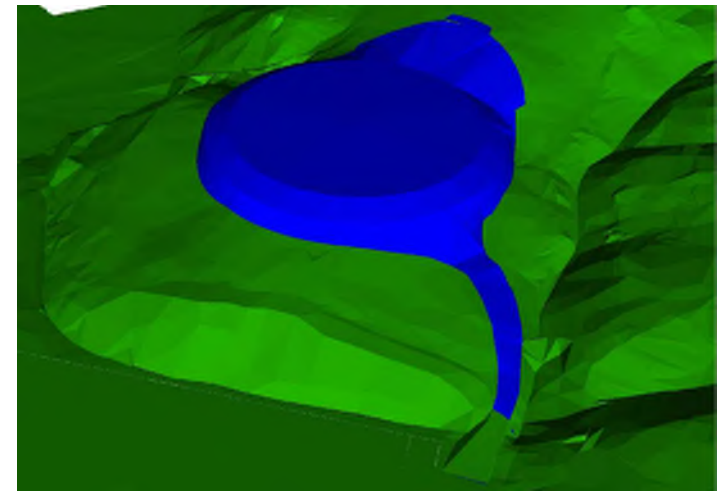
1 LONG SECTION
CE-K02 SCALE 1:500



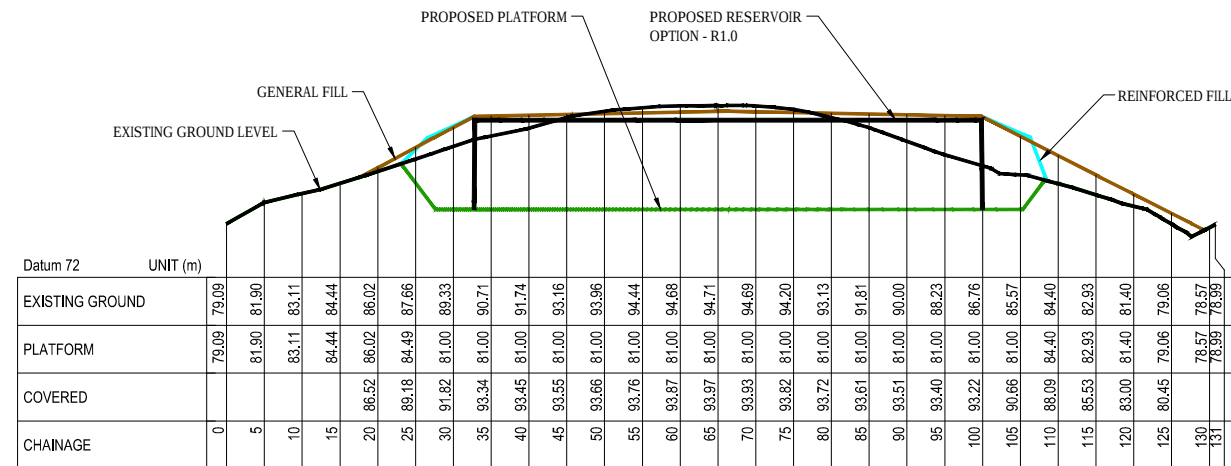
OPTION R1.0 - PLATFORM



OPTION R1.0 - COVERED
(FILL SLOPE 1V : 2H)



OPTION R1.0 - REINFORCED COVER
(FILL SLOPE 1V : 0.36H)



2 CROSS SECTION
CE-K02 SCALE 1:500

C	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
B	FINAL P & S REPORT REPORT	AJP	CJM	SAE	14.09.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

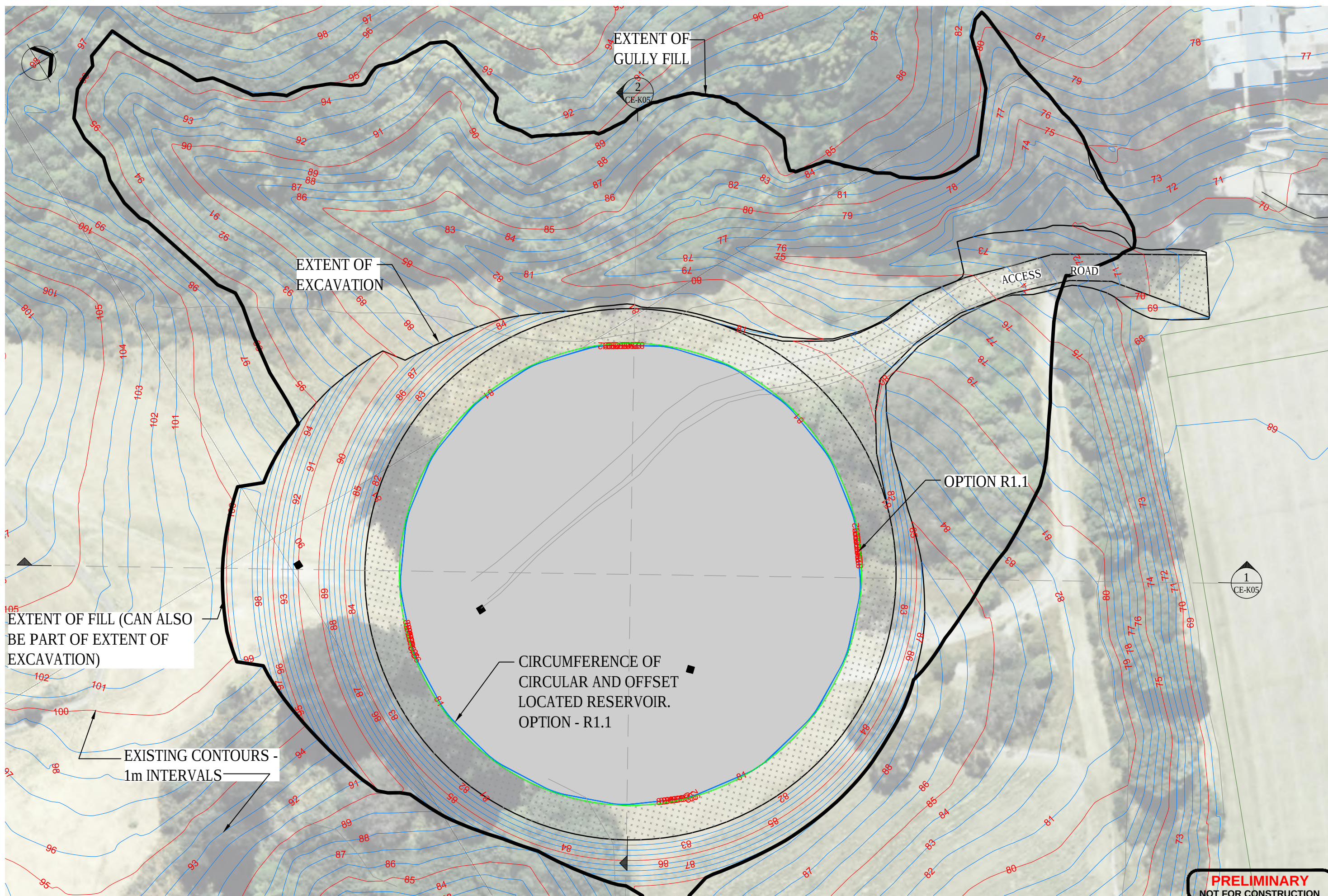
Drawing Originator:	Original Scale (A1) 1:500	Drawn	CJM	31.07.12	Approved For Construction*
	Reduced Scale (A3) 1:1000	Dsg Verifier	AJP	31.07.12	Date
		Dsg Check	BS	31.07.12	
		* Refer to Revision 1 for Original Signature			

Client:	Project:
	HOSPITAL PRINCE OF WALES RESERVOIR

Discipline:	Rev.
CIVIL	C

Title:	Discipline:	Rev.
SECTIONS OF RESERVOIR OPTION - R1.0	CIVIL	C
Drawing No.	6517439-CE-K03	

PRELIMINARY
NOT FOR CONSTRUCTION



PRELIMINARY
NOT FOR CONSTRUCTION

C	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
B	DISTURBED GROUND EXTENTS ADDED	AJP	CJM	SAE	21.08.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:

CH2M BDO



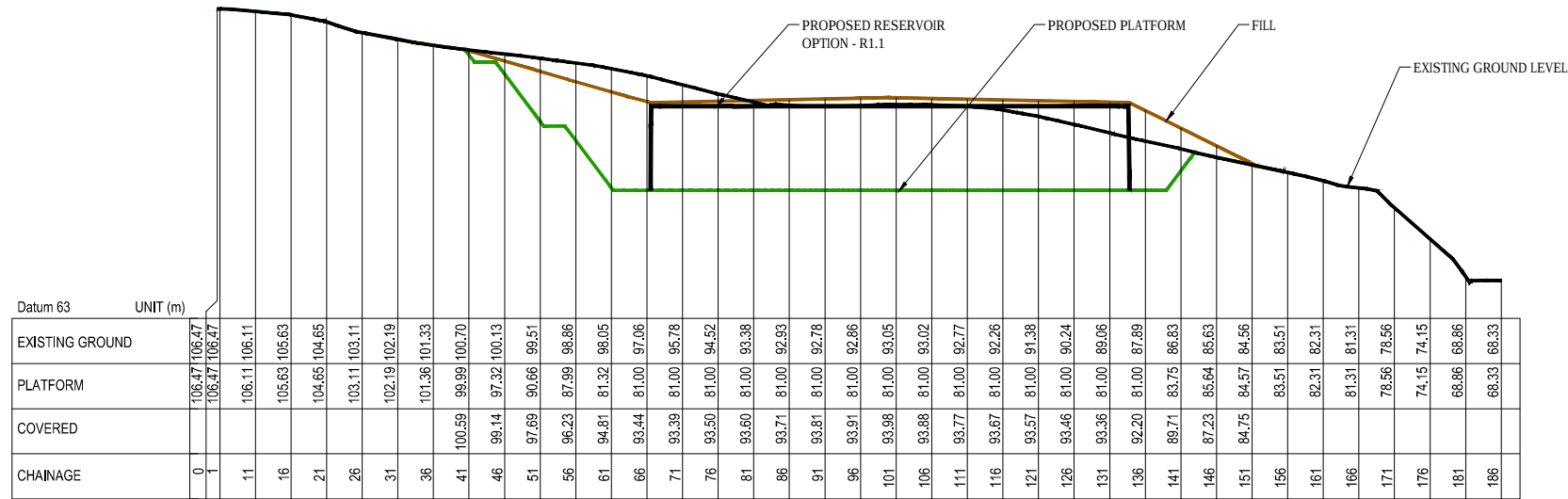
Original Scale (A1)	1:250(A1)	Drawn	AJP	31.07.12	Approved For Construction*
Reduced Scale (A3)	1:500(A3)	Dsg. Verifier	BS	31.07.12	Date
* Refer to Revision 1 for Original Signature					



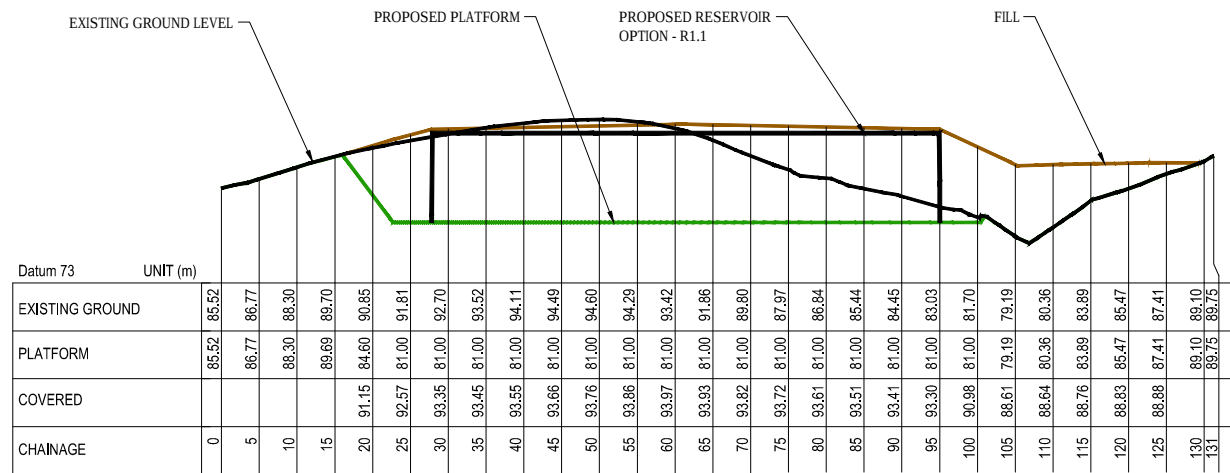
Client: HOSPITAL PRINCE OF WALES RESERVOIR

Title: PLAN OF RESERVOIR OPTION - R1.1

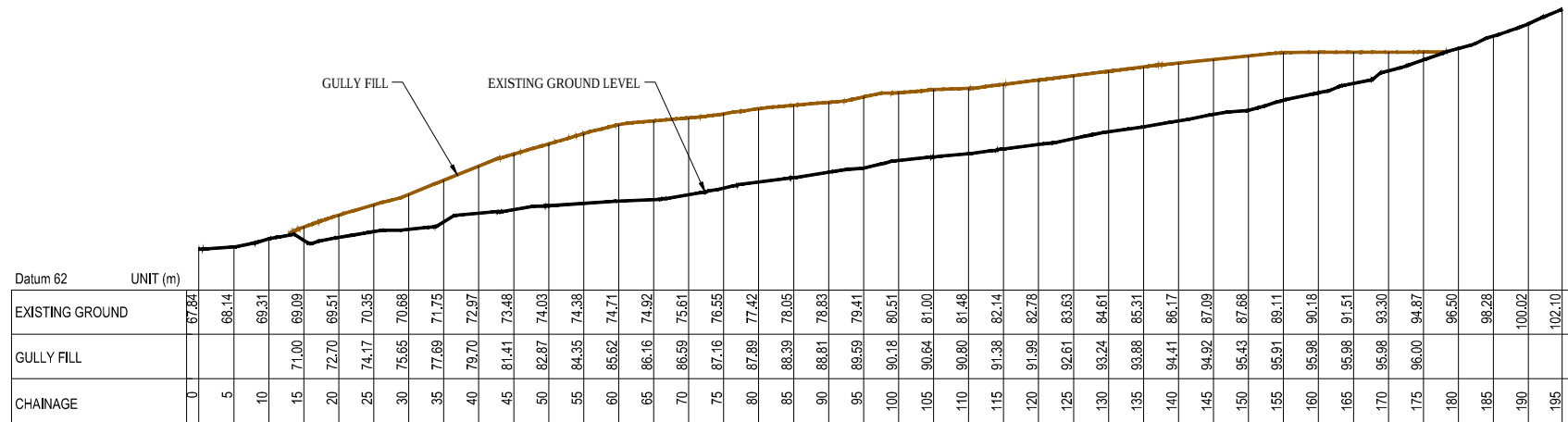
Discipline	CIVIL
Drawing No.	6517439-CE-K04
Rev.	C



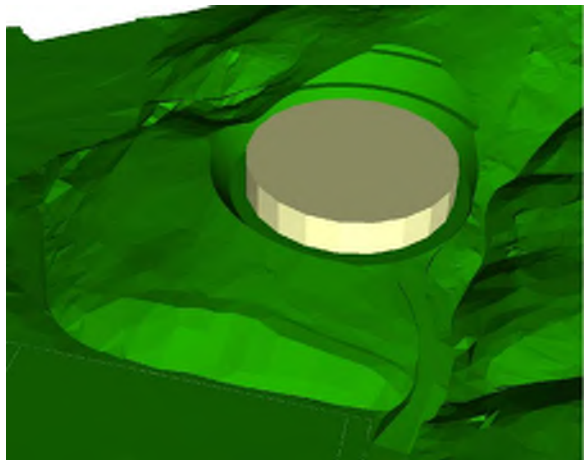
1 LONG SECTION
CE-K04 SCALE 1:1000



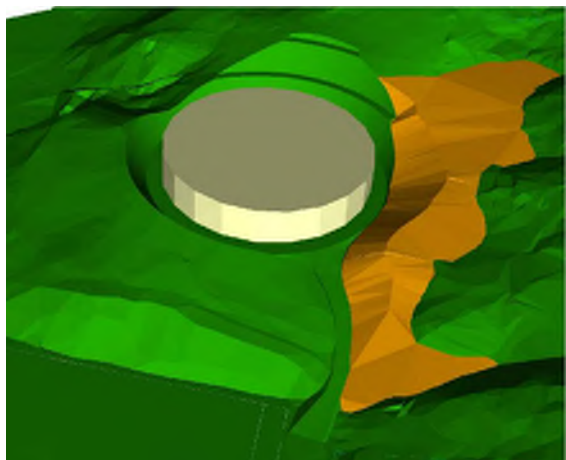
2 CROSS SECTION
CE-K04 SCALE 1:500



- GULLY LONG SECTION
CE-K04 SCALE 1:500



OPTION R1.1 - PLATFORM



OPTION R1.1 - PLATFORM WORKING
(GULLY FILL)



OPTION R1.1 - COVERED

C	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
B	FINAL P & S REPORT	AJP	CJM	SAE	14.09.12
A	ISSUED FOR INFORMATION	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

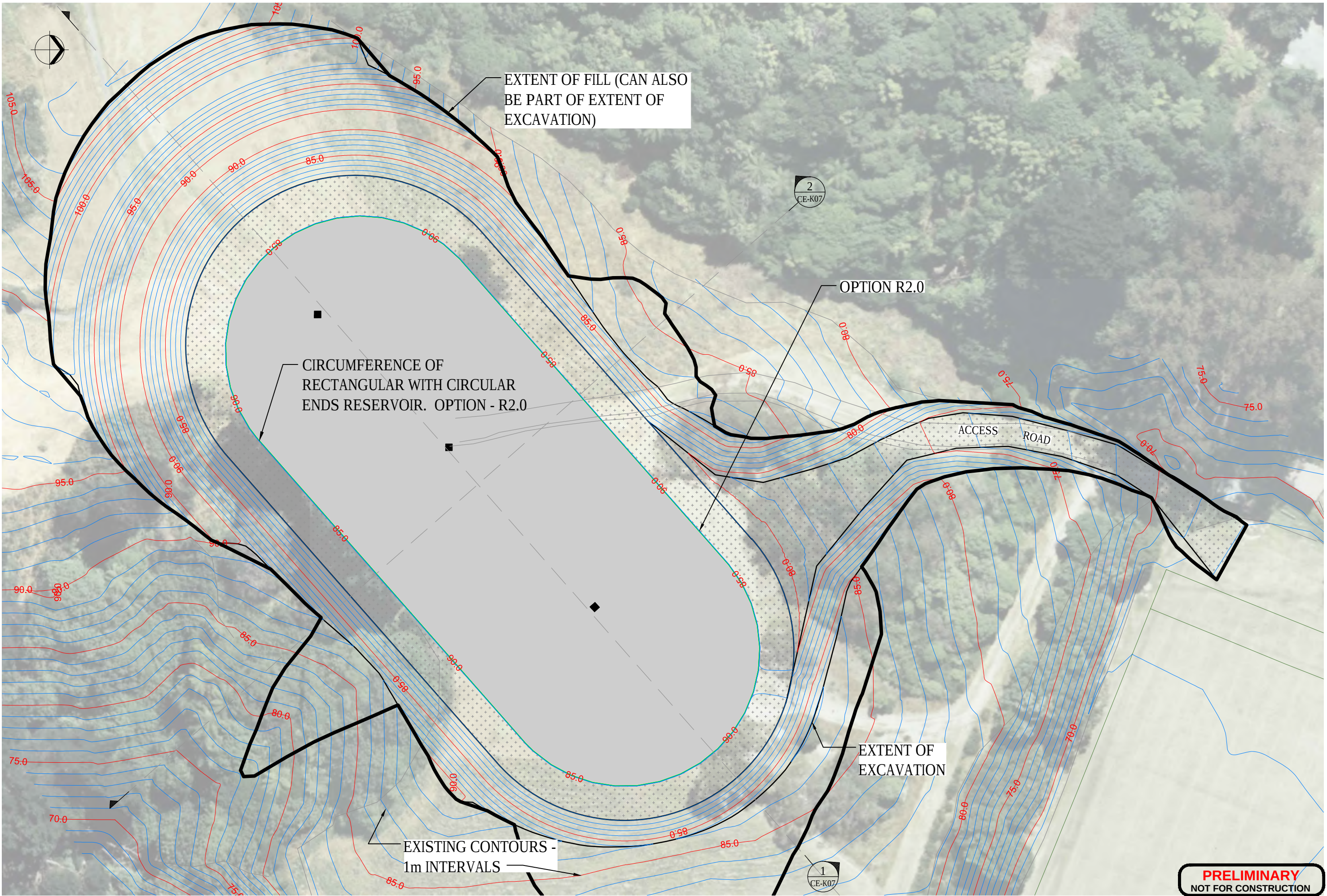
Drawing Originator:		

Original Scale (A1)	1:500	Drawn	CJM	31.07.12	Approved For Construction*
Reduced Scale (A3)	1:1000	Dsg Verifier	AJP	31.07.12	Date
		Dsg Check	BS		

Client:		Project:	HOSPITAL PRINCE OF WALES RESERVOIR

Title:	SECTIONS OF RESERVOIR OPTION - R1.1	Discipline:	CIVIL
Drawing No.	6517439-CE-K05	Rev.	C

PRELIMINARY NOT FOR CONSTRUCTION	



PRELIMINARY
NOT FOR CONSTRUCTION

D	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
C	FINAL P & S REPORT	AJP	CJM	SAE	14.09.12
B	DISTURBED GROUND EXTENTS ADDED	AJP	CJM	SAE	21.08.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:



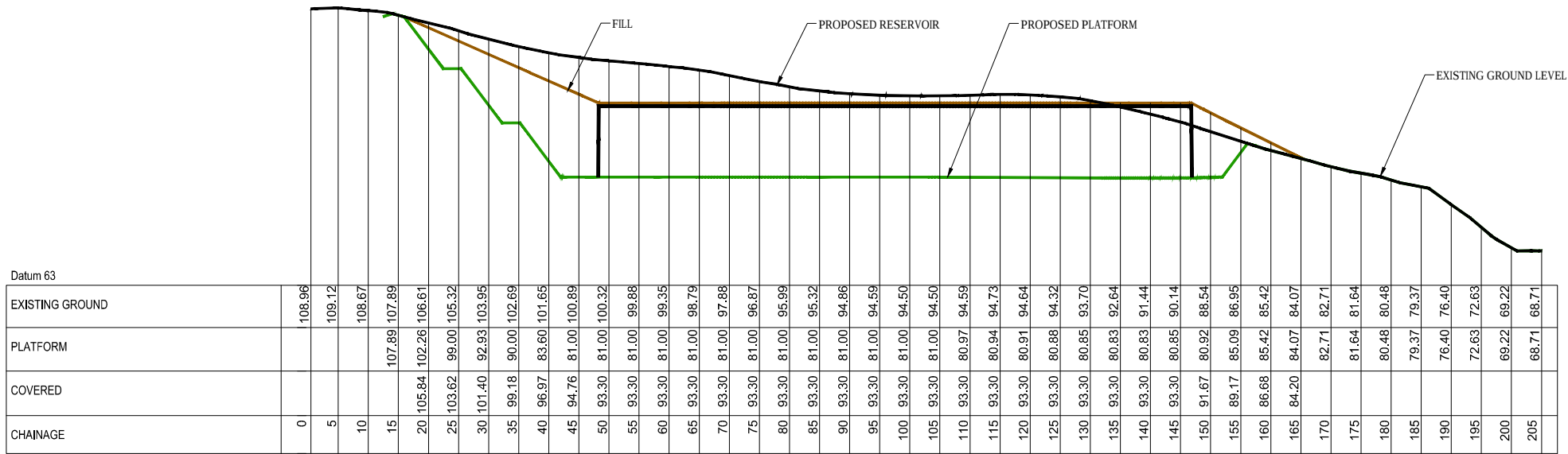
Original Scale (A1)	1:250 (A1)	Drawn	AJP	31.07.12	Approved For Construction*
Reduced Scale (A3)	1:500 (A3)	Dsg Verifier	BS	31.07.12	Date
* Refer to Revision 1 for Original Signature					



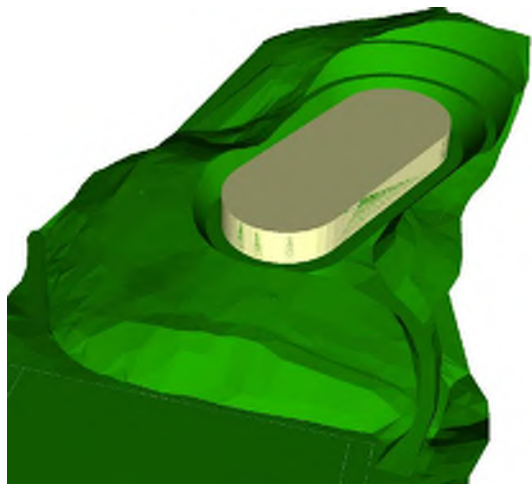
Client: PROJECT: HOSPITAL PRINCE OF WALES RESERVOIR

Title: PLAN OF RESERVOIR OPTION - R2.0

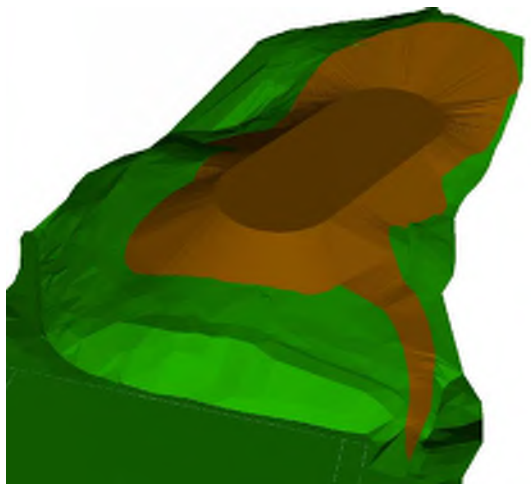
Discipline	CIVIL
Drawing No.	6517439-CE-K06
Rev.	D



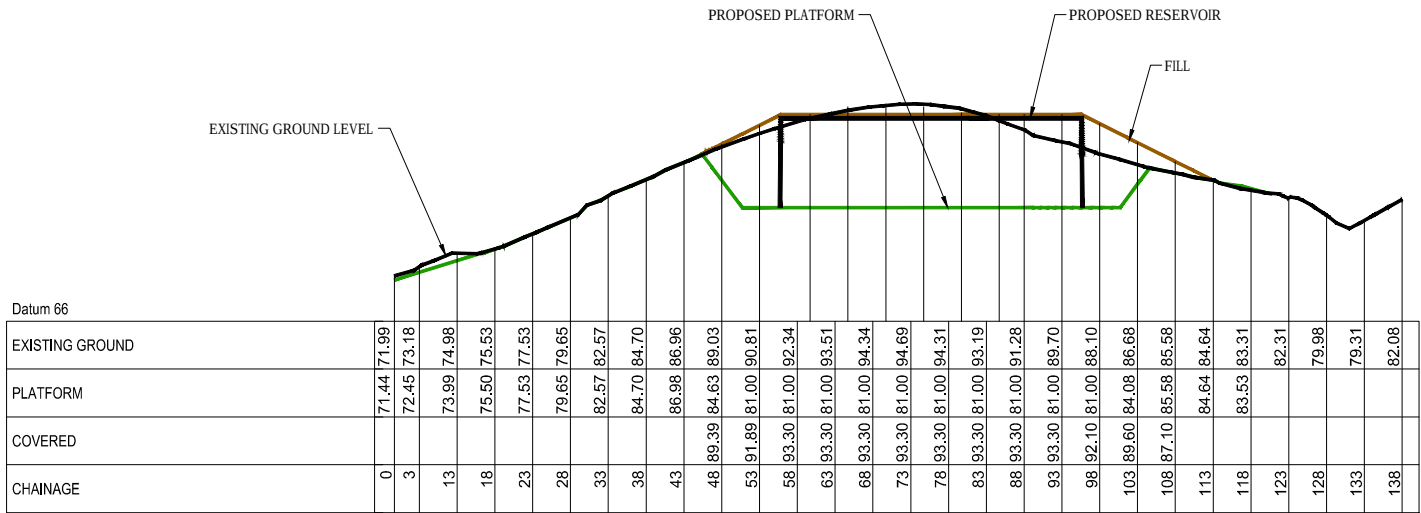
1 LONG SECTION
CE-K06 SCALE 1:500



OPTION R2.0 - PLATFORM



OPTION R2.0 - COVERED



2 CROSS SECTION
CE-K06 SCALE 1:500

C	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
B	FINAL P & S REPORT	AJP	CJM	SAE	14.09.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	07.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:



Original Scale (A1) 1:500
Reduced Scale (A3) 1:1000

Designed CJM 31.07.12
Drawn AJP 31.07.12
Dsg Verifier BS 31.07.12
Dwg Check
* Refer to Revision 1 for Original Signature

Approved For Construction*
Date

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

SECTIONS OF RESERVOIR
OPTION - R2.0

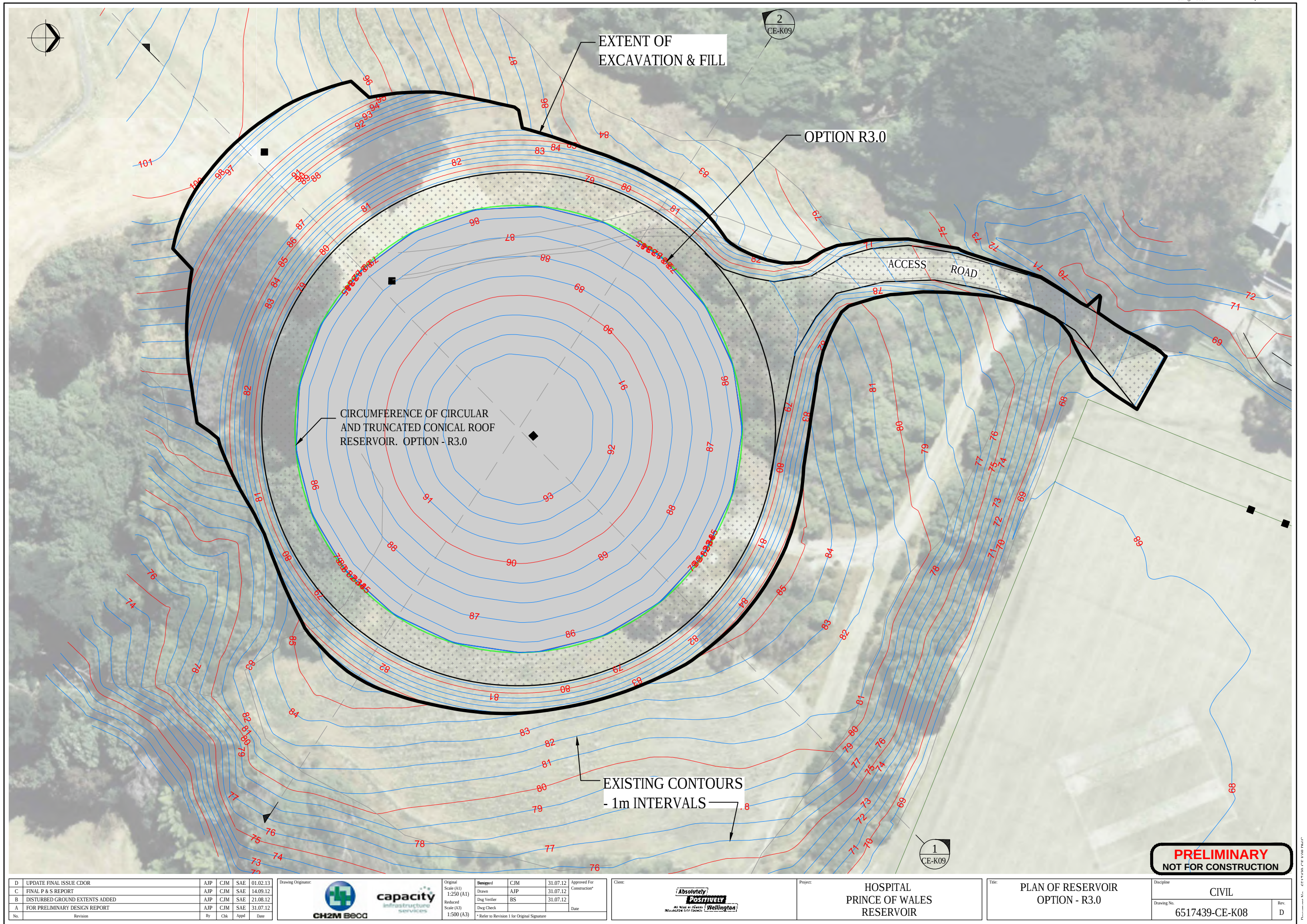
Discipline:

CIVIL

Drawing No. 6517439-CE-K07

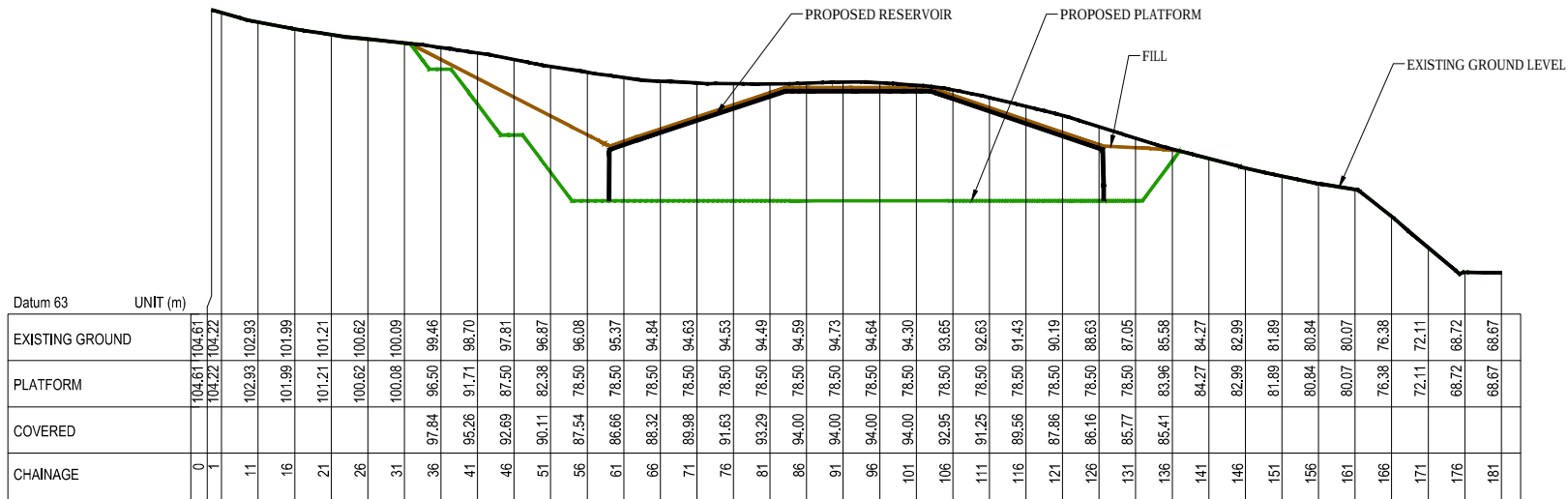
Rev. C

PRELIMINARY
NOT FOR CONSTRUCTION



PRELIMINARY
NOT FOR CONSTRUCTION

D UPDATE FINAL ISSUE CDOR				AJP	CJM	SAE	01.02.13	Drawing Originator:  	Original Scale (A1) 1:250 (A1) Reduced Scale (A3) 1:500 (A3) * Refer to Revision 1 for Original Signature	Designed: Drawn: Dsg. Verifier: Dsg. Check: * Refer to Revision 1 for Original Signature	CJM AJP BS Date	31.07.12 31.07.12 31.07.12 Date	Approved For Construction*	Client:  ALL WORK IN ACCORDANCE WITH THE WELLINGTON WELLINGTON OFFSHORE	Project: HOSPITAL PRINCE OF WALES RESERVOIR	Title: PLAN OF RESERVOIR OPTION - R3.0	Discipline: CIVIL	Drawing No. 6517439-CE-K08	Rev. D
C FINAL P & S REPORT				AJP	CJM	SAE	14.09.12												
B DISTURBED GROUND EXTENTS ADDED				AJP	CJM	SAE	21.08.12												
A FOR PRELIMINARY DESIGN REPORT				AJP	CJM	SAE	31.07.12												
No.	Revision			By	Chk	Appd	Date												



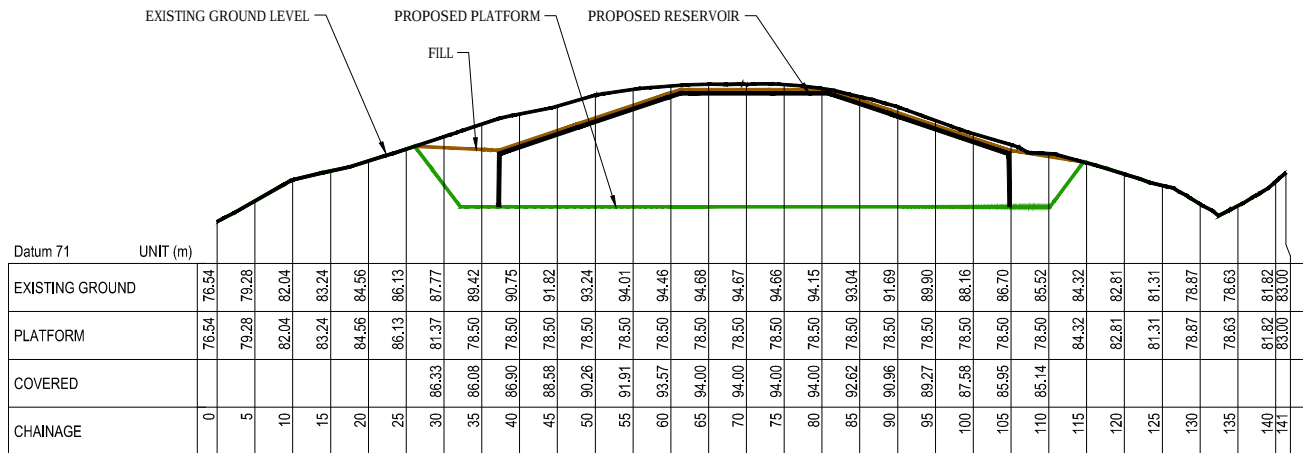
1 LONG SECTION
CE-K08 SCALE 1:500



OPTION R3.0 - PLATFORM



OPTION R3.0 - COVERED



2 CROSS SECTION
CE-K08 SCALE 1:500

C	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
B	FINAL P & S REPORT	AJP	CJM	SAE	14.09.12
A	FOR PRELIMINARY DESIGN REPORT	AJP	CJM	SAE	31.07.12
No.	Revision	By	Chk	Appd	Date

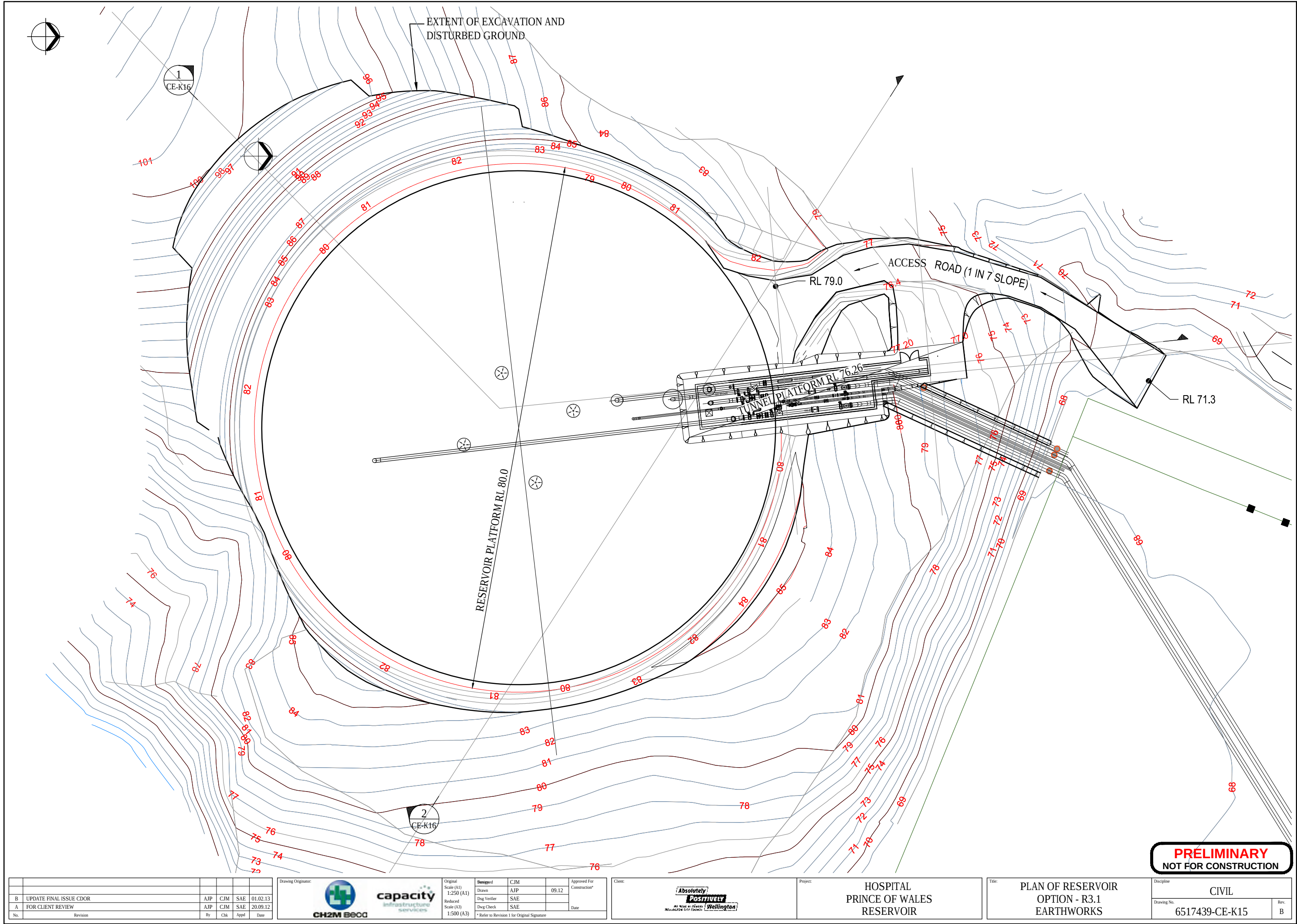
Drawing Originator:	Original Scale (A1) 1:500	Drawn	CJM	31.07.12	Approved For Construction*
	Reduced Scale (A3) 1:1000	Dsg Verifier	BS	31.07.12	Date
		Dsg Check			
		* Refer to Revision 1 for Original Signature			

Client:	Project:
	HOSPITAL PRINCE OF WALES RESERVOIR

Discipline:	Rev.
CIVIL	C

Title:	Drawing No.
SECTIONS OF RESERVOIR OPTION - R3.0	6517439-CE-K09

PRELIMINARY
NOT FOR CONSTRUCTION



PRELIMINARY
NOT FOR CONSTRUCTION

B		UPDATE FINAL ISSUE CDOR		AJP	CJM	SAE	01.02.13
A		FOR CLIENT REVIEW		AJP	CJM	SAE	20.09.12
No.	Revision	By	Chk	Appd	Date		

Drawing Originator:



Original Scale (A1)
1:250 (A1)
Reduced Scale (A3)
1:500 (A3)

Designed
Drawn
Dsg Verifier
Dwg Check

CJM
AJP
SAE
SAE

09.12

Approved For Construction*

Date

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

PLAN OF RESERVOIR
OPTION - R3.1
EARTHWORKS

Discipline:

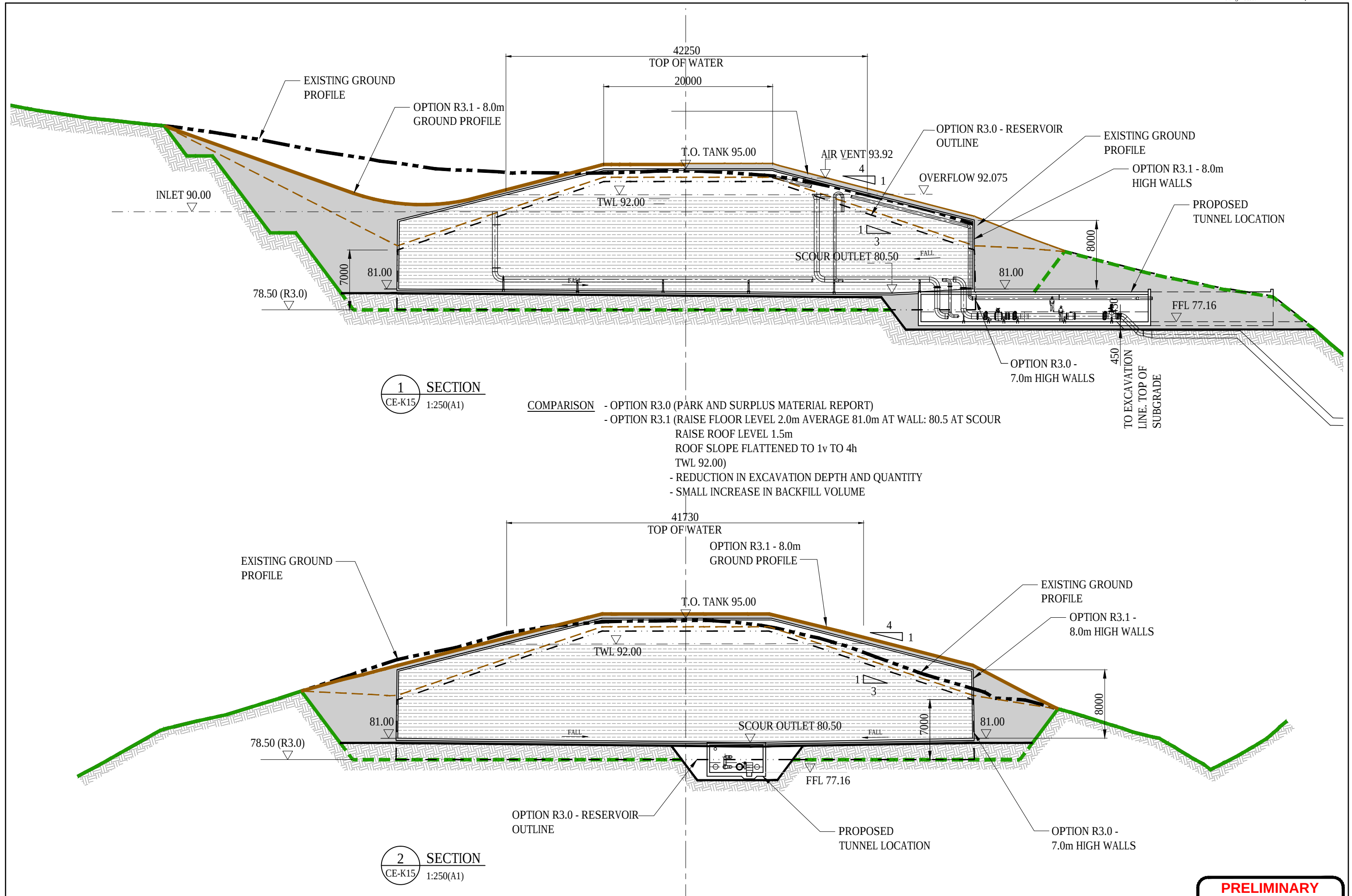
CIVIL

Drawing No.

6517439-CE-K15

Rev.

B



PRELIMINARY
NOT FOR CONSTRUCTION

C	UPDATE FINAL ISSUE CDOR	AJP		SAE 01.02.13
B	FOR CLIENT REVIEW	AJP		SAE 20.09.12
A	SKETCH FOR INFORMATION ONLY	AJP		15.08.12
No.	Revision	By	Chk	Appd Date

Drawing Originator:



Original
Scale (A1)
1:250 (A1)

Designed
SAE
09.12
Drawn
AJP
09.12
Dsg. Verifier
Dwg. Check
Date

Approved For
Construction*

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

SECTIONS OF RESERVOIR
OPTION - R3.1

Discipline:

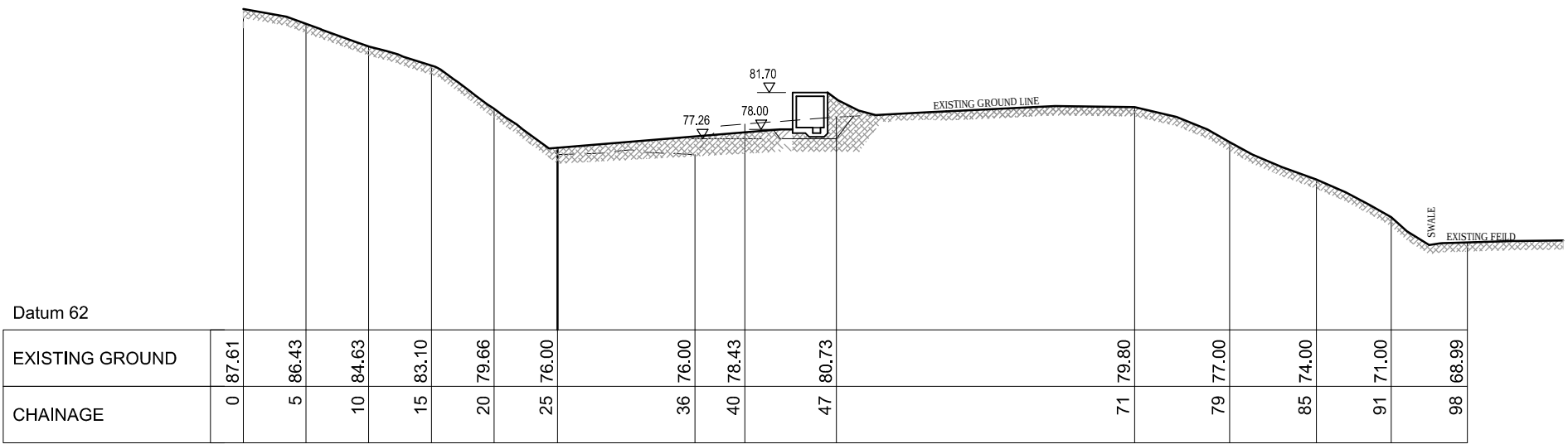
CIVIL

Drawing No.

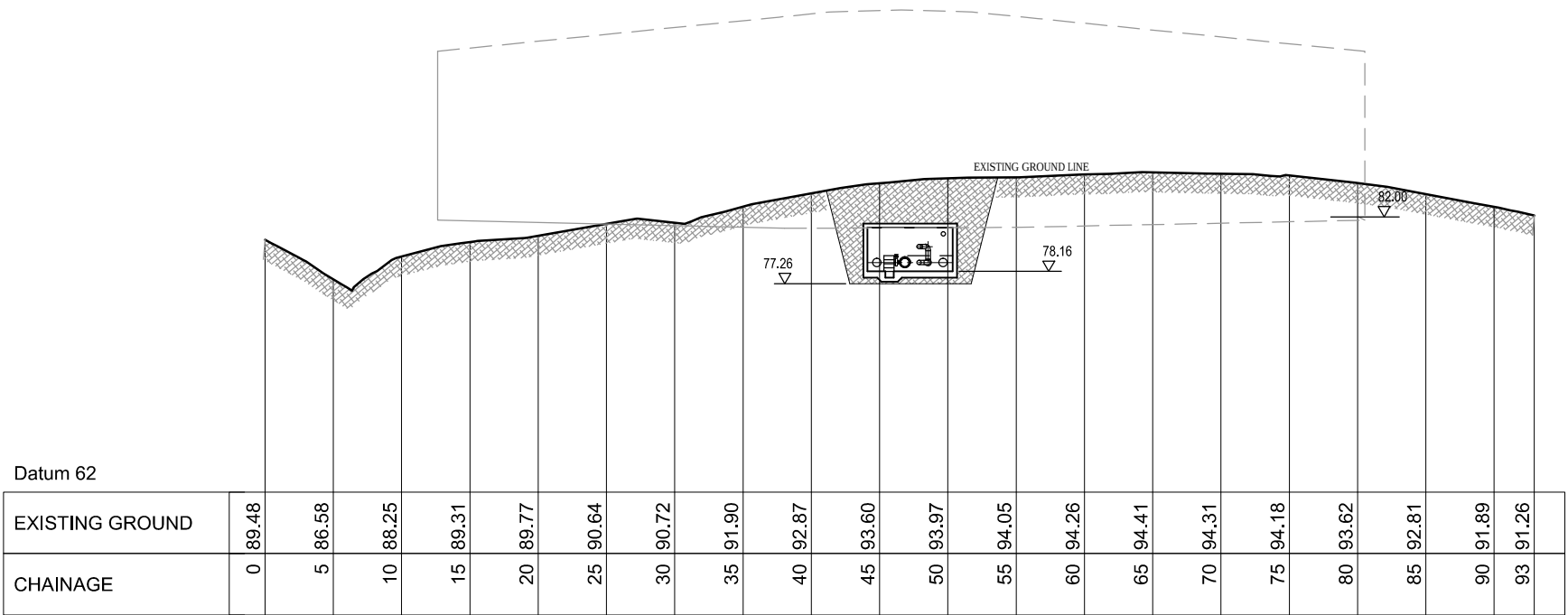
6517439-CE-K16

Rev.

C



4 SECTION
CE-K25 1:250(A1)



3 SECTION
CE-K25 1:250(A1)

PRELIMINARY
NOT FOR CONSTRUCTION

B	UPDATE FINAL ISSUE CDOR	AJP	CJM	SAE	01.02.13
A	FOR CLIENT REVIEW	AJP	CJM	SAE	20.09.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:			Original Scale (A1) 1:250 (A1) Reduced Scale (A3) 1:500 (A3)	Designed Drawn Dsg Verifier Dwg Check	CJM AJP SAE	09.12	Approved For Construction*	Date
* Refer to Revision 1 for Original Signature								

Client:	
Project:	HOSPITAL PRINCE OF WALES RESERVOIR

Title:	PIPE TUNNEL CROSS SECTION OPTION - R1.0

Discipline:	CIVIL
Drawing No.	6517439-CE-K17
Rev.	B

B	UPDATE FINAL ISSUE CDOR	AJP	SAE	SAE 01.02.13
A	FOR GEOTECH REPORT ADDENDUM	AJP	SAE	SAE 18.01.13
No.	Revision	By	Chk	Appd Date

Drawing Originator:		 CH2M BCO	 capacity infrastructure group SAFETY VISORS	Original Scale (A1) 1:250 (A1) Reduced Scale (A3) 1:500 (A3)	<table><tr><td>Designed</td><td>SAE</td><td>01.13</td><td rowspan="4">Approved For Construction*</td></tr><tr><td>Drawn</td><td>AJP</td><td>01.13</td></tr><tr><td>Dwg Verifier</td><td>JVP</td><td>01.13</td></tr><tr><td>Dwg Check</td><td>SAE</td><td>01.13</td><td>Date</td></tr></table>	Designed	SAE	01.13	Approved For Construction*	Drawn	AJP	01.13	Dwg Verifier	JVP	01.13	Dwg Check	SAE	01.13	Date
Designed	SAE	01.13	Approved For Construction*																
Drawn	AJP	01.13																	
Dwg Verifier	JVP	01.13																	
Dwg Check	SAE	01.13		Date															
* Refer to Revision 1 for Original Signature																			

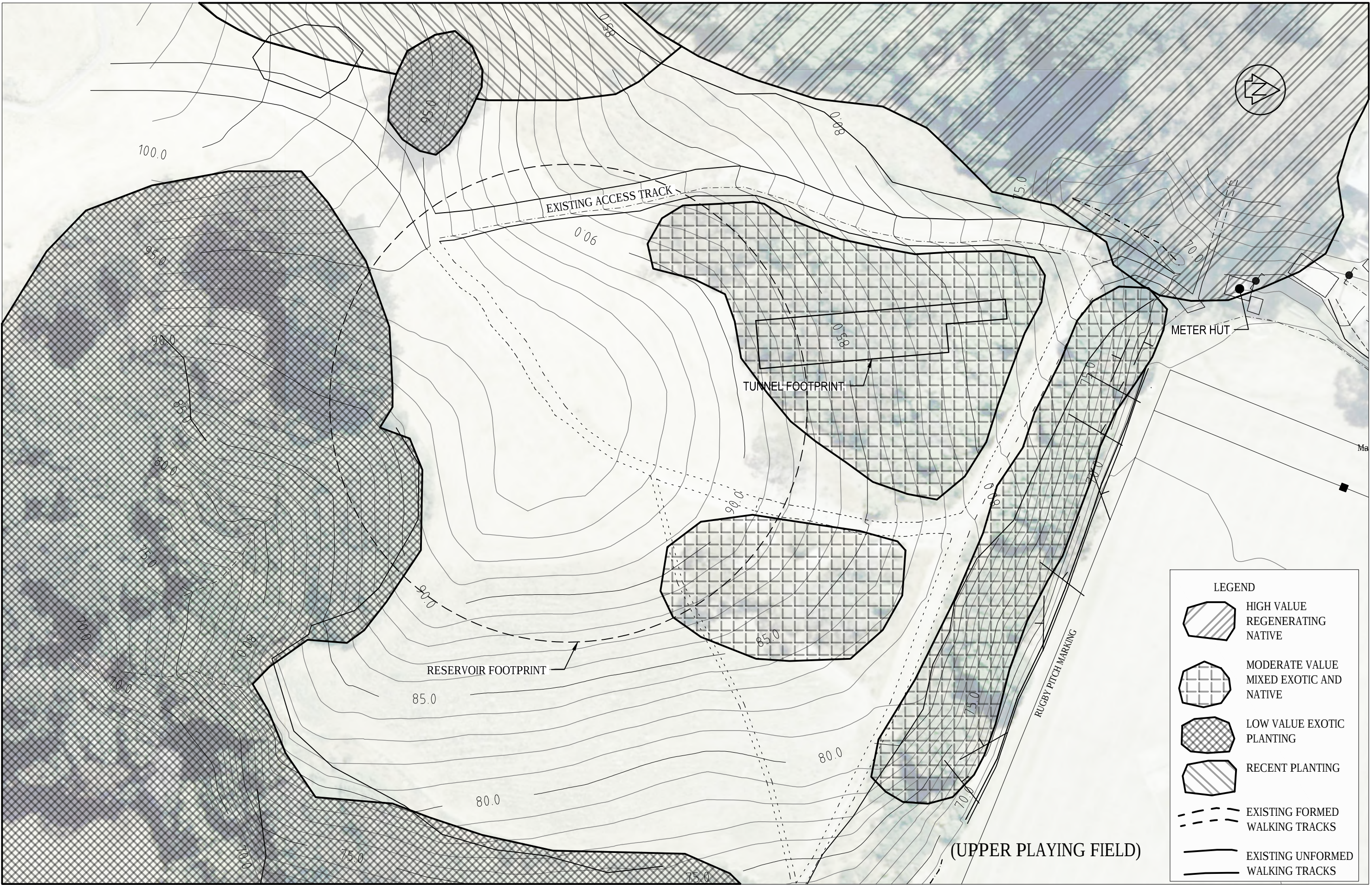
Client: 

Project:	HOSPITAL PRINCE OF WALES RESERVOIR

Title: SECTIONS OF RESERVOIR
OPTION - R1.0

Discipline	
CIVIL	
Drawing No.	Rev.
6517439-CE-K18	B

Discipline		CIVIL
Drawing No.	6517439-CE-K25	Rev. B



PRELIMINARY
NOT FOR CONSTRUCTION

A	ISSUED FOR CDOR	HLV	SRW	SAE	01.02.13
No.	Revision	By	Chk	Appd	Date

Drawing Originator:



Original Scale (A1)	500	Design	JDB	Approved For Issue*
Reduced Scale (A3)	1000	Drawn	HLV	26.07.12
		Checked	JDB	26.06.12
		Dwg Check	SAE	27.06.12
				Date

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

EXISTING SITE
FEATURES

Discipline:

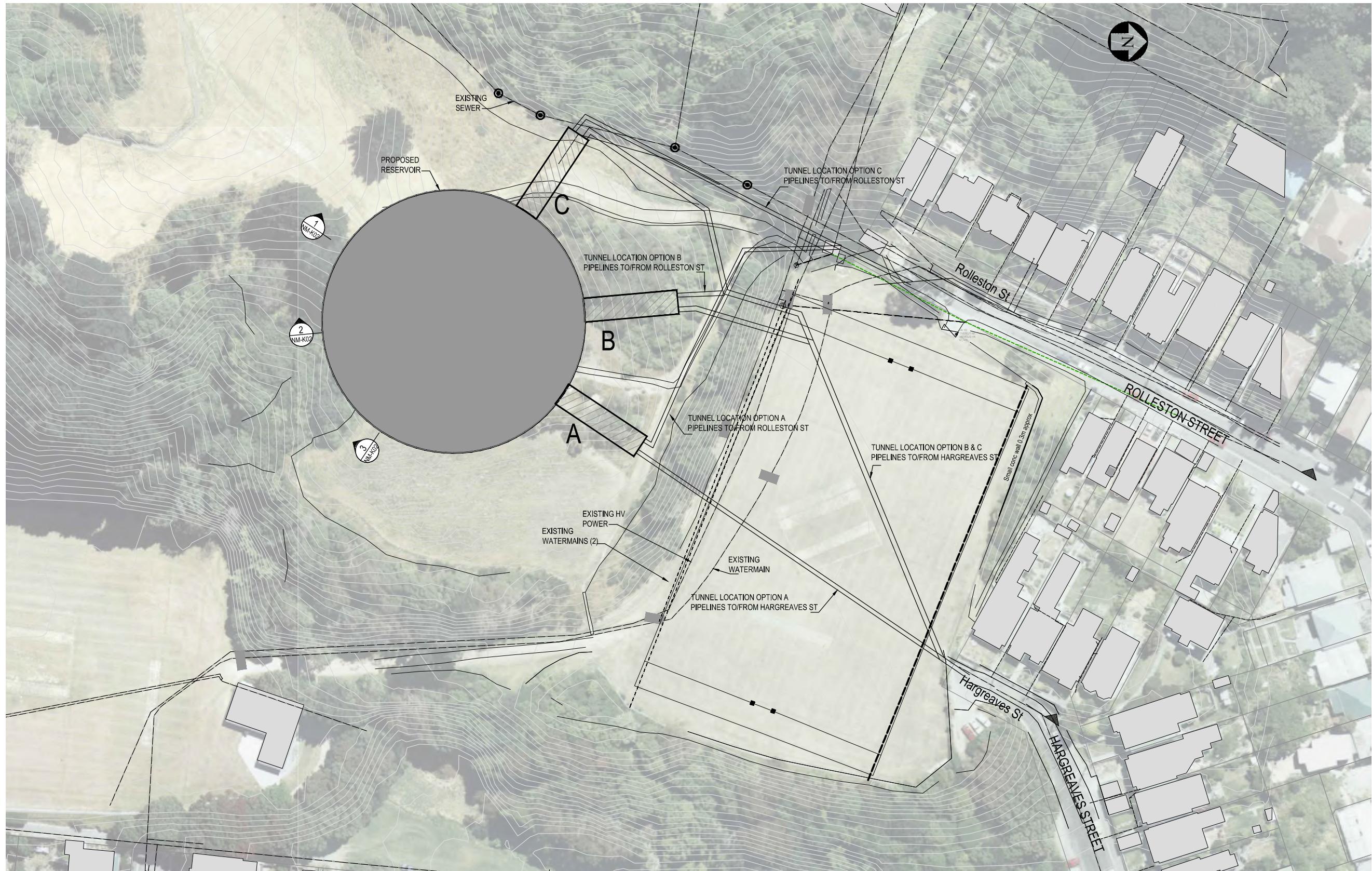
CIVIL

Drawing No.

6517439-CE-K30

Rev.

A



NOTES

A, B, C = OPTIONAL POSITIONS FOR PIPE TUNNEL OPTIONS 1, 2, 3

PLAN
1:500

PRELIMINARY
NOT FOR CONSTRUCTION

A FOR INFORMATION		GW	DRH	SAE	15.06.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:		

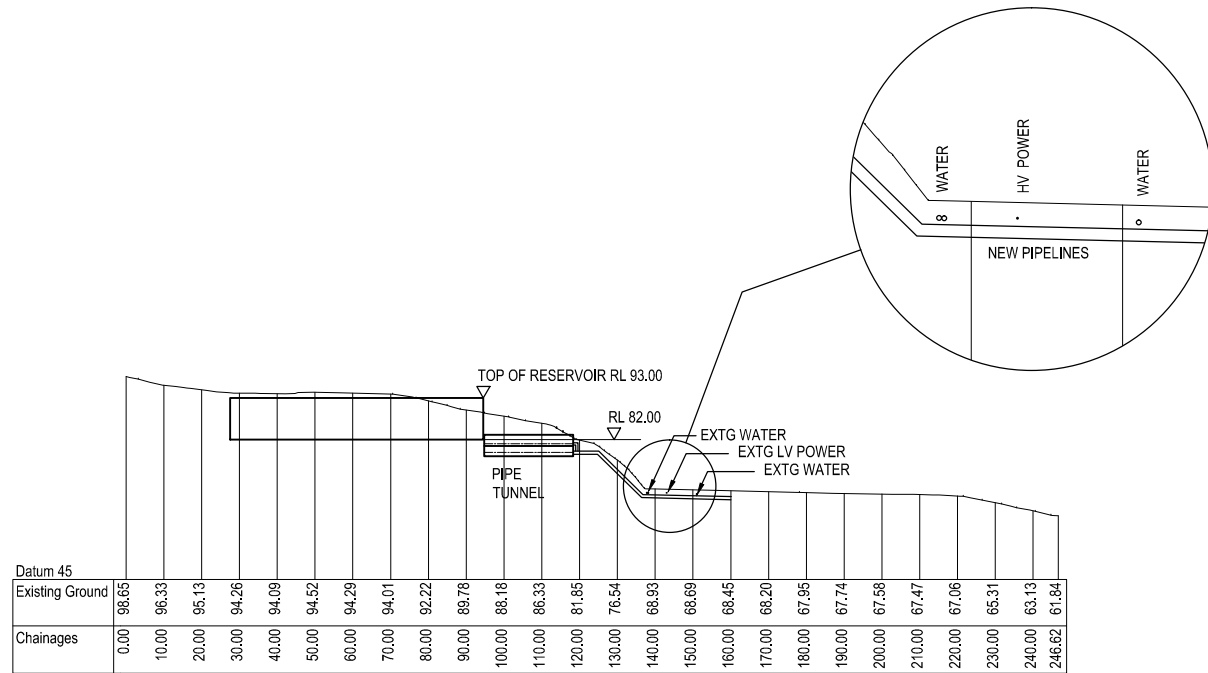
Original Scale (A1)	1:500	Designed	GAL MY	15.06.12	Approved For Construction*
Reduced Scale (A3)		Drawn	GW GAI	15.06.12	
		Dwg Verifier	SE	15.06.12	
		Dwg Check	DRH	15.06.12	
* Refer to Revision 1 for Original Signature					

Client:	

Project:	HOSPITAL PRINCE OF WALES RESERVOIR

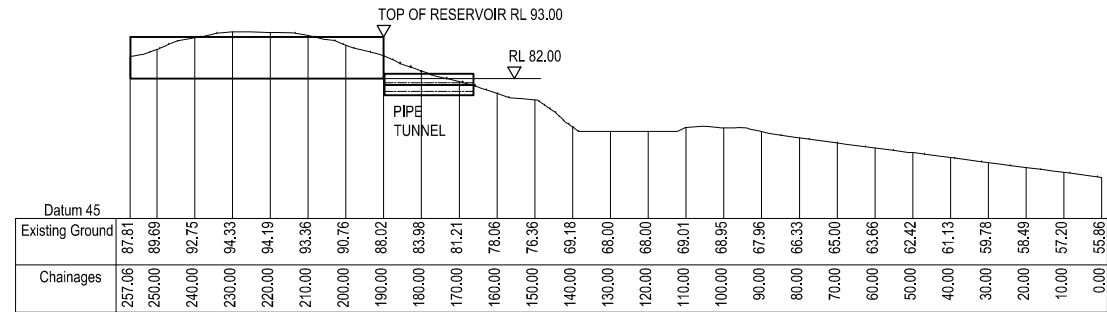
Title:	PLAN TUNNEL LOCATION OPTIONS

Discipline:	MECHANICAL
Drawing No.	6517439-NM-K01
Rev.	A



Horizontal Scale 1:1000
Vertical Scale 1:1000

1 SECTION
NM-K01 1:1000

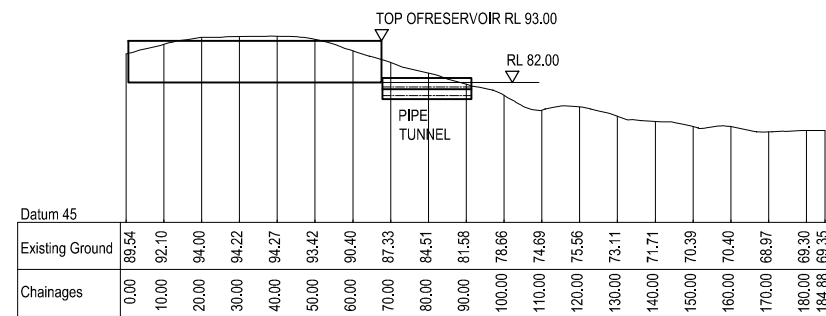


Horizontal Scale 1:1000
Vertical Scale 1:1000

2 SECTION
NM-K01 1:1000

NOTES

- TUNNEL OPTION 2 (TWO LEVEL) SHOWN
- TUNNEL MAY BE SET TO LOWER LEVEL TO BE LESS VISIBLE



Horizontal Scale 1:1000
Vertical Scale 1:1000

3 SECTION
NM-K01 1:1000

A	FOR ONFORMATION	GW	DRH	SAE	15.06.12
No.	Revision	By	Clk	Appd	Date

Drawing Originator:	Original Scale (A1) 1:1000	Designed	GAL, MY	15.06.12	Approved For Construction*
		Drawn	GW, GAL	15.06.12	
		Dwg Verifier	SE	15.06.12	
		Dwg Check	DRH	15.06.12	Date
		* Refer to Revision 1 for Original Signature			

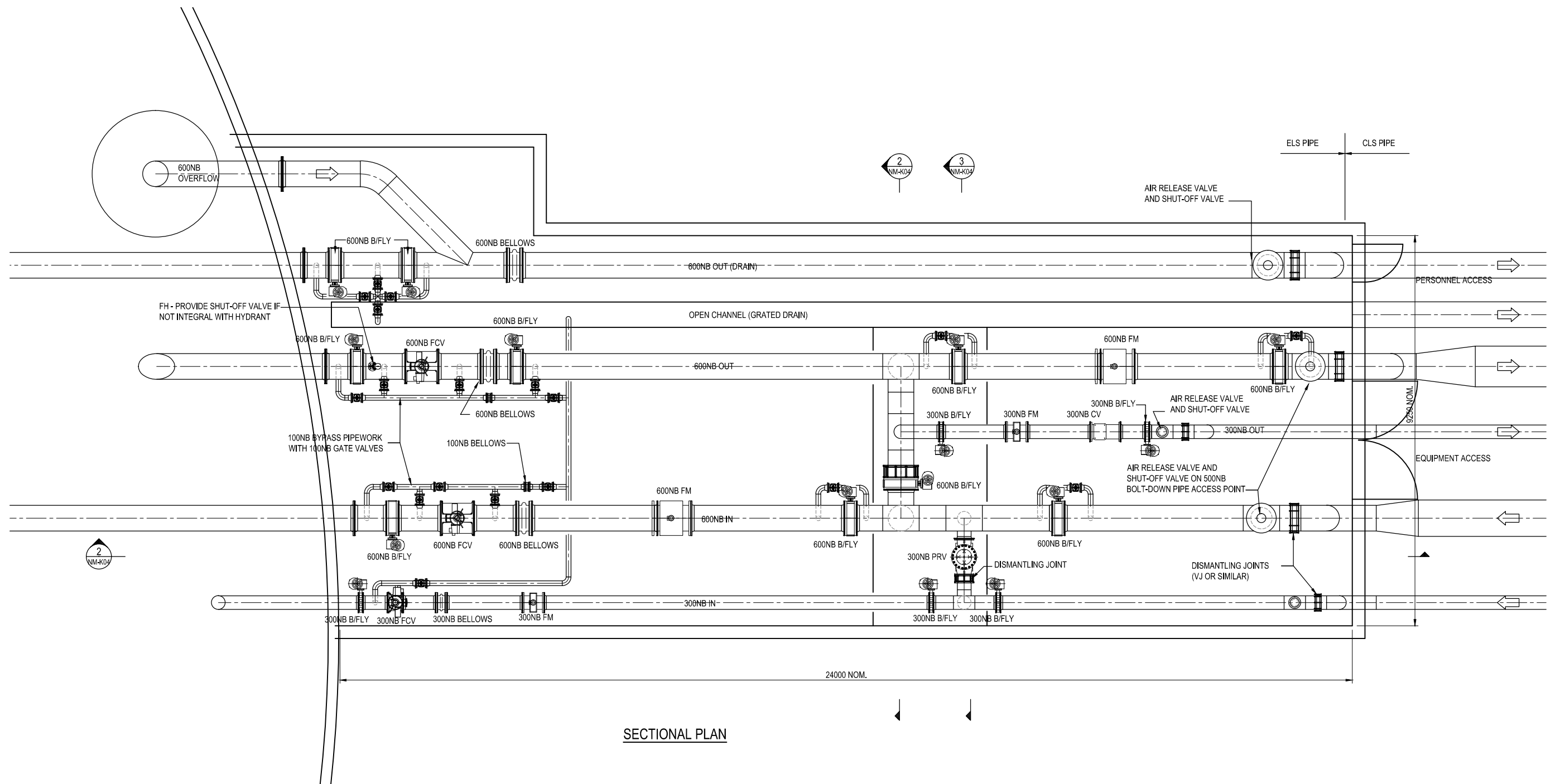
Client:	Absolutely Positively No Work No Money Wellington City Council

Project:	HOSPITAL PRINCE OF WALES RESERVOIR

Title:	SECTIONS TUNNEL LAYOUT OPTIONS

Discipline:	MECHANICAL
Drawing No.	6517439-NM-K02
Rev.	A

PRELIMINARY
NOT FOR CONSTRUCTION



LEGEND

B/FLY BUTTERFLY VALVE
CV CHECK VALVE
FCV FLOW CONTROL VALVE
FH FIRE HYDRANT
FM FLOW METER

No.	Revision	By	Chk	Appd	Date
A	FOR INFORMATION	GAI	DRH	SAE	15.06.12

Drawing Originator:



Original Scale (A1) 1:50
Reduced Scale (A3)

Designed	GAI, MY	15.06.12	Approved For Construction*
Drawn	GAI	15.06.12	
Dwg Verifier	SE	15.06.12	
Dwg Check	DRH	15.06.12	

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

TUNNEL PIPEWORK LAYOUT
OPTION 1.0
PLAN

Discipline:

MECHANICAL

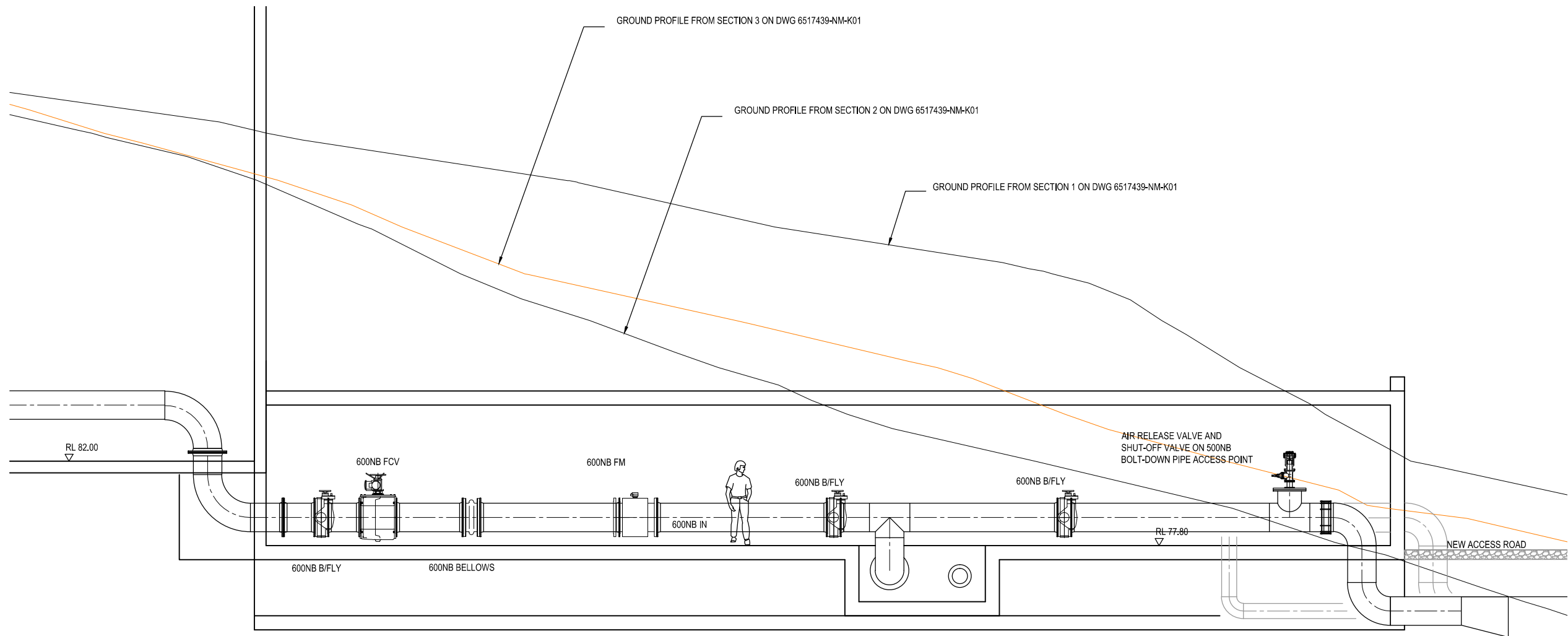
Drawing No.

6517439-NM-K03

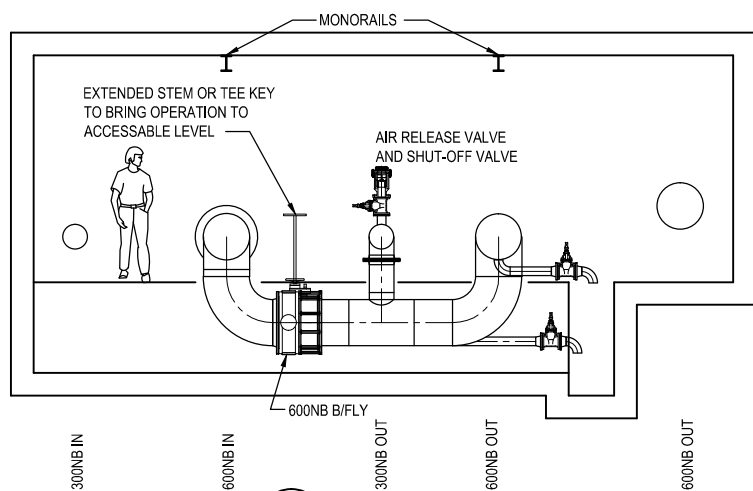
Rev.

A

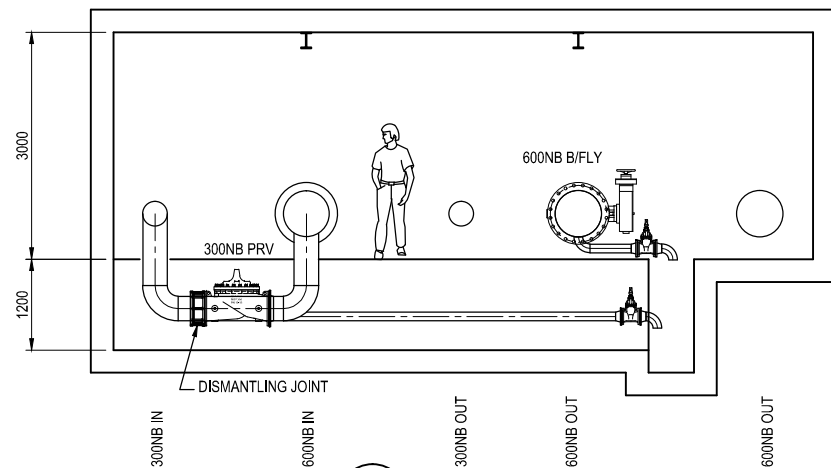
PRELIMINARY
NOT FOR CONSTRUCTION



1 SECTION
NM-K03



2 SECTION
NM-K03



3 SECTION
NM-K03

LEGEND

B/FLY BUTTERFLY VALVE
CV CHECK VALVE
FCV FLOW CONTROL VALVE
FH FIRE HYDRANT
FM FLOW METER

PRELIMINARY
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	FOR INFORMATION	GAI	DRH	SAE	15.06.12

Drawing Originator:



Original Scale (A1) 1:50
Reduced Scale (A3)

Designed	GAI, MY	15.06.12	Approved For Construction*
Drawn	GAI	15.06.12	
Dwg Verifier	SE	15.06.12	
Dwg Check	DRH	15.06.12	

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

TUNNEL PIPEWORK LAYOUT
OPTION 1.0
SECTIONS

Discipline:

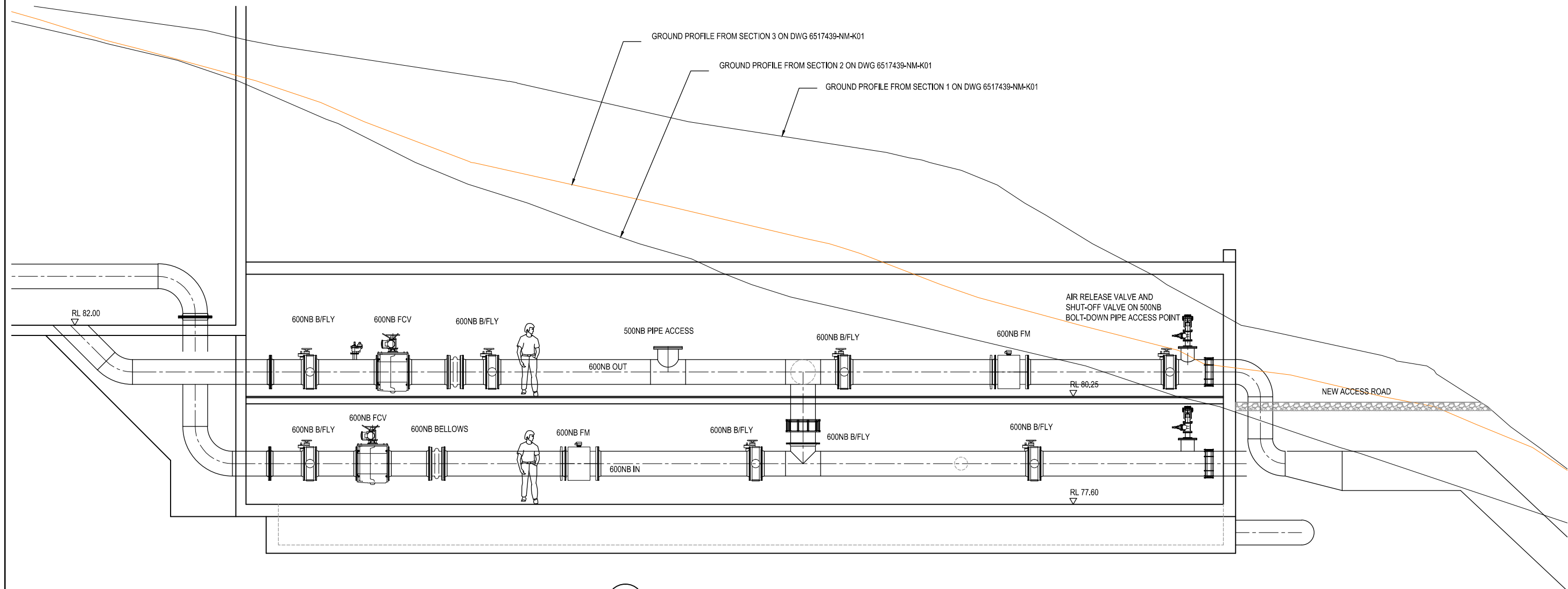
MECHANICAL

Drawing No.

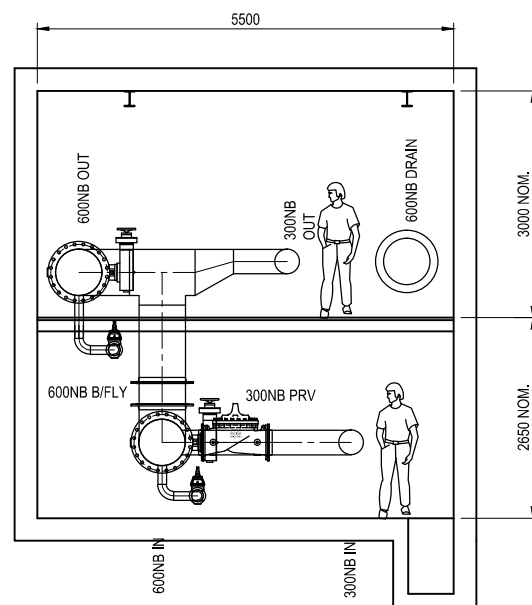
6517439-NM-K04

Rev.

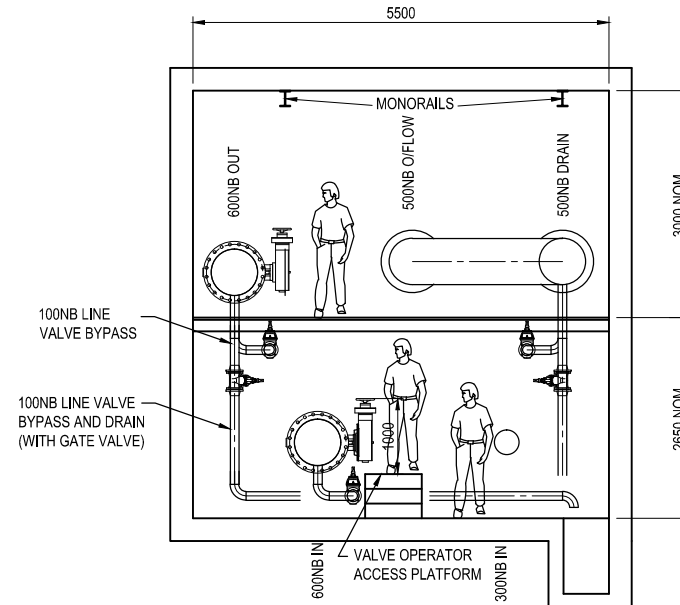
A



1 SECTION
NM-K05



2 SECTION
NM-K05 1:50



3 SECTION
NM-K05 1:50

LEGEND

B/FLY BUTTERFLY VALVE
CV CHECK VALVE
FCV FLOW CONTROL VALVE
FH FIRE HYDRANT
FM FLOW METER

PRELIMINARY
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
A	FOR INFORMATION	GAI	DRH	SAE	15.06.12

Drawing Originator:



Original Scale (A1) 1:50	Designed GAI MY 15.06.12	Approved For Construction*
Reduced Scale (A3)	Drawn GAI 15.06.12	
	Dwg Verifier SE 15.06.12	
	Dwg Check DRH 15.06.12	Date

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

TUNNEL PIPEWORK LAYOUT
OPTION 2.0
SECTIONS

Discipline:

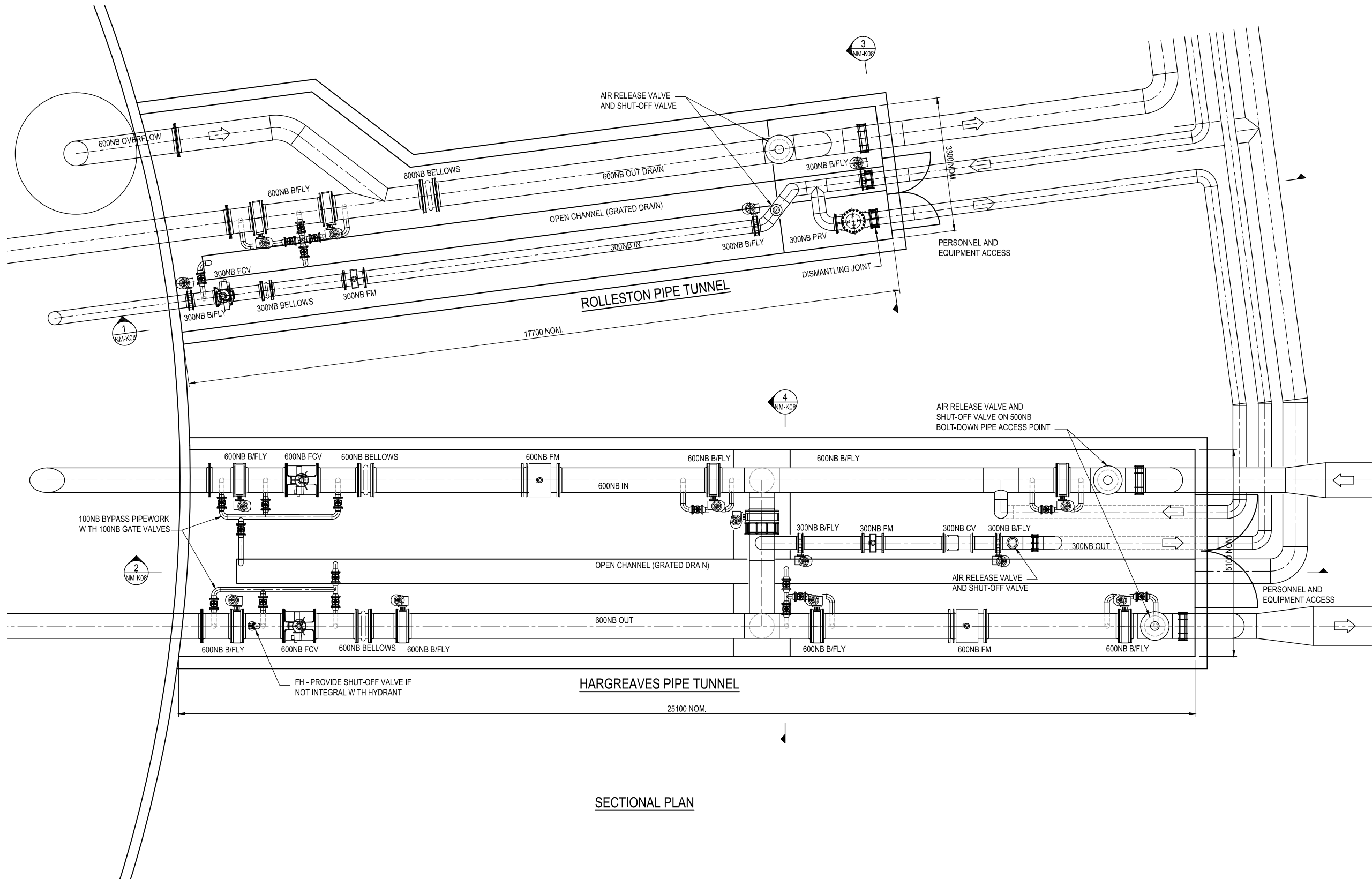
MECHANICAL

Drawing No.

6517439-NM-K06

Rev.

A



LEGEND

B/FLY BUTTERFLY VALVE
CV CHECK VALVE
FCV FLOW CONTROL VALVE
FH FIRE HYDRANT
FM FLOW METER

A		FOR INFORMATION	GAI	DRH	SAE	15.06.12
No.	Revision		By	Chk	Appd	Date

Drawing Originator:




Original Scale (A1)	Designed	GAI, MY	15.06.12	Approved For Construction*
Reduced Scale (A3)	Drawn	GAI	15.06.12	
	Dwg Verifier	SE	15.06.12	
	Dwg Check	DRH	15.06.12	
* Refer to Revision 1 for Original Signature				

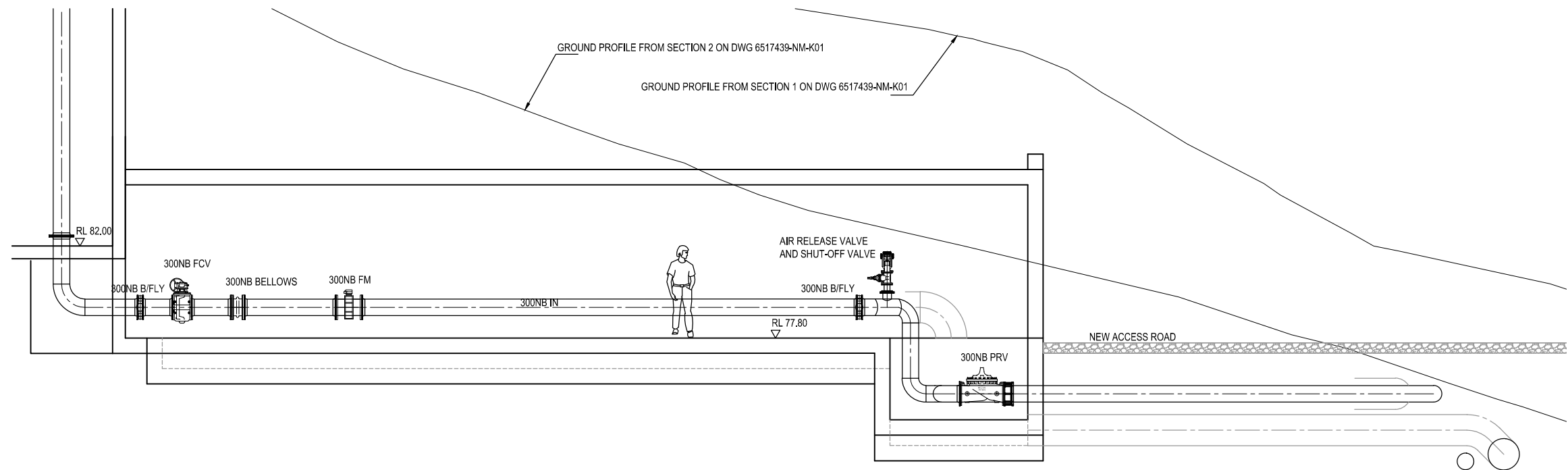
Client:


Project:
HOSPITAL
PRINCE OF WALES
RESERVOIR

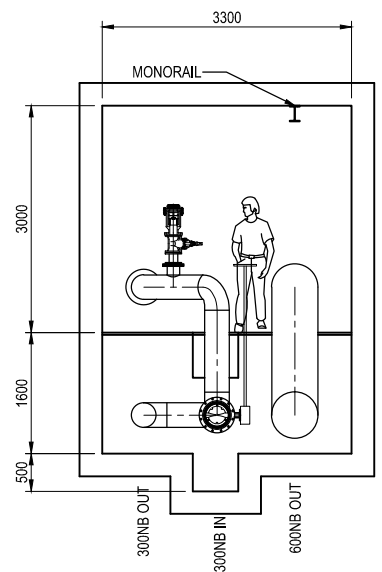
Title:
TUNNEL PIPEWORK LAYOUT
OPTION 3.0
PLAN

Discipline:		MECHANICAL	
Drawing No.	Rev.	6517439-NM-K07	A

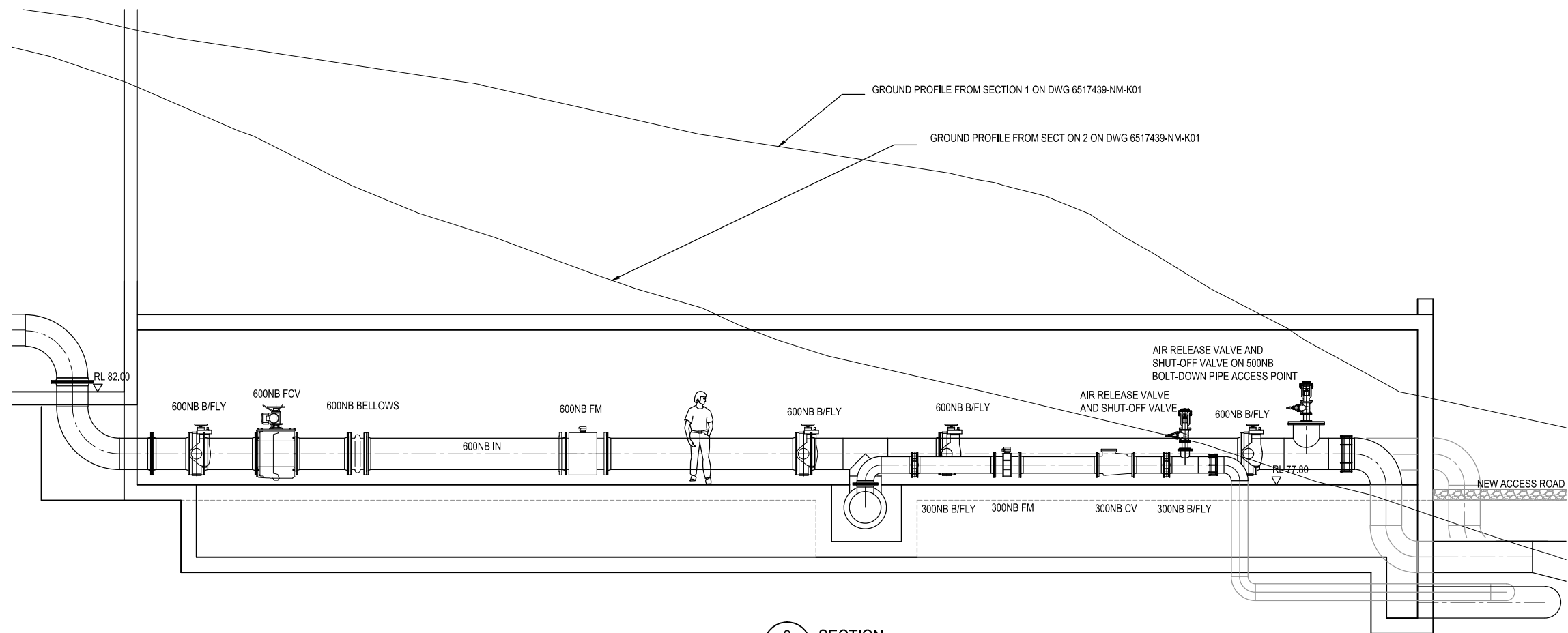
PRELIMINARY
NOT FOR CONSTRUCTION



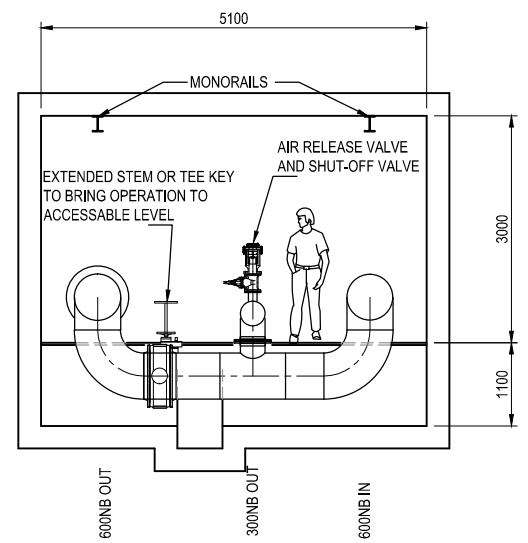
1 SECTION
NM-K07



2 SECTION
NM-K07



3 SECTION
NM-K07



4 SECTION
NM-K07

LEGEND

B/FLY BUTTERFLY VALVE
CV CHECK VALVE
FCV FLOW CONTROL VALVE
FH FIRE HYDRANT
FM FLOW METER

No.	Revision	By	Chk	Appd	Date
A	FOR INFORMATION	GAI	DRH	SAE	15.06.12

Drawing Originator:



Original Scale (A1) 1:50	Designed GAI MY 15.06.12	Approved For Construction*
Reduced Scale (A3)	Drawn GAI 15.06.12	
	Dwg Verifier SE 15.06.12	
	Dwg Check DRH 15.06.12	Date

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

TUNNEL PIPEWORK LAYOUT
OPTION 3.0
SECTIONS

Discipline:

MECHANICAL

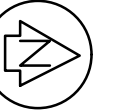
Drawing No.

6517439-NM-K08

Rev.

A

PRELIMINARY
NOT FOR CONSTRUCTION



Legend

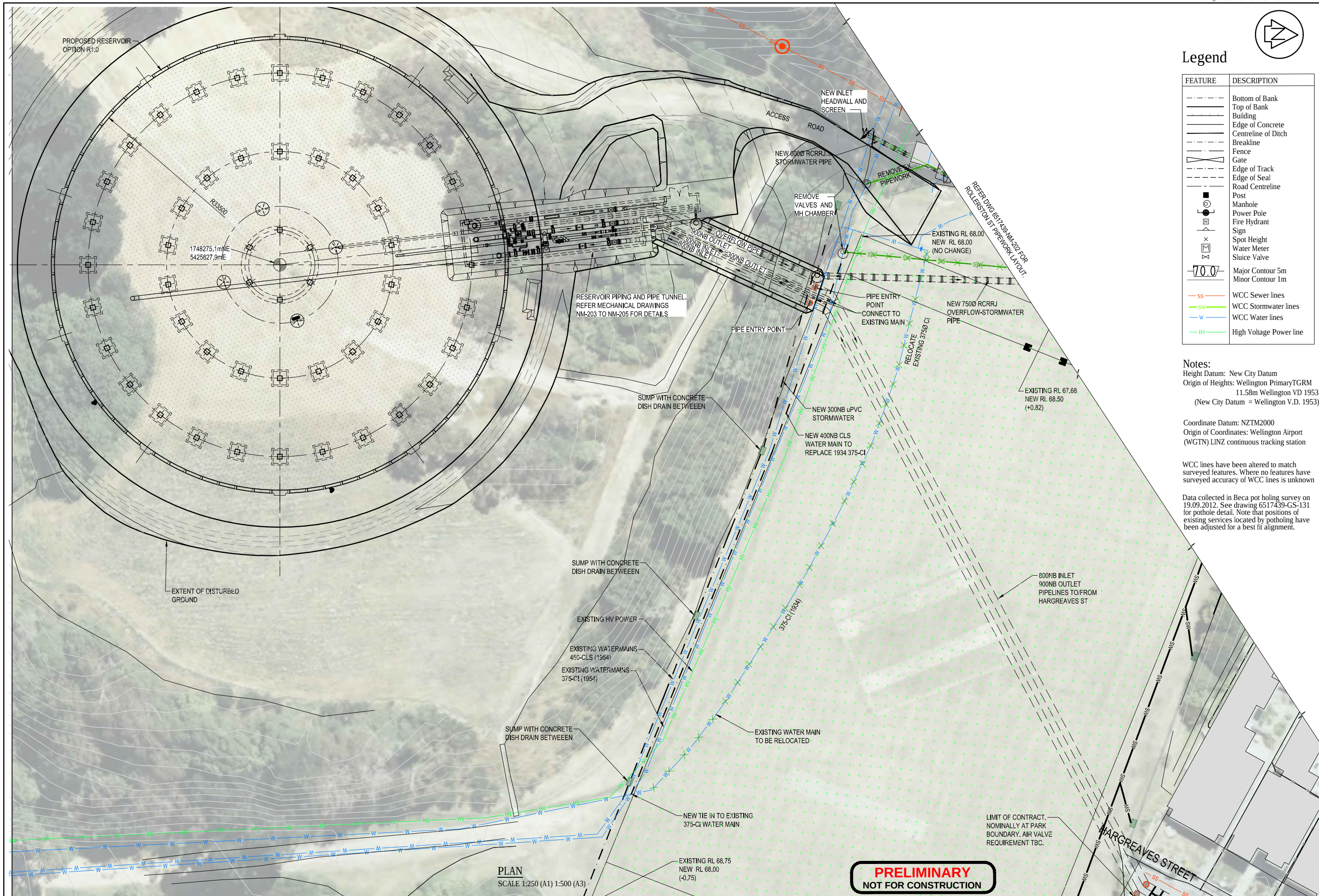
FEATURE	DESCRIPTION
	Bottom of Bank
	Top of Bank
	Building
	Edge of Concrete
	Centreline of Ditch
	Breakline
	Fence
	Gate
	Edge of Track
	Edge of Seal
	Road Centreline
	Post
	Manhole
	Power Pole
	Fire Hydrant
	Sign
	Spot Height
	Water Meter
	Sluice Valve
	Major Contour 5m
	Minor Contour 1m
	WCC Sewer lines
	WCC Stormwater lines
	WCC Water lines
	High Voltage Power line

Notes:
Height Datum: New City Datum
Origin of Heights: Wellington PrimaryTGRM
11.58m Wellington VD 1953
(New City Datum = Wellington V.D. 1953)

Coordinate Datum: NZTM2000
Origin of Coordinates: Wellington Airport
(WGTM) LINZ continuous tracking station

WCC lines have been altered to match surveyed features. Where no features have surveyed accuracy of WCC lines is unknown

Data collected in Beca pot holing survey on 19.09.2012. See drawing 6517439-GS-131 for pothole detail. Note that positions of existing services located by potholing have been adjusted for a best fit alignment.



PLAN
SCALE 1:250 (A1) 1:500 (A3)

PRELIMINARY
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
B	UPDATE FINAL ISSUE CDOR	AJP	DRH	SAE	01.02.13
A	FOR CLIENT REVIEW	AJP	DRH	SAE	03.10.12

Drawing Originator:		
Original Scale (A1)	1:250	
Reduced Scale (A3)	1:500	

Designed	GAL MY	Approved For Construction*	
Drawn	AJP	Date	
Drawn/Check	SAE		
Drawn/Check	DRH		
* Refer to Revision 1 for Original Signature			

Client:		Project:	HOSPITAL PRINCE OF WALES RESERVOIR
MILTONS OFF FURNISH (Wellington)			

Title:	MECHANICAL SITE LAYOUT PLAN SHEET 1

Discipline	MECHANICAL
Drawing No.	6517439-NM-201
Rev.	B

FEATURE	DESCRIPTION
	Bottom of Bank
	Top of Bank
	Building
	Edge of Concrete
	Centreline of Ditch
	Breakline
	Fence
	Gate
	Edge of Track
	Edge of Seal
	Road Centreline
	Post
	Manhole
	Power Pole
	Fire Hydrant
	Sign
	Spot Height
	Water Meter
	Sluice Valve
	Major Contour 5m
	Minor Contour 1m
	WCC Sewer lines
	WCC Stormwater lines
	WCC Water lines
	High Voltage Power line

Height Datum: New City Datum
Origin of Heights: Wellington Primary TGRM
11.58m Wellington VD 1953
(New City Datum = Wellington V.D. 1953)

Coordinate Datum: NZTM2000
Origin of Coordinates: Wellington Airport
(WGTN) LINZ continuous tracking station

WCC lines have been altered to match surveyed features. Where no features have surveyed accuracy of WCC lines is unknown

Pot holing survey data from Beca survey on 19.09.2012 incorporates.

1. For costing purposes, all manholes to be 1200mm I and 2.0m deep unless noted otherwise.
2. Manhole lids buried if located in sports field. Allow full height for costing purposes.
3. New SW pipes to have water stops at every 7.5m in accordance with WCC standard detail DR03.
4. SW trench backfill and surrounds in accordance with WCC standard detail DR03.
4. SW manholes to WCC standard detail DR01. All manholes 1200 I unless noted otherwise.
5. All SW pipes to be RCRRJ class 4.

PRELIMINARY
NOT FOR CONSTRUCTION

										Drawing Originator:  capacity infrastructure services										Client: Absolutely POSITIVELY NO MESS OR SLACKS WELLINGTON CITY COUNCIL										Project: HOSPITAL PRINCE OF WALES RESERVOIR										Title: MECHANICAL SITE LAYOUT PLAN SHEET 2										Discipline: MECHANICAL									
B UPDATE FINAL ISSUE CDOR										AJP		DRH		SAE		01.02.13		Original Scale (A1) 200										Drawn		AJP		09.12		Approved For Issue:																									
A FOR CLIENT REVIEW										AJP		DRH		SAE		03.10.12		Reduced Scale (A3) 400										Check						Date																									
No. Revision										By		Chk		Appd		Date		* Refer to Revision 1 for Original Signature										Dwg Check																															

600 IN

600 IN

300 IN

PIPEWORK DETAILS PLAN

SCALE 1:50 (A1) 1:100 (A3)

MATCHING LINE

B	UPDATE FINAL ISSUE CDOR	GW	DRH	SAE 01.02.13
A	FOR CLIENT REVIEW	GW	DRH	SAE 03.10.12
No.	Revision	By	Chk	Date



Original Scale (A1)	1:50
Reduced Scale (A3)	1:100

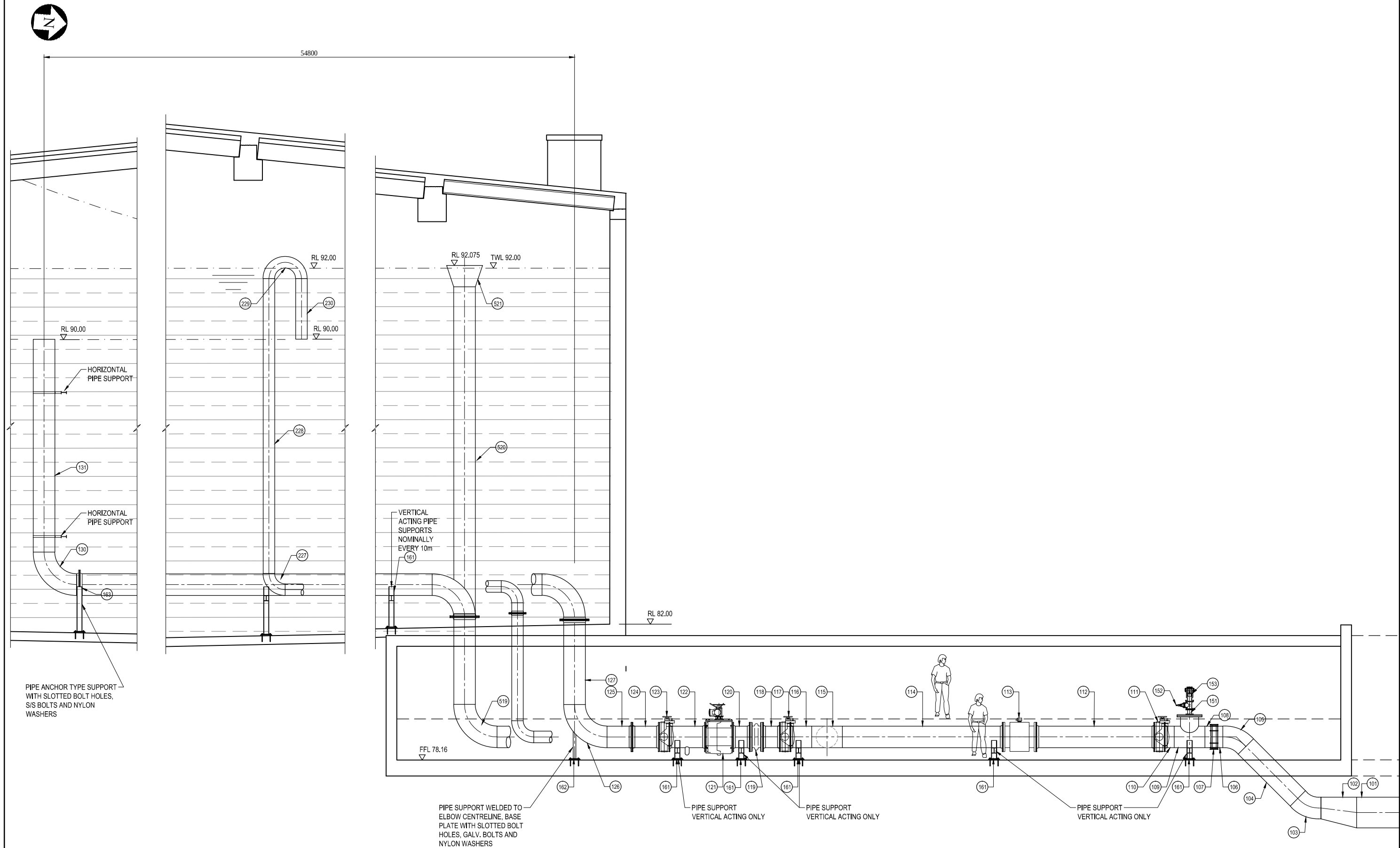
Designed	GAI, MY	09.12	Approved For Construction*
Drawn	GW GAI	09.12	
Dsg. Verifier	SE	09.12	
Dwg Check	DRH	09.12	Date

* Refer to Revision 1 for Original Signature

Absolutely POSITIVELY
We Mean It! Please, Wellington
Wellington Gift Centre

Project: **HOSPITAL
PRINCE OF WALES
RESERVOIR**

Discipline		MECHANICAL	
Drawing No.		Rev.	
6517439-NM-203		B	



1 SECTION
NM-203 SCALE 1:50 (A1) 1:100 (A3)

PRELIMINARY
NOT FOR CONSTRUCTION

No.	Revision	By	Chk	Appd	Date
B	UPDATE FINAL ISSUE CDOR	GW	DRH	SAE	01.02.13
A	FOR CLIENT REVIEW	GW	DRH	SAE	03.10.12

Drawing Originator:



Original Scale (A1) 1:50	Designed GAL MY 09.12	Approved For Construction*
Reduced Scale (A3) 1:100	Drawn GW GAI 09.12	Date
	Dwg Verifier SE 09.12	
	Dwg Check DRH 09.12	

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

PIPE TUNNEL
SECTIONS
SHEET 1

Discipline:

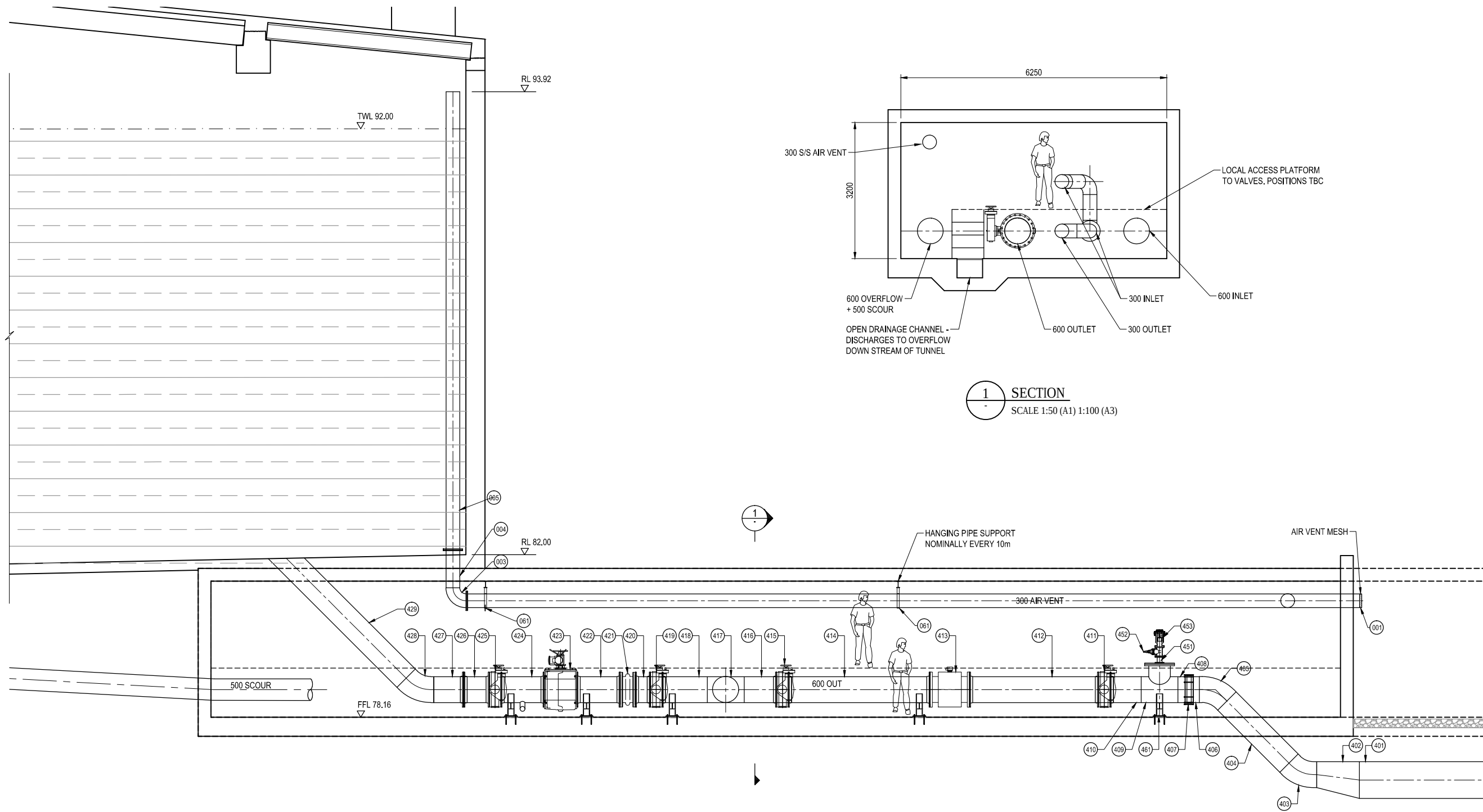
MECHANICAL

Drawing No.

6517439-NM-204

Rev.

B



1 SECTION
SCALE 1:50 (A1) 1:100 (A3)

2 SECTION
NM-203 SCALE 1:50 (A1) 1:100 (A3)

B	UPDATE FINAL ISSUE CDOR	GW	DRH	SAE	01.02.13
A	FOR CLIENT REVIEW	GW	DRH	SAE	03.10.12
No.	Revision	By	Chk	Appd	Date

Drawing Originator:



Original Scale (A1) 1:50
Reduced Scale (A3) 1:100

Designed	GAL, MY
Drawn	GW GAL
Dsg Verifier	SE
Dwg Check	DRH

* Refer to Revision 1 for Original Signature

Client:



Project:

HOSPITAL
PRINCE OF WALES
RESERVOIR

Title:

PIPE TUNNEL
SECTIONS
SHEET 2

Discipline:

MECHANICAL

Drawing No.

6517439-NM-205

Rev.

B

PRELIMINARY
NOT FOR CONSTRUCTION

