February 2015

GEOTECHNICAL STAGE THREE REPORT: FALLS DAM PRELIMINARY DESIGN AND COST ESTIMATE

Manuherikia Catchment Feasibility Study

Submitted to:
The Manuherikia Catchment Water Strategy Group

Report Number: 1378110270-3000-3090-R-Rev1-313

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this geotechnical *Stage Three Falls Dam Preliminary Design and Cost Estimate* report is to document the preliminary designs and cost estimates for the new roller compacted concrete (RCC) dam with full supply levels of 592.2 m, 580.4 m and 570.6 m options (previously referred to as the 27 m dam raise, 15 m dam raise and 6 m dam raise options, respectively) at Falls Dam. The full supply level 592.2 m option is described in detail herein while the designs for the full supply levels 580.4 m and 570.6 m options are briefly summarized since they have the same design criteria as the full supply level 592.2 m option.

The final dam configuration (size and type) will not be confirmed by the Manuherikia Catchment Water Strategy Group (MCWSG) until after the current feasibility study. Future final design and analysis work such as the potential impact category (PIC) classification, static and seismic stability evaluation, and spillway and offtake structure configurations will need verification once the final height is determined.

This report completes the preliminary design drawings, descriptive report, cost estimate based on preliminary design development, dam break assessment, and construction methodology scope of work as part of the Geotechnical and Engineering Assessment portion of the MCWSG Feasibility Study.

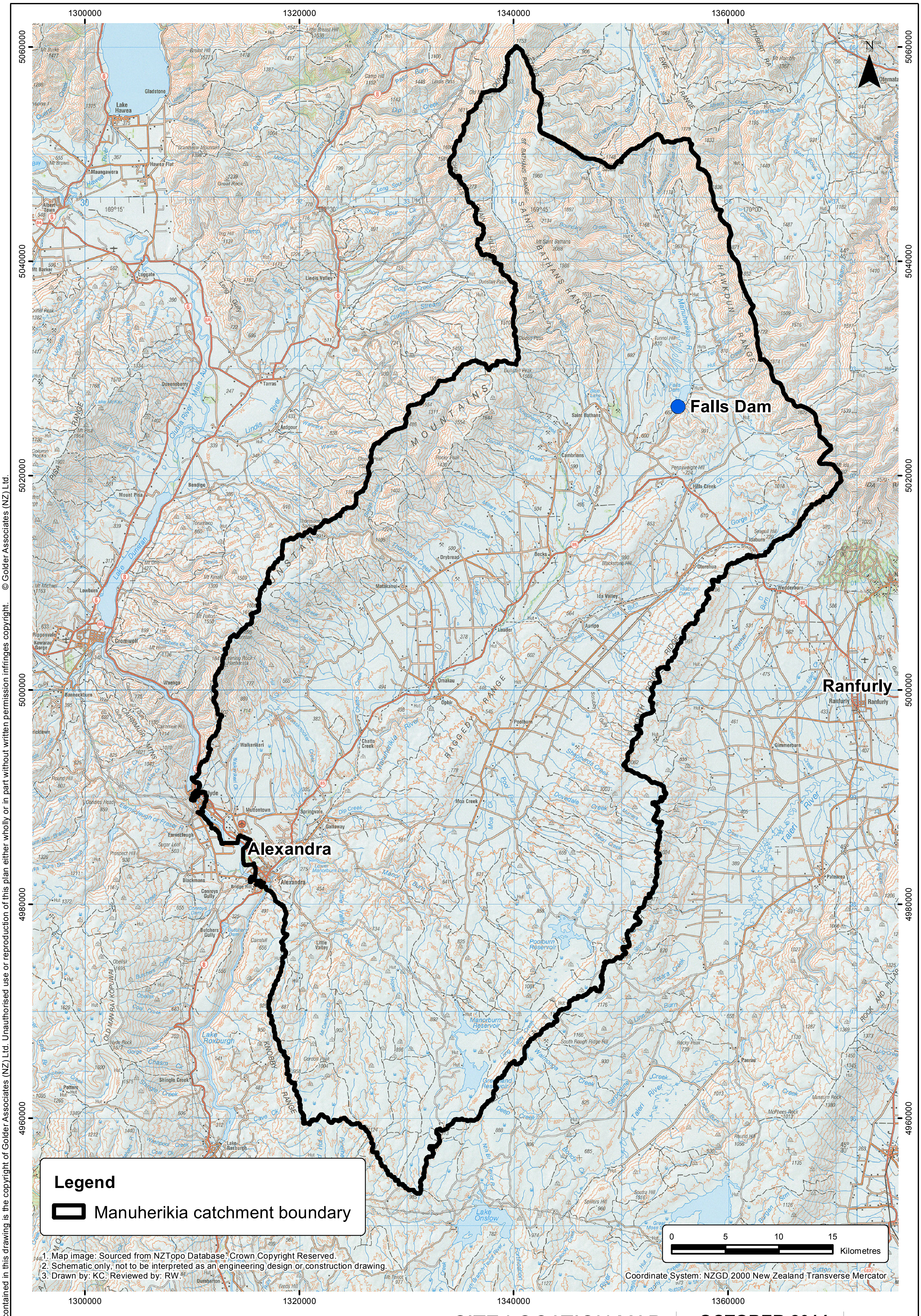
1.2 Report Limitations

Your attention is drawn to the document, “Report Limitations”, as attached (Appendix A). The statements presented in that document are intended to advise you of what your realistic expectations of this report should be, and to present you with recommendations on how to minimise the risks to which this report relates which are associated with this project. The document is not intended to exclude or otherwise limit the obligations necessarily imposed by law on Golder Associates (NZ) Limited, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

2.0 PROJECT DESCRIPTION

The new RCC dam options downstream of the existing Falls Dam are being investigated as part of MCWSG Feasibility Study which is examining the capability of the Manuherikia Catchment to provide increased water storage and distribution capabilities for irrigation. The Manuherikia River is the main water source for the valley and it has always been recognized that an increased dam height at the Falls Dam location could provide additional storage capabilities (MoW 1974). The current Falls Dam is located in the upper portion of the catchment and is managed by the Falls Dam Company.

The current Falls Dam has an estimated storage of 10 Mm^3 which provides about 6,500 ha of land with irrigation water. The construction of a new RCC dam downstream of the existing Falls Dam to a full supply level of 592.2 m increases storage to 114.1 Mm^3 and provides better water reliability and additional irrigation capability. The construction of a new RCC dam downstream of the existing Falls Dam to a full supply level of 580.4 m increases storage to 50 Mm^3 and a dam with a full supply level of 570.6 m provides 19 Mm^3 of storage. An overall site map of the project site is presented on Figure 1. Details on the configuration of the existing Falls Dam are provided in the Golder Manuherikia: Falls Dam Recommended Option letter report dated 4 July 2014 (Golder 2014b).



Golder Associates

TITLE SITE LOCATION MAP

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3.0 DESIGN CRITERIA AND METHODOLOGY

The following sections summarise the design criteria and methodology for the new RCC dam option with a full supply level of 592.2 m.

3.1 Operations

During most years, the current Falls Dam fills to full capacity, near the spillway invert, during the spring runoff season and is drawn down during the irrigation season which runs throughout the summer and early fall. The current Falls Dam releases water through a powerhouse located at the downstream toe which then discharges to the Manuherikia River. Flow through the morning glory spillway discharges directly into the Manuherikia River.

Operation of the new Falls Dam with a full supply level of 592.2 m will operate in a similar manner in that it will fill during the spring runoff months and will be lowered throughout the irrigation season. Water will be discharged downstream into a powerhouse, irrigation canal, or the Manuherikia River.

3.2 Hydrology and Hydraulics

3.2.1 Dam Break Analysis

A dam break analysis was performed by Golder in July 2014 and is presented in Appendix B. The dam break analysis concluded that the proposed new RCC dam with a full supply level of 588 m is a high potential impact category (PIC) classification. The full supply level elevation of 588 m was based on old survey data and outdated datum and the new topographic survey data (provided by Landpro, formerly BTW South, in June 2014) indicates that the previously assumed full supply level of 588 m is approximately equal to a full supply level of 592.2 m. The reservoir volume used in the dam break assessment is approximately the same as the reservoir volume with a full supply level of 592.2 m. Once a final dam height is selected, the reservoir volumes should be confirmed and the dam break assessment should be verified. Note that there are many high PIC dams in New Zealand and the classification does not indicate the likelihood of failure but rather identifies the design criteria. The High PIC classification for the proposed new Falls Dam requires the highest (i.e., most stringent) design requirements and results in a more conservative design.

3.2.2 Offtake Structure

The Falls Dam option with a full supply level of 592.2 m includes an offtake structure consisting of an intake tower on the upstream face of the dam, an offtake conduit through the dam, and piping downstream of the dam. The intake tower provides multiple intake elevations to address potential water quality concerns for discharges from the reservoir. A control house is conceptually situated on top of the intake tower at the dam crest to accommodate the equipment for manipulating the position of the intake tower control gates and electrical controls for dam monitoring and reservoir operations. The seismic performance of the proposed intake tower has not been assessed as part of the preliminary design and should be evaluated as part of the final design. Based on the results of the seismic analysis, the intake tower and offtake conduit may need to be anchored to stable rock. The preliminary design provided for the concrete sections of the intake tower present a simple structure and does not account for the possible need for shear keys or other concrete design features which should be assessed as part of final design.

The offtake conduit for the new Falls Dam with a full supply level of 592.2 m is designed for several criteria, including downstream flushing flows, drawdown requirements, irrigation demands, and power generation. The preliminary design considered an estimated flushing flow requirement of 12 cubic metres per second (cumecs). Flushing flows were estimated to be three times the median dam inflow, which Aqualinc provided as 3.88 cumecs (2014a). Drawdown requirements were analysed using criteria for evacuating reservoirs established by the USBR (1990) for a high hazard dam. Drawdown timeframes were modelled using an estimate of the new reservoir stage-storage relationship and different offtake configurations.

The irrigation demand was assumed to be 6 cumecs and power generation was accommodated using a piping connection from the offtake conduit. In this preliminary design, the flushing flows controlled the sizing of the offtake conduit, and only one offtake conduit is proposed; further design and cost information may suggest two offtake conduits could be installed, one for low pressure / high flow discharges (flushing / drawdown) and one for high pressure / low flow discharges (power generation).

3.2.3 Overflow Spillway

The overflow spillway is designed to pass the inflow design flood (IDF). According to New Zealand Society on Large Dams (NZSOLD 2000), the IDF for high potential impact dams is usually between a 1 in 10,000 year event (0.01 % annual probability of occurrence) and the probable maximum flood (PMF). For the preliminary design, the 1 in 10,000 year event was used as the IDF. Aqualinc (2014b) provided an estimate of the 1 in 10,000 year flood watershed runoff hydrograph which Golder routed through the reservoir using the estimated stage-storage relationship to determine the peak discharge through the overflow spillway. The overflow spillway was sized using a rectangular weir equation (with a coefficient of discharge for an ogee weir, but the type of weir should be selected during final design) by varying the length along the dam crest to maintain the required dam freeboard. The overflow spillway flows pass down the RCC dam face into an energy dissipator at the downstream toe, which was sized using USBR (1984) methodology for a Type II stilling basin.

3.2.4 Inflow Design Flood

Determining the probable maximum flood (PMF) is not typically completed during the preliminary design phase, but based on the high PIC classification, the 1 in 10,000 year flood event was used as the IDF for the preliminary analysis. Aqualinc (2013) provided a storm hydrograph for the 1 in 10,000 year event which estimated the peak flow to be 530 m^3/sec. Additional analysis during final design will be required to estimate the PMF event and determine the IDF.

3.3 Seismic Hazard

3.3.1 Deterministic Seismic Hazard

A preliminary deterministic site specific assessment of ground motions at Falls Dam was undertaken for the preliminary dam design and the results are presented in Golder's *Geotechnical Stage One Report: Background Review and Investigations* (Golder 2014b). NZSOLD (2000) specifies that only minor damage is acceptable during the operating basis earthquake (OBE) and the reservoir of the dam is required to be maintained during the maximum design earthquake (MDE). Due to the high PIC classification at Falls Dam, the 1 in 10,000 year event was selected for the MDE for the preliminary design. The deterministic seismic hazard resulted in a peak horizontal ground acceleration of 1.0g for the 1 in 10,000 year ground motion. For the preliminary design, a simplified pseudo-static seismic stability analysis was performed for the MDE event. For the final design, NZSOLD (2000) recommends producing a site specific seismic hazard assessment and evaluating performance of the dam during the MDE and OBE.

3.4 Geotechnical Conditions

3.4.1 Foundation Conditions

The foundation of the Falls Dam site is comprised of colluvium deposits and localised deposits of river gravels (alluvium) overlying Torlesse Group basement rock. The Torlesse Group is composed of deformed bedded 'Greywacke' sandstone (also known as arenite) and mudstone (known as argilite). The surficial deposits (colluvium and alluvium) are seldom more than one meter thick. Further explanation of the foundation conditions at Falls Dam is summarized in the *Geotechnical Stage One Report: Background Review and Investigations* by Golder (Golder 2014a).

Torlesse sandstone is typically strong, having an unconfined compressive strength (UCS) expected to be within the range of 50 to 100 MPa. The mudstone is weaker and is typically sheared and fissile.

The controlling factor for the foundation in terms of stability will be the presence of defects, in particular their orientation, strength, persistence (i.e., length), frequency and openness.

A simplified wedge sliding block analysis was performed using the Dips software by Rocscience (2013) to identify any joint surfaces that could form moveable blocks. The joint data was taken from Appendix K in Golder's *Geotechnical Stage One Report: Background Review and Investigations* (Golder 2014a) and analysed for potential sliding planes daylighting into the abutment excavations. One bedding set and four joint sets were identified from the field data. The joint orientations are presented in Table 1 below.

The most visually dominant defect set is bedding (Set 1), which is highlighted by the alternating layers of light grey sandstone and dark grey mudstone. These are oriented upstream towards the northeast and dip at 60° to 80°. Bedding plane shears were observed in rock outcrops, particularly in the mudstone. South of the powerhouse, bedding is less obvious due to the absence of mudstone. The rock mass in this area is more massive and is dominated by sandstone.

Downstream of the powerhouse are some continuous (greater than 5 m) planar, low angle joints dipping at 20° towards the southwest (234° i.e., downstream). These joints are orientated within defect Set 3 locus and are relevant as they are less favourably orientated in regard to stability and are critical for producing potential failure modes. These joints are mostly smooth, persistent to lengths over 10 m, open, and have clay infilling indicating they may have relatively low friction angles. For the analysis, it was assumed that Set 3 defects have friction angles of 25°.

Table 1: Bedding and Joint Data at Falls Dam.

Set Number	Mean Dip	Mean Dip Direction	Defect Type	Number of Defects in Set
1	70	52	Bedding	20
2	81	330	Joint	15
3	26	244	Joint	10
4	36	141	Joint	7
5	23	65	Joint	6

The sliding wedge analysis indicates the potential for slope failure in the left and right abutments under static loading conditions, however the likelihood of significant failure is considered minimal. This is supported by observations of the exposed rock face in the area downstream of the dam site, which appears to have performed well in the nearly 80 years since exposure. There is little evidence of block failure in this area. Following elevated reservoir levels, there is potential for reduced stability under hydrologic loading conditions, which will require evaluation at the final design stage.

It is recommended that additional mapping of the foundation and analysis be completed prior to final design to determine if the potentially moveable blocks can be supported during construction using small-scale, localised measures (e.g., by over excavation or shotcrete) or if a more robust and extensive stabilisation approach is necessary. For the preliminary design, it is assumed that small-scale measures will be required and that any potential issues will be dealt with during construction. Extensive support is not included in the cost estimate.

No evidence of fault rupture hazard been identified at the footprint of the proposed main dam. The likelihood of a significantly 'active' fault that is, as yet, unrecognised, affecting the dam site, is judged to be acceptably low. The bedrock at the site is judged to be suitable for a concrete dam foundation.

Groundwater was not observed as springs or exiting the face of outcrops during the 2014 field investigations. Groundwater depth and dewatering efforts will have to be verified during future field explorations. No cost for dewatering the dam foundation has been included in the cost estimate.

3.4.2 Site Hazards

Other site hazards, such as reservoir rim stability, landslides, and abandoned mines and gravel pits will have to be further evaluated during final design. There are no known landslides mapped in the reservoir area but there is geomorphic evidence of landslides along the reservoir rim. Impacts to these landslides due to increased reservoir levels and fluctuating reservoir levels should be assessed as part of final design and monitored thereafter.

3.4.3 Stability Analysis

The sliding stability of the RCC section was evaluated under usual, flood, and unusual (10,000 year earthquake event) loading conditions. Detailed discussion of the stability analysis is included in Appendix C. The stability of the dam in sliding and uplift was assessed through calculation of factors of safety (against sliding). The stability of the dam in overturning was based on force resultant location and the check of allowable stresses was done by comparing the normal stresses developed at the upstream and downstream of the dam section against allowable stress for the different loading conditions. The required factors of safety for the stability analysis are included in Table 2 below while the results of the analysis for the full supply level of 592.2 m option is presented in Table 3.

Table 2: Required Factors of Safety for Stability Analysis[1].

Loading Case	Sliding		Resultant Location, % of Base in Compression (Overturning)	Concrete Stress	
	Peak	Residual		Compressive	Tensile
Usual – Static	3.0	1.5	Middle 1/3, 100 %	0.3 f'_c	0
Unusual – IDF Loading	2.0	1.3	Middle ½, 75 %	0.5 f'_c	0.6 $f'^{2/3}_c$
Extreme – MDE (pseudo-static)	1.3	1.0	Within Base, N/A	0.9 f'_c	1.5 $f'^{2/3}_c$

1. Required factors of safety based on New Zealand Society on Large Dams (NZSOLD) guidelines and resultant location, % of base in compression, and concrete stresses are based on guidelines from the US Army Corps of Engineers (1995).

Table 3: Results of Stability Analysis – Full Supply Level 592.2 m.

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	5.9	2.4	OK	OK	OK
Unusual – IDF Loading	5.4	2.2	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.1[2]	0.4	FAIL	OK	FAIL[4]
Extreme – MDE Sustained Ground Accel. (pseudo-static)	1.8	0.7[3]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. Only applies to base to el. 535, FS above 1.3 above el. 535.
3. If peak factors of safety are met, materials not expected to reduce to residual strengths.
4. Only the base joint failed.

The dam meets the minimum required factors of safety under usual and unusual loading conditions but the factors of safety are not met under the extreme loading events. Even though the factors of safety were not met for all loading conditions, it does not mean the dam will fail catastrophically but rather that there may be some movement or cracking along lift lines. This simplified pseudo-static analysis is a screening tool that indicates a more rigorous dynamic analysis will be required in the next phase of design to verify stability. A better understanding of the amount and direction of movement will be required to estimate the response of the dam during the maximum design earthquake.

More rigorous and detailed analysis should be completed during final design. Additional design features, such as bolting, shear keys, anchors, increase footprint size, sloping of the upstream face, or adding a curvature to the dam alignment may be required to improve seismic performance. A deformation analysis will likely be required during final design. Further refinement of the dam design and geometry should be expected as a result of these more rigorous analyses and will likely impact construction costs.

4.0 PRELIMINARY DESIGN

The followings sections summarise the preliminary design for the new RCC dam option with a full supply level of 592.2 m. Preliminary design drawings are included in Appendix F.

4.1 Configuration

The proposed RCC dam layout consists of a 71.2 m high RCC gravity dam, with a crest length of 212 m, a crest width of 8 m and maximum crest elevation of 596.6 m. The upstream slope is vertical with 4 m tall vertical portion transitioning to an overall 1H:1V downstream slope. This relatively conservative dam cross section is due to the high earthquake design requirements and it is anticipated that steepening of the downstream slope may be justified by further stability modelling during detailed design. The new dam alignment is located just downstream of the existing powerhouse and substation as this allows for additional construction space between the existing dam and the new dam and also allows for a more complete grout curtain as it avoids the tunnels in the foundation of the existing dam. The more perpendicular alignment of the dam to the valley also provides buttressing of the dam, potentially improving stability.

The full supply level of 592.2 m allows for 4 m of freeboard which is adequate to accommodate wave run up and setup during normal operations and during the design storm event. The dam crest contains a 35 m wide free overflow spillway located near the centre of the embankment with an overflow spillway crest elevation of 592.2 m. Flows during the design flood event are estimated to be 3 m in depth resulting in one meter of freeboard. The stepped spillway chute discharges into an energy dissipator which empties into the Manuherikia River downstream. The offtake structure consists of an intake tower with three intake elevations which are gated and protected by a trashrack. A single gated 2 m diameter conduit running through the dam at an invert elevation of 545 m releases irrigation, flushing, and emergency drawdown flows downstream. The conduit discharges into the new powerhouse along the right side of the embankment. Release valves before and after the new powerhouse allow for offtake flows to enter Manuherikia River.

A saddle dam is required in Shamrock Gully to contain the full supply reservoir pool at elevation 592.2 m. The saddle dam will have a maximum height of approximately 5 m, a crest elevation of 596.6 m, and a crest width of 5 m to allow vehicle traffic. The saddle dam will mostly act as a freeboard structure and will be constructed with a low permeability core with a chimney drain extending up to the full supply level. The upstream slope will consist of a layer of riprap to reduce wave erosion. Previous field explorations encountered sandy gravels to silty clays along the saddle dam alignment and shallow excavation of foundation soils and cutoff trench are anticipated (Golder 2014a). However, a void, filled with water, was encountered at the bottom of one of the test pits along the saddle dam alignment. The test pit was backfilled after logging was completed but settling of the backfill material of up to nearly 1 m was observed approximately 6 months after the test pit was backfilled. This may indicate there is potentially a larger void, cavity or channel in this area which will have to be addressed in final design.

Further field investigations and mapping efforts will be required to estimate the extent and severity of this feature and the results may impact the design and cost estimate of the saddle dam.

4.2 RCC Design

The preliminary design includes 1 m thick cast-in-place conventional concrete facing along the upstream and downstream faces of the dam to prevent freeze-thaw issues and to provide a durable surface. The concrete will likely be placed simultaneously with the RCC and care will need to be taken to ensure the interface between the concrete and RCC is thoroughly consolidated and mixed.

Bedding mortar or grout will be placed over the full lift surface to increase the bond strength between lifts and to increase water tightness. The bedding mortar will be a high-slump, high-cement content material and should be placed immediately before the next layer of RCC is placed.

A RCC mix design has not been created at this stage in the design process but the final mix design will be based on available materials, laboratory testing results of the available materials, chemistry of the reservoir water, required RCC strengths to withstand seismic loads, and results from the test section. The mix design used in the cost estimate is based on a previously completed RCC dam design that was subject to similar earthquake loadings.

4.3 Seepage Considerations

Controlling seepage is an important design consideration for an RCC dam. Seepage pathways may exist through RCC lifts or through cracks resulting from thermal volume changes or foundation irregularities (USACE 2000). Careful consideration will have to be given to the final mix design and proportions to reduce cracking and seepage. Water stops will be supplied at all control joints and crack inducer joints. For the preliminary design, it is assumed that water stops will be placed at 6 m on centre.

The RCC dam will also have an internal drainage system allowing for collection of seepage through the embankment and foundation and to help to reduce uplift pressures. For the preliminary design, the drainage gallery is assumed to be located at elevation 545 m at the maximum section of the dam and offset from the upstream face by 8 m. The gallery elevation will increase at the abutments to account for the shorter dam height. The gallery will collect seepage from the face drains which will consist of vertical drill holes located near the upstream face of the dam. Joint drains may also be installed downstream of the water stop joints to collect any additional seepage. Foundation drains will collect foundation seepage and will be located downstream from the grout curtain.

4.4 Grout Curtain

Seepage protection measures include a grout curtain located upstream of the centreline of the embankment. The preliminary design includes two rows of grout holes terminated at a depth equal to the maximum reservoir head at each location. One row of the grout holes will be vertical and the second row of the grout holes will be angled to intercept more vertical joints. Secondary or tertiary angle holes will likely be required at highly fractured zones or areas of high grout take. Grouting completed during the construction of the existing Falls Dam indicated that grout takes were relatively low with a few zones of high permeability (Gilkison 1937). Future field explorations will better define the extent of the grout curtain.

4.5 Offtake Structure

The intake tower has three intake levels with a low-level intake at elevation 545 m. The low-level intake is situated to accommodate a single dam penetration with a discharge to a powerhouse near a natural bench at the right abutment. The low-level elevation allows for a minimum reservoir volume for environmental

purposes by leaving a 20 m deep dead pool; dam safety implications of this dead storage should be further evaluated or means to drawdown the pool level should be incorporated into the design.
The intermediate and high intake levels will be determined during final design and after a water-quality study, if necessary. Each opening will have a trashrack and a control gate, with operation of all gates from the control house at the top of the intake tower. A guard gate is located at the bottom of the intake structure on the 2 m diameter offtake conduit that conveys flow through the dam. This offtake conduit size allows for manageable flow velocities during flushing operations as well as the large flows required to draw down the reservoir within guideline timeframes (approximately 30 days). Downstream of the dam, the offtake conduit has anticipated connections to the powerhouse with a discharge into the irrigation race and direct discharge to the river for flushing and drawdown flows. The control house on the dam crest will contain the electrical and control equipment along with backup generators for operation of the gates. Real-time monitoring and control of the gates will be incorporated during final design.

4.6 Overflow Spillway

The preliminary design includes a stepped 35 m wide, uncontrolled spillway centred within the non-overflow RCC section. Flow depth through the spillway during the 1 in 10,000 year IDF is approximately 3 m, leaving 1 m of freeboard to the dam crest. The spillway configuration is rectangular and is planned to include an ogee crest to improve the hydraulic efficiency. Flow down the spillway will enter a USBR Type II stilling basin energy dissipator at the base of the dam, which uses baffle blocks and an end sill to force the hydraulic jump to occur within the dissipator structure. The energy dissipater is the same width as the spillway (35 m), 30 m long, and 7 m deep with a flat concrete floor and vertical concrete walls. The 7 m depth is intended to contain the hydraulic jump before releasing the water into the receiving river. The energy dissipater releases flow into the natural river channel through a channel transition zone, which includes riprap to facilitate the transition. A USBR Type VII flip-bucket energy dissipator may be an alternative to the Type II stilling basin which could be considered during final design. A tailwater analysis will also be required as part of final design.

The catchment upstream of the reservoir does not appear to be heavily forested so large logs and debris are not expected to impact spillway operations during a flood event. However, a log boom may be a prudent design feature considered during final design.

4.7 Instrumentation

A detailed instrumentation plan was not designed as part of this preliminary design but costs for installation of vibrating wire piezometers, v-notch seepage measuring weirs, structural monitoring points, and early warning system has been included in the cost estimate. Remote monitoring of the instrumentation at the site will also be recommended. A robust instrumentation and monitoring program will be required to adequately monitor the dam and to identify any potential deficiencies.

5.0 CONSTRUCTION METHODOLOGY

The following sections summarise the construction methodology for the new RCC dam option with a full supply level of 592.2 m.

5.1 Reservoir Restrictions during Construction

The existing reservoir is expected to stay relatively full during construction while also meeting irrigation and flushing flow requirements. However, the dam and stream diversion will have to accommodate the construction design flood event. The reservoir may need to be lowered to contain some of the construction design flood as it is anticipated that the offtake structure will not be able to pass the entire flood.

The existing dam is anticipated to be breached once the new RCC dam is constructed to address operational and water quality issues associated with the small volume of the reservoir isolated between the existing embankment and the new downstream embankment. The existing reservoir will also have to be low during breaching of the existing dam to minimise impacts to the new RCC structure. Breaching of the existing dam can occur at a time when the reservoir is typically low, such as at the end of the irrigation season.

5.2 Access and Haul Roads and Quarry

Staging areas are anticipated to be located on the ridge above the right abutment. There is an existing access road from State Highway 85 to the existing dam. This road is anticipated to need widening and an 80 m long permanent bridge is anticipated to be built over the Manuherikia River to allow access to the right side of the valley downstream from the new dam. Access roads will be constructed from the proposed dam location up to the right abutment staging areas and back down to the dam footprint area. The roads will create a loop to accommodate one-way on-site traffic.

A new quarry will likely be developed for this project. The quarry will provide aggregate and sand for the project while including space for stockpiled material and processing plants which will screen the aggregate to meet construction specifications. If sand or other imported materials are required to meet the specified material gradations, the mixing will likely be completed in the quarry area as space will be limited at other locations. A suitable quarry will have sufficient material that is easy to access and excavate while also being located close enough to production areas to reduce haul lengths. For the preliminary design, the quarry is anticipated to be located above the right abutment. Location of a suitable quarry will be optimized in final design.

5.3 Staging and Production Areas

Level areas will be required for equipment staging, maintenance areas and the laboratory. These areas will need to be located close to the production and batching plant. The area required for the RCC production will have to accommodate the RCC plant, aggregate stockpiles, cement (and flyash) silos, feeding systems, material delivery area, and a material loading area. The laboratory will need to be located in an enclosed building. For the preliminary design, the staging and production areas are anticipated to be located above the right abutment.

5.4 Construction Materials

The construction materials required for RCC include aggregate and sand, cement, fly ash (if possible), mix water, and admixtures. The majority of these materials will be imported to the site during construction with the exception being the aggregate and sand which will be quarried and processed on site. Additional materials will be required for the saddle dam which includes low permeability soil, rockfill, sand, and riprap. These materials are anticipated to be supplied by on-site borrow areas. The sources of each material require evaluation and testing to ensure quality materials are available for construction of the project. The construction materials will likely be stockpiled near the production plant prior to the start of construction.

5.5 RCC Placement

Prior to RCC placement, the foundation will be prepared to provide a smooth surface. All cavities, voids, surface irregularities, and places where RCC cannot be compacted will be filled with dental concrete. Any overburden or rock material found to be unsuitable as foundation material will be removed during construction. Conventional concrete foundation bedding will be placed between the RCC and the foundation bedrock.

RCC placement is constructed from the bottom lift up and will likely be placed by trucks, spread by a dozer and compacted by a vibratory smooth-drum roller. A rotating beam laser will be used to control the lift thickness to allow for a compacted 0.3 m thick layer. A double-drum, self-propelled vibratory roller that requires 4 to 6 passes to meet compaction densities and smaller equipment including walk-behind rollers and manual compaction equipment (in smaller or tighter areas where the vibratory roller cannot access) will likely be used. The performance of the drum rollers, small compaction equipment and the number of prescribed passes will be determined during construction of the RCC test section. Adequate bonding between RCC lifts will likely require compaction of the next lift within 15 minutes of spreading and within 45 minutes of production. Due to the relatively large size of each lift and the bonding strength required between each lift, bedding or grout mortar will be required between each lift. Use of other placement techniques can be assessed and tested during final design and the test section, respectively.

As conveyor equipment required for construction of an RCC dam of this magnitude is difficult to obtain, it is likely that an all truck placement system will be used. Trucks will leave the RCC plant area and drive along the haul roads to the RCC placement area access ramp. The trucks will enter the placement area in reverse and dump the RCC material at the point of placements on the lift. The truck will then return along the same route moving forward off the RCC placement area at the access ramp, allowing the next truck to enter and exit the lift in the same manner. The dozer(s) will spread the RCC with the vibratory compactor(s) following close behind. Depending on the lift size, it may be possible to have multiple dozers and compactors on each lift but only one of each will fit on the smaller lifts near the top of the embankment. Once the lift is compacted and cured to the point where it can support traffic on the lift surface, the top of the lift is cleaned using brooms, water, air, and vacuum. Then grout or mortar bedding is placed onto the lift, likely by concrete trucks and manual labour spreading the mortar with brooms. After placement of the mortar, the RCC placement process repeats for each succeeding step until all the RCC is placed.

6.0 PRELIMINARY LEVEL DESIGN COST ESTIMATES

Typically, preliminary designs are based on a partially optimized design from the limited field explorations, project information, and technical analyses. Further optimization is completed at the later detailed design stage. Estimated construction costs are based on the preliminary design which will likely change during detailed design and any design changes will impact the construction cost estimates. The cost estimates will also be sensitive to future escalation of key cost components such as labour rates, fuel prices, and material prices.

Fish passage has not been included in the preliminary design or cost estimate but its need should be evaluated as part of the final design. Cost to develop documents and programs such as emergency action plans (EAP), operation and maintenance (O&M) manual, dam safety assurance plans, and an inspection program are included under the engineering and design line item.

The preliminary cost estimate for the RCC dam option with a full supply level of 592.2 m is presented in Table 4. Estimates of the cost for construction management, engineering and design, consenting, bonds and insurance, and a contingency have also been included as separate line items. The detailed cost estimate is presented in Appendix D.

Table 4: Falls Dam Cost Estimates – Full Supply Level 592.2 m.

Item	Description	Cost Estimate*
Site Establishment	Includes items such as site access and setup, quarry establishment, power supply, and demolition of existing dam and powerhouse.	$13,870,000
Foundation Treatment	Includes items such as foundation rock excavation, backfill / dental concrete and grout curtain.	$3,390,000
RCC and Spillway	Includes items such as producing and placing RCC and concrete for overtopping spillway, instrumentation, and drainage features.	$94,920,000
Offtake Structures	Includes items such as of concrete for intake tower, gates and control for gates.	$3,080,000
Saddle Dam	Includes items such as saddle dam foundation excavation and embankment placement.	$1,010,000
Base Construction Cost (BCS)		**$116,270,000**
Construction Management	7 % of BCS	$8,140,000
Engineering and Design	10 % of BCS	$11,630,000
Bonds and Insurance	5 % of BCS	$5,820,000
Consenting	2 % of BCS	$2,330,000
Direct Construction Cost (DCS)		**$144,190,000**
Uncosted Items	35 % of DCS	$50,470,000
Total Estimated Preliminary Project Costs		**$194,660,000**
*Costs are rounded up to the nearest $10,000 and exclude GST.		

7.0 ADDITIONAL DAM HEIGHTS

Preliminary designs for two additional RCC dam options with full supply levels of 580.4 m and 570.6 m were also prepared. The same design criteria and methodology used for the full supply level 592.2 m dam option described above were used for the two additional dam height options. The preliminary designs and unit rates from the full supply level 592.2 m option were used to estimate costs for the two additional dam heights. The preliminary designs and cost estimates are described below. Preliminary design drawings are included in Appendix F.

7.1 Preliminary Design for Additional Dam Height Options

The dam options with full supply levels of 580.4 m and 570.6 m are both anticipated to be high PIC dams. The dam breach for these smaller dam configurations will result in smaller flood releases but the economic damage is still anticipated to be major and there is the possibility for loss of life due to the relatively close proximity of the dam to road ways, residences, and the bike trail. The flood flows downstream of the dam are still expected to be fast moving and deep as far downstream as Blackstone where as many as 80 people would be at risk from the flood waters. Future dam break modelling may result in reducing the dam classification to a medium PIC but it is still expected to be on the high end of the medium PIC spectrum which would result in similar design criteria as a high PIC dam. As a result, the same hydrologic and seismic events as used to design the preferred alternative are used in the dam options with full supply levels 580.4 m and 570.6 m. Again it should be stressed that there are many high PIC dams in New Zealand and the classification does not indicate the likelihood of failure but rather identifies the design criteria. The High PIC classification for the new Falls Dam indicates that the highest design requirements are necessary.

The centreline alignment of the two additional dam options is the same as the preferred option which results in same foundation conditions for all three dams. The same foundation preparation as expected for the full supply level 592.2 m option is expected for the 580.4 m and 570.6 m options.

A stability analysis was performed for both additional dam height options. Detailed discussion of the stability analysis is presented in Appendix C. The results of the stability analysis for the full supply levels of 580.4 m and 570.6 m are presented in Table 5 and Table 6, below.

Table 5 : Results of Stability Analysis – Full Supply Level 580.4 m

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	7.1	2.6	OK	OK	OK
Unusual – IDF Loading	6.4	2.3	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.3	0.5[2]	FAIL[3]	OK	OK
Extreme – MDE Sustained Ground Accel. (pseudo-static)	2.1	0.7[2]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. If peak factors of safety are met, materials not expected to reduce to residual strengths.
3. The resultants for most lift layers are acceptable; the resultant is only outside of the base below El. 535m.

Table 6: Results of Stability Analysis – Full Supply Level 570.6 m.

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	7.7	2.6	OK	OK	OK
Unusual – IDF Loading	6.8	2.3	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.4	0.5[2]	FAIL[3]	OK	OK
Extreme – MDE Sustained Ground Accel. (pseudo-static)	2.3	0.8[2]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. If peak factors of safety are met, materials not expected to reduce to residual strengths.
3. The resultants for most lift layers are acceptable; the resultant is only outside of the base below El. 535m.

As with the full supply level 592.2 m option, the dams meet the minimum required factors of safety under usual and unusual loading conditions and these lower dam height options also meet the minimum required factors of safety for the extreme events with peak strengths. The factors of safety were not met under the extreme event with residual strengths but as the peak factors of safety are met, it is not anticipated that the materials will reduce to residual strengths.

Even though the factors of safety were not met for all loading conditions, it does not mean the dam will fail catastrophically but rather that there may be some movement or cracking along lift lines. This simplified pseudo-static analysis is a screening tool that indicates a more rigorous dynamic analysis will be required in the next phase of design to verify stability. A better understanding of the amount and direction of movement will be required to estimate the response of the dam during the maximum design earthquake.

More rigorous and detailed analysis will be required during final design. Additional design features, such as bolting, shear keys, anchors, increase footprint size, sloping of the upstream face, or adding a curvature to the dam alignment may be required to improve seismic performance. A deformation analysis will likely be required during final design. Further refinement of the dam design and geometry should be expected as a result of these more rigorous analyses.

The RCC dam layout for the full supply level of 580.4 m option consists of a 59.4 m high RCC dam, with a crest length of 175 m, a crest width of 8 m and maximum crest elevation of 584.4 m. The RCC dam layout for the full supply level of 570.6 m option consists of a 49.6 m high RCC dam, with a crest length of 150 m, a crest width of 8 m and maximum crest elevation of 574.6 m. The upstream slope for both options is vertical with a 4 m tall vertical section transitioning to an overall 1H:1V downstream slope. Both dams allow for 4 m of freeboard which is adequate to accommodate wave run up and setup during the normal operations and during the design storm event. The dam crest contains a 42 m wide free overflow spillway for the full supply level 580.4 m option and a 50 m wide free overflow spillway for the full supply level 570.6 m option. The spillways are both located near the centre of the embankment and flows during the design flood event are anticipated to be 3 m in depth resulting in one meter of freeboard during the design storm event. The stepped spillway chute discharges into an energy dissipator which empties into the Manuherikia River downstream. The spillway widths are different for the different height options (42 m for the full supply level of 580.4 m option and 50 m for the full supply level of 570.6 m option) because as the dam height increases the reservoir area above normal pool increases providing more storage in the reservoir to attenuate the storm event flow. For both dam options, the offtake structure consists of an intake tower with three intake elevations which are gated and protected by a trashrack. A single gated conduit through the dam at an invert elevation of 545 m provides for irrigation, flushing, and emergency drawdown flows downstream. The conduit discharges into the powerhouse along the right side of the embankment. Release valves before and after the powerhouse allow for offtake flows to enter Manuherikia River. The offtake conduit is consistent at a diameter of 2 m for all options because the flushing flow was determined to control the sizing of the conduit (and the flushing flow of 12 cumecs is the same for all options). Similar to the full supply level of 592.2 m option, the 2 m diameter offtake conduit allows for drawdown timeframes within USBR (1984) guidelines as well as connections for irrigation and power generation discharges.

The RCC dams, for both the full supply level options of 580.4 m and 570.6 m, will have 1 m thick RCC facing, water stops, grout curtain, drainage galleries, and instrumentation as described above for the full supply level option 592.2 m. The quantity and extent of each of these features are scaled down from the full supply level 592.2 m option. The construction methodology for the full supply level options of 580.4 m and 570.6 m is the same as described above for the full supply level 592.2 m option.

7.2 Preliminary Level Design Cost Estimates for Additional Dam Height Options

Typically, preliminary designs are based on a partially optimized design from the limited field explorations, project information, and technical analyses. Further optimization is completed at the later detailed design stage. Estimated construction costs are based on the preliminary design which will likely change during detailed design and any design changes will impact the construction cost estimates. The cost estimates will also be sensitive to future escalation of key cost components such as labour rates, fuel prices, and material prices.

Fish passage has not been included in the preliminary design or cost estimate but its need should be evaluated as part of the final design. Cost to develop documents and programs such as emergency action plans (EAP), operation and maintenance (O&M) manual, dam safety assurance plans, and an inspection program are included under the engineering and design line item. The preliminary cost estimate for the RCC dam option with a full supply level of 580.4 m is presented in Table 7 and in Table 8 for the full supply level 570.6 m option. Estimates of the cost for construction management, engineering and design, consenting, bonds and insurance, and a contingency have also been included as separate line items. The detailed cost estimate is presented in Appendix E. The unit rates estimated for the full supply level 592.2 m option were used and applied to these lower dam height options.

Table 7: Falls Dam Cost Estimates – Full Supply Level 580.4 m.

Item	Description	Cost Estimate*
Site Establishment	Includes items such as site access and setup, quarry establishment, power supply, and demolition of existing dam and powerhouse.	$11,470,000
Foundation Treatment	Includes items such as foundation rock excavation, backfill / dental concrete and grout curtain.	$2,470,000
RCC and Spillway	Includes items such as producing and placing RCC and concrete for overtopping spillway, instrumentation, and drainage features.	$68,470,000
Offtake Structures	Includes items such as of concrete for intake tower, gates and control for gates.	$2,470,000
Base Construction Cost (BCS)		**$84,880,000**
Construction Management	7 % of BCS	$5,940,000
Engineering and Design	10 % of BCS	$8,490,000
Bonds and Insurance	5 % of BCS	$4,250,000
Consenting	2 % of BCS	$1,700,000
Direct Construction Cost (DCS)		**$105,260,000**
Uncosted Items	35 % of DCS	$36,840,000
Total Estimated Preliminary Project Costs		**$142,100,000**
*Costs are rounded up to the nearest $10,000 and exclude GST.		

Table 8: Falls Dam Cost Estimates – Full Supply Level 570.6 m.

Item	Description	Cost Estimate*
Site Establishment	Includes items such as site access and setup, quarry establishment, power supply, and demolition of existing dam and powerhouse.	$10,270,000
Foundation Treatment	Includes items such as foundation rock excavation, backfill / dental concrete and grout curtain.	$2,330,000
RCC and Spillway	Includes items such as producing and placing RCC and concrete for overtopping spillway, instrumentation, and drainage features.	$47,270,000
Offtake Structures	Includes items such as of concrete for intake tower, gates and control for gates.	$2,280,000
Base Construction Cost (BCS)		**$62,150,000**
Construction Management	7 % of BCS	$4,350,000
Engineering and Design	10 % of BCS	$6,220,000
Bonds and Insurance	5 % of BCS	$3,110,000
Consenting	2 % of BCS	$1,240,000
Direct Construction Cost (DCS)		**$77,070,000**
Uncosted	35 % of DCS	$26,980,000
Total Estimated Preliminary Project Costs		**$104,050,000**
*Costs are rounded up to the nearest $10,000 and exclude GST.		

8.0 DESIGN OPTIMISATION

The three dam options presented in this report do not represent the only water storage options at the site but provide indicative construction costs and design solutions meeting current standards and guidelines for the selected heights at the selected locations. Adjustments to the height, alignment, appurtenant structures, configuration and dam type will impact construction cost and an optimised solution will likely provide the most cost effective option.

In determining the most cost effective option, an understanding of how design changes impact construction costs should be evaluated. There are some relatively fixed costs associated with building a dam at the site that are independent of the dam configuration, including site preparation, bridge and road construction, demolition of the existing dam and upgrades to existing offtake structure/spillway for use as stream diversion during construction. Other costs are directly related to the dam size and location. Reducing the required storage volume, effectively lowering the dam height, will not only decrease dam volume but will eliminate the need for a saddle dam while also decreasing the size of the quarry, the grouting depths, construction duration and required instrumentation.

When optimising the dam alignment and height, more than dam volume must be considered. The prefeasibility level study proposed a dam alignment located closer to the toe of the existing dam which may provide for a lesser dam volume but cost increases due to increased foundation preparation and treatment, excavation, grouting, and reduced construction access are anticipated. As the dam height decreases to less than an 8 to 10 meter raise, it may be more cost effective to raise the existing dam rather than construct a new dam. However this option would likely require draining the reservoir for at least an irrigation season and there is increased risk and uncertainty associated with this option, as discussed in Golder's "Manuherikia: Falls Dam Recommended Option" report (Golder 2014b).

A detailed risk assessment may also be beneficial in future design stages as potential risks associated static, seismic and hydrologic loadings may be better understood, resulting in more focused design efforts. There are also additional design and construction features that have not been discussed in detail during this feasibility level design that could potentially have a large impact on the total cost including RCC mix design, seismic loadings, deformation analysis and the inflow design flood. An optimised dam height and location are currently being assessed as part of ongoing work.

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APPENDIX A

Report Limitations

Report Limitations

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APPENDIX B

Dam Break Assessment

July 2014

REPORT

MANUHERIKIA CATCHMENT WATER STRATEGY GROUP

Dam Break Assessment - raised Falls Dam, full supply level of 588 m

Submitted to:
Manuherikia Catchment Water Strategy Group

Report: 1378110270_2000_214_R_Rev0_219

Golder Associates

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List of Abbreviations

AADT	annual average daily traffic
amsl	above mean sea level
Aqualinc	Aqualinc Research Limited
CFRD	concrete faced rockfill dam
DEM	digital elevation model
FEMA	Federal Emergency Management Agency
FSL	full supply level
GIS	geographic information systems
MCWSG	Manuherikia Catchment Water Strategy Group
NZSOLD	New Zealand Society on Large Dams
OCRT	Otago Central Rail Trail
PAR	population at risk
PIC	potential impact category
RCC	roller compacted concrete
SH85	State Highway 85
XPSWMM	XP stormwater and wastewater management tool

1.0 INTRODUCTION

1.1 Overview

Golder Associates (NZ) Limited has been commissioned by the Manuherikia Catchment Water Strategy Group (MCWSG) to provide a feasibility level assessment of irrigation options in the Manuherikia catchment. Opus (2013) prepared an engineering prefeasibility study on options for raising Falls Dam. However, that study did not include a dam break assessment and recommended that such an assessment be completed as part of the feasibility study.

This report describes the findings of a dam break assessment of a roller compacted concrete dam with a full supply level (FSL) of 588 m above mean sea level (amsl) constructed immediately downstream of the existing dam. This assessment considers the effects that a dam breach may have on downstream areas and identifies a potential impact category for the dam. This assessment forms part of wider feasibility level investigations.

1.2 Objectives

The purpose of this study is to inform the wider feasibility level assessments of the implications of a dam break of the Falls Dam. In particular this assessment will:

1) Determine the Potential Impact Category (PIC) of the dam; this will influence the dam design parameters.

2) Assess the potential flooding hazard and risk in the event of a dam break, which is required during resource consenting of any dam.

1.3 Location

Falls Dam is located on the upper reaches of the Manuherikia River, approximately 60 km upstream of Alexandra, in Central Otago (Figure 1). The dam provides storage for four existing irrigation schemes (Blackstone, part of Omakau, Manuherikia and part of Galloway) which cover approximately 6,500 ha in the Manuherikia Valley. The Manuherikia River flows past several small townships to Alexandra, where it converges with the larger Clutha River.

Falls Dam is an existing concrete faced rockfill dam (CFRD) approximately 33.5 m high, with a FSL of 561.4 m amsl. The current feasibility study is evaluating increased storage options up to a FSL of 588 m amsl.

1.4 Report Limitations

Your attention is drawn to the document, “Report Limitations”, as attached in Appendix A. The statements presented in that document are intended to advise you of what your realistic expectations of this report should be, and to present you with recommendations on how to minimise the risks to which this report relates which are associated with this project. The document is not intended to exclude or otherwise limit the obligations necessarily imposed by law on Golder Associates (NZ) Limited, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

2.0 DAM BREAK INPUTS

2.1 Methodology

A dam break assessment simulates the release of stored water behind a dam over a specified failure time. Although the risk of failure of a suitably designed dam is very small, the New Zealand Dam Safety Guidelines (NZSOLD 2000) requires dams to be categorised according to their consequences of failure. These potential consequences include; loss of life, socio-economic, financial and environmental damage.

The general methodology for a dam break assessment involves:

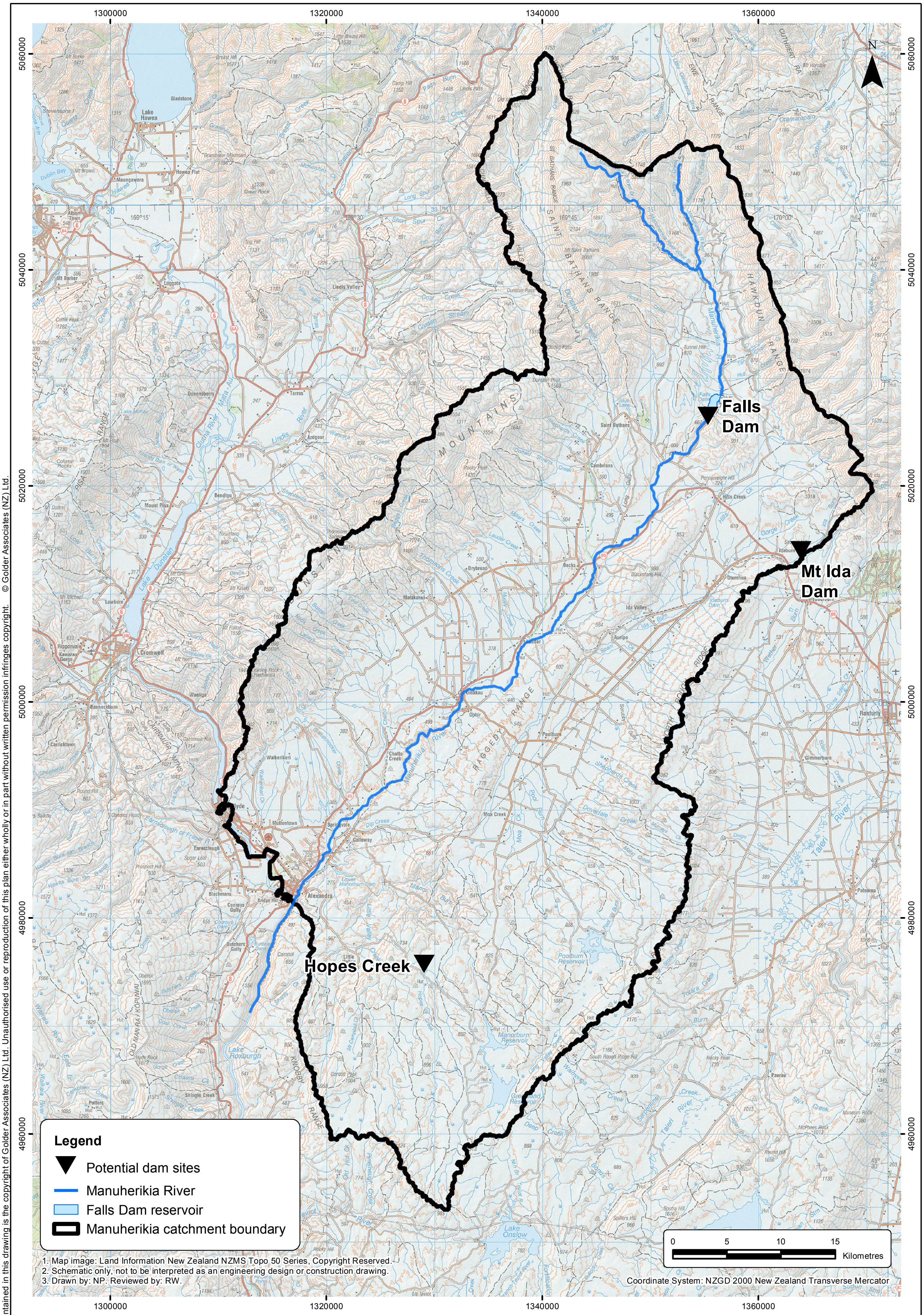
1) Determination of dam breach parameters.

2) Determination of breach discharge hydrograph.

3) Evaluation of the timing and extent of the flood wave.

4) Identification of the Potential Impact Category (PIC).

PIC classification is an important stage in dam design and evaluation because a number of the dam design criteria are dictated by the PIC.



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TITLE MANUHERIKIA CATCHMENT

JULY 2014

PROJECT 1378110270

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2.2 Breach Conditions

It is normal practice to undertake two dam failure scenarios; 'sunny day' and 'rainy day'. The 'sunny day' scenario simulates a structural failure (i.e., earthquake, piping, etc.) under normal flow conditions, and the 'rainy day' scenario assumes that dam breach occurs during a flood event.

This evaluation considers a new Falls Dam constructed from roller compacted concrete (RCC). Concrete dam failures are typically modelled as structural failures (FEMA 2013). This construction type is very unlikely to fail due to overtopping as RCC is designed to overtop during flood events. Therefore, the 'sunny day' failure scenario is the most critical and the only scenario to be modelled in this assessment.

2.3 Breach Parameters

Dam breach parameters include the parameters needed to physically describe the breach (breach depth and width) as well as parameters that define the time required for breach initiation and failure. Time to failure plays a significant role in the determination of peak outflow from the dam breach. FEMA (2013) recommends a range of failure times for concrete dams from 6 to 30 minutes. An average of 15 minutes is applied in this model. A shorter time to failure gives the highest peak flows while the longest time to failure gives the lowest peak flows.

Table 1: Falls Dam FSL of 588 m amsl breach parameters.

Parameter	Inputs
Construction materials	Roller compacted concrete
Impounded volume[A]	100 Mm^3
Crest length[A]	195 m
Breach width[B]	98 m
Dam height[C]	61 m
Breach depth[D]	61 m
Time to failure[E]	6 to 30 minutes

Notes: [A] Parameters derived from Opus (2013) report; [B] FEMA (2013) suggests an average breach width equal to half the entire length of the dam; [C] Based on a dam base level of 532 m amsl, a FSL of 588 m amsl and a 5 m freeboard allowance; [D] The bottom of the breach should generally be assumed to be at the foundation level of the dam; [E] FEMA (2013) suggests a range of failure times for concrete dams from 6 to 30 minutes.

2.4 Breach Discharge Hydrograph

To predict peak flow and the dam breach hydrograph, various methods can be applied including: a triangular hydrograph, level-pool routing, dynamic wave simulation, regression relationships, and comparative analysis to similar dams that have failed. All methods have shortcomings such as lack of data, lack of case studies and poor understanding of breach mechanics.

The breach discharge hydrograph was developed using the triangular hydrograph method. This is based on the dam impounded volume and time to failure. In this method, it was assumed that it would take 15 minutes from the start of the breach to the full extent of the breach to occur (time to failure), and the entire volume of the dam will be discharged in 30 minutes. Therefore, the area under the dam breach hydrograph is equal to the reservoir volume during the 'sunny day' event. The peak discharge in a 'sunny day' dam break scenario is estimated at 111,100 m^3/s (Figure 2).

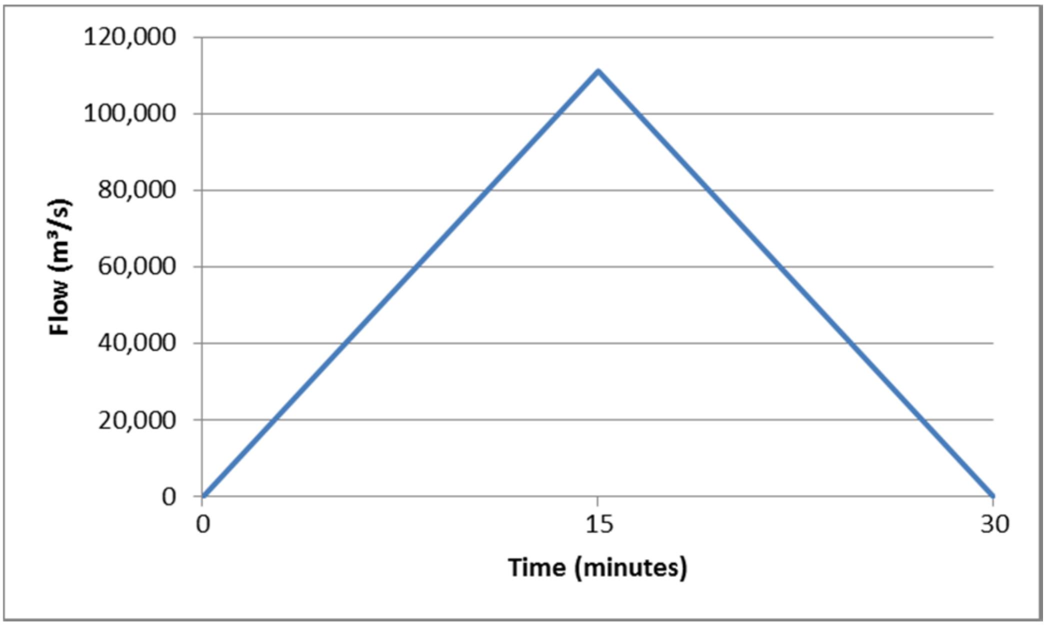


Figure 2: Dam breach hydrograph (Triangular method).

2.5 Hydrology

Flood routing of the dam breach discharge through the catchment requires an understanding of the catchment hydrology. The hydrology for the catchment has been assessed by Aqualinc Research Limited and is documented in two main reports (Aqualinc 2012a and 2012b). Aqualinc (2012a) states the following:

The Manuherikia River has a mean naturalised flow at the Clutha River confluence of 18.5 m^3/s. The major tributaries of the Manuherikia River are the Manuherikia above Falls Dam, the Pool Burn, Dunstan Creek, Manor Burn, Lauder Creek, Thomsons Creek and Chatto Creek. Collectively these tributaries provide almost 90% of the total catchment flow. Aqualinc (2012a) Page 4.

Mean naturalised flow in the seven main tributaries ranges from a high of 4.8 m^3/s for the Manuherikia above Falls Dam to a low of 0.7 m^3/s for Chatto Creek (Aqualinc 2012a). These tributary flows are very small (four to five orders of magnitude smaller) compared to the expected dam breach flows. Therefore flood routing of the dam breach discharge through the catchment will be largely insensitive to tributary inflows. To improve the runtime efficiencies of the flood routing model, tributary inflows were excluded from the model.

2.6 Flood routing

XPSWMM 2013, a hydraulic and hydrological modelling tool is used to route the flood wave downstream. XPSWMM uses the TUFLOW computational engine that links 1-D and 2-D modelling to simulate flood propagation.

The following are components of the hydraulic model:

- Digital Elevation Model (DEM) – developed by geographic information systems (GIS) based on 20 m contour data (LINZ 2014) combined with 5 m contour data (MWD 1976) around the Manuherikia River channel.
- Model extents – The Manuherikia River main stem is modelled from Falls Dam to the Ophir gorge.

- Downstream boundary condition – The downstream boundary condition is set to a shallow depth, forcing a critical depth to occur at the downstream end.
- Dam break hydrograph – The generated breach hydrograph is incorporated into the XPSWMM model as a flow boundary condition at the Falls Dam site.
- Tributary inflows – Flows from tributaries were not included in this model.
- Model nodes – A number of nodes are positioned throughout the model at the locations of infrastructure and towns. These nodes enable the modelled water depth, flow and velocity to be easily reviewed at these points of interest.
- Manning's roughness – The Manning's value can be expected to change throughout the reach of the Manuherikia River and its tributaries. However, for simplicity, a fixed value has been used. A Manning's value of 0.04 was selected as this is considered reasonable for a gravel-cobble river channel and surrounding pasture floodplains.
- 2D grid resolution – Grid size of 30 m was used.
- Time step – A time step of 0.5 seconds was used in the XPSWMM model.

For this dam break analysis, the downstream boundary of the XPSWMM model is the Ophir Gorge. An extended model to Alexandra was preferable, but a compromise exists between model extent and detail. In order to provide sufficient detail and accuracy to the model, the extent was limited to the reach of the Manuherikia River from Falls Dam to Ophir Gorge.

There are several hydraulic structures within the watercourse downstream of the dam. These include road and pedestrian bridges, irrigation siphons and intake structures. Due to the scale of the overall system model, these structures are largely ignored in terms of their impact on flood routing. Hydraulic structures are assessed in terms of their potential for damage due to inundation.

In any dam break model, calibration is difficult as peak flows of this magnitude rarely occur. Furthermore, the critical factor in the model is the time to failure and as such, errors associated with cross-sections, hydraulic structures and calibration will be less significant.

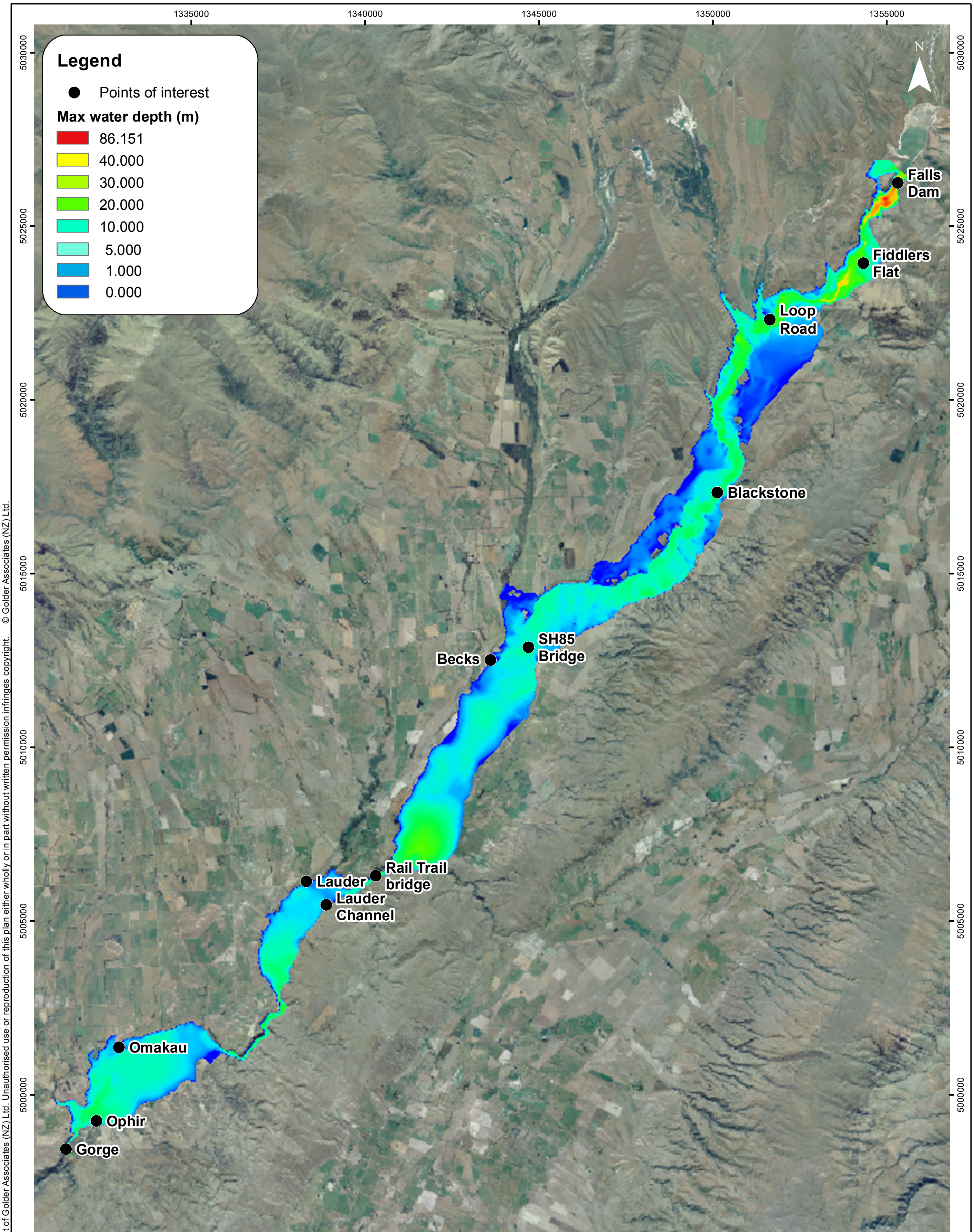
3.0 DAM BREAK MODELLING

3.1 Model Results

The modelled maximum flood extent and water depth is shown in Figure 3. Overall there is significant out of channel flooding throughout the length of the modelled Manuherikia River reach. The exceptions to this are where the river flows through gorges: north of Lauder, north of Omakau and south of Ophir.

Table 2 summarises the timing, depth, flow and velocity of the dam break flood flow at points of interest in the catchment. The towns of Becks, Omakau and Ophir are expected to suffer inundation to varying degrees, however Lauder is located just beyond the extent of the flooding.

The wetted front of the dam break flood travels through the modelled catchment within 3 hours. However, it takes almost 8 hours for the peak flood depth to travel from Falls Dam to the gorge at Ophir. Flood flows and velocities generally decrease throughout the river reach. The dam break peak flow of 111,100 m^3/s is estimated to dissipate to a peak of 1,600 m^3/s by the time it reaches the gorge at Ophir.



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TITLE FALLS DAM BREAK (FSL OF 588 m amsl)
MAXIMUM FLOOD EXTENT

JULY 2014

PROJECT 1378110270

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Table 2: Summary of dam break results for Falls Dam with a FSL of 588 m amsl.

Location	Distance downstream (km)	Time to arrival of wetted front [A] (hr:min)	Time to arrival of maximum depth (hr:min)	Maximum depth of water (m)	Maximum flow [B] (m^3/s)	Maximum velocity [C] (m/s)	Wetted floodplain width (m)
Fiddlers Flat	2.9	0:05	0:24	27.6	89,000	16	1,000
Loop Road	6.6	0:17	0:28	4.8	74,000	2.0	2,050
Blackstone	12.7	0:27	0:36	6.7	65,000	5.6	1,250
SH85 Bridge	22.0	0:47	0:53	10.8	51,000	7.4	1,500
Becks	22.0	0:50	0:54	0.6	51,000	1.3	1,500
Rail Trail Bridge	30.5	1:12	1:42	11.4	8,200	1.7	170
Lauder channel	32.7	1:18	1:44	1.8	8,100	2.7	800
Lauder	32.7	-	-	0.0	8,100	0.0	800
Omakau	42.7	3:45	7:47	4.7	4,600	0.13	1,800
Ophir	44.0	2:55	7:48	11.3	3,200	0.30	1,400
Gorge	45.4	-	-	-	1,600	-	-

Notes: [A] Time to an inundation depth of 0.1 m. [B] Maximum flow across floodplain cross section. [C] Locations vary between points in the main river channel and points of interest in the floodplain, refer to Figure 3.

4.0 DAM BREAK CONSEQUENCES

4.1 Population at Risk

The population at risk (PAR) is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. The population at risk varies throughout the day and throughout the seasons (USBR 1999). The number of people undertaking recreational activities outdoors in and around the Manuherikia River will change depending on whether it is summer or winter and the distribution of the population will vary between day and night.

4.2 Population at Risk for Falls Dam – FSL of 588 m amsl

The PAR downstream from Falls Dam was estimated based on the inundation area from the dam break analysis. Population and census data was used to estimate permanent residences in the inundation area. The population per residence in the inundation area is based on a projected average of 2.5 people per household (Statistics NZ 2014) and an inspection of aerial imagery to determine number of residences inundated by a dam failure. For the towns of Omakau and Ophir, where the flood map indicates the whole town is likely to be inundated, population estimates were adopted (CentralOtagoNZ 2014).

The Otago Central Rail Trail is a 150 km long cycling trail from Clyde to Middlemarch. The trail travels up the Manuherikia Valley from Clyde and crosses into the Ida Valley near Lauder (approximately 40 km) (Otago Central Rail Trail 2014). It is estimated that approximately 10,000 - 12,000 people cycle the trail each year (ODT 2011). The majority of the users are likely in the warmer months (November to April) which results in 66 users per day during the summer. Assuming 5 % of the Otago Central Rail Trail could be affected (7.5 km), approximately 3 cyclists would be at risk.

State Highway 85 (SH85) follows the Manuherikia River through the catchment and at times is located very close to the river channel. Other rural roads may also be inundated. However traffic rates on these roads are too low and the population at risk is estimated to be very low. There is an estimated annual average daily traffic volume (AADT) of 500 on SH85 near Lauder (Transit 2006). Assuming 10 % of the Manuherikia River length of SH85 (6 km) could be affected by inundation from a dam break over a 6 hour period, approximately 12 vehicles would be at risk. This could equate to approximately 25 people at risk on the roads.

The rivers and lakes in the Manuherikia catchment are popular trout fisheries. Other recreational uses of waterways include game bird hunting, kayaking and swimming (MCWSG 2013). Assuming access may be gained to the Manuherikia River primarily around the locations of road bridges and townships, it is estimated that 10 people could be at risk during a dam break event during summer.

The estimated population at risk for various downstream locations are presented in Table 3.

Table 3: Estimated population at risk at various locations downstream from Falls Dam.

Location	Distance downstream (km)	Estimated Population at Risk (PAR)			
		Properties	OCRT	Roads	Recreation
Falls Dam to Fiddlers Flat	2.9	3			
Fiddlers Flat to Loop Road	6.6	5			
Loop Road to Blackstone	12.7	5			2
Blackstone to SH85 Bridge/Becks	22.0	20		6	2
SH85 Bridge/Becks to Rail Trail Bridge	30.5	55	1	6	
Rail Trail Bridge to Lauder	32.7	3	1		2
Lauder to Omakau	42.7	45	1	6	2
Omakau to Ophir	44.0	140[2]		6	2
Ophir to Gorge	45.4	40[2]			
Estimated PAR		**353**			

Notes: Property estimates based on a projected average of 2.5 people per household (Statistics NZ 2014), and an inspection of aerial imagery to determine number of residences. [2] Based on Central Otago Population Statistics for Omakau and Ophir (CentralOtagoNZ 2014). OCRT – Otago Central Rail Trail.

5.0 GUIDELINES AND LEGISLATION

New Zealand Building (Dam Safety) Regulations 2008 (amended 2010) identifies a damage level, based on damage to homes, critical infrastructure, natural environment and community recovery time (Table 4). A subsequent dam classification is based on the damage level and the population at risk (Table 5).

Table 4: Determination of damage level (DBH 2008).

	Residential houses	Critical or major infrastructure		Natural environment	Community recovery time
		Damage	Time to restore to operation		
Catastrophic	>50 houses destroyed	Extensive and widespread destruction of and damage to several major components	>1 year	Extensive and widespread damage	Many years
Major	4 – 49 houses destroyed	Extensive destruction of and damage to more than one major component	Up to 12 months	Heavy damage and costly restoration	Years
Moderate	1 – 3 houses destroyed	Significant damage to at least one major component	Up to 3 months	Significant but recoverable damage	Months
Minimal	Minor damage	Minor damage	Up to 1 week	Short-term damage	Days to weeks

Table 5: Determination of dam classification (DBH 2008).

Assessed damage level	Population at risk			
	0	1 to 10	11 to 100	More than 100
Catastrophic	High	High	High	High
Major	Medium	Medium/High	High	High
Moderate	Low	Low/Medium/High	Medium/High	Medium/High
Minimal	Low	Low/Medium/High	Low/Medium/High	Low/Medium/High

The New Zealand Society on Large Dams (NZSOLD) provides initial screening advice regarding the PIC of dams, related to broad dam height and storage volume parameters. NZSOLD also indicates potential impact categories in terms of failure consequences (life, financial, environmental and socio-economic) (Table 6).

Table 6: Potential impact categories for dams in terms of failure consequences (NZSOLD 2000).

Potential Impact Category	Potential incremental consequences of failure	
	Life	Socio-economic, financial and environmental
High	Fatalities	Catastrophic damages
Medium	A few fatalities are possible	Major damages
Low	No fatalities expected	Moderate damages
Very low	No fatalities	Minimal damages beyond owner's property

6.0 POTENTIAL IMPACT CATEGORY

The purpose of a PIC is to understand the potential consequences (loss of life, socio-economic, financial and environmental) of failure of Falls Dam with a FSL of 588 m amsl.

Based on an inspection of aerial imagery within the floodplain, an estimated 126 residences would be inundated to some degree. The level of damage to these properties would vary, but according to Table 4, this would be considered a 'major' to 'catastrophic' damage level.

At least 6 road / pedestrian bridges span the Manuherikia River between Falls Dam and Ophir Gorge. Other critical infrastructure in the floodplain includes; the 1.2 MW capacity hydropower scheme located at the base of Falls Dam; at least 3 pieces of significant irrigation infrastructure (major intakes and siphons); community electricity distribution networks and various other local community infrastructure. The consequences of a dam break would be considered as widespread and extensive damage to several infrastructure components, and likely to be described as 'catastrophic' damage according to Table 4.

Dam break modelling indicates an expected peak flow of 1,600 m^3/s at the Ophir Gorge. This is almost twice the estimated 1 in 500 year return period peak flow for the Ophir site of 940 m^3/s (Aqualinc 2012b). This large flood event, and the large floodplain width, indicates significant damage to the natural environment. According to Table 4 this would likely be considered a 'moderate' to 'major' damage level.

Large tracts of agricultural land would suffer inundation, and community infrastructure and facilities would be damaged or destroyed. With a failure of Falls Dam, potentially 21,000 ha of land would lose its supply of irrigation water. This would have a major impact on the livelihoods of farmers and the community economy. As the area consists of small, rural communities, the time to repair and reconstruct communities would span years and would likely be considered as 'major' to 'catastrophic' damage according to Table 4.

Overall, based on Table 4, the assessed damage level for a dam break of Falls Dam with a FSL of 588 m amsl would be ‘major’ to ‘catastrophic’. When combined with the estimated population at risk of 353 (Table 3), Table 5 determines the dam to be of High PIC classification.

With regard to the NZSOLD guidelines (Table 6), a PIC classification of High is also estimated. The flood wave travel time to the properties closest to the dam is very short (<15 minutes) and the flood water depths are significant (over 25 m high near Fiddlers Flat) making evacuation difficult. The water will also be moving quickly and evacuation routes are limited. The flood wave travel time to the more populated areas (Becks) is still under an hour and still moving quickly, potentially making warning and evacuation difficult. Due to the proximity of the population at risk and the high flood wave velocity, fatalities are probable. Combined with the previously discussed catastrophic damages to infrastructure, communities and the environment, a High PIC is concluded.

7.0 MODEL LIMITATIONS

A sensitivity analysis has not been undertaken and there are limitations to the accuracy of the model output. Model limitations are noted below:

- Natural flows in the Manuherikia River and its tributaries have been ignored in the model. This is to increase model runtime efficiencies.
- A Manning’s value of 0.04 was applied as a constant value across the entire river channel and floodplain. However, differing vegetation in the channel and floodplain could cause this to vary.
- Due to the magnitude and speed of the breach flow, some water appears to flow upstream (north and west of Falls Dam) down a small gully. This flow is lost to the model, but the volume lost is not considered of significance.
- Due to the extent of the modelled area, a model grid of 30 m was applied. This grid size limits the accuracy of the model in narrow areas such as gorges downstream of Lauder and downstream of Ophir. The model may be creating additional backwater effects which would have the following effects:
  - increasing the time to inundation of downstream infrastructure, and
  - decreasing the magnitude of inundation of downstream infrastructure.

  As the areas of the model which receive the largest and most rapid inundation are upstream of these gorges, it is not considered to have a significant impact on the results of the model. However, it should be considered in future modelling for evacuation planning purposes at detailed design stage.
- The underlying ground elevation data for the model was compiled from a number of sources including 20 m topographical data (supplied electronically) and 5 m topographical data (only available on hard-copy maps). There are a number of limitations on this data:
  - Alignment of data between sources.
  - Delineation of hard-copy topographical maps into an electronic version.
  - Age of the data sources (some map sources from 1976) and potential river channel changes.
  - River channel depth was ignored.
  - Truncation of the topographical data due to the model grid size.

It is recommended that the model is refined during detailed design once the dam configuration is confirmed.

During final design is it suggested that the terrain model be refined through site specific topographic surveys and a sensitivity analysis, possibly varying channel roughness (Manning's n), hydrograph, grid size, and other variables. Extending the model to the confluence of the Clutha River is also recommended. This may require an increase in the processing capability of the modelling software.

Even though only the dam break of Falls Dam with a FSL of 588 m amsl was analysed, a similar PIC is estimated for smaller RCC raises. Flood extents, depths, and velocities may be reduced for a smaller dam raise but major to catastrophic damages are still expected to critical infrastructure and the population at risk will not likely be reduced significantly (>100). If a concrete faced rockfill dam is selected for final design, the PIC is again not expected to change. The critical failure mode will likely become a rainy day failure which would result in a more water being released downstream if a failure were to occur. This increase in flows is likely to offset the longer breach formation but this should be confirmed during final design.

8.0 CONCLUSIONS

The resulting PIC for Falls Dam with a FSL of 588 m amsl is High. Various dam options are currently being assessed and final dam configuration (size and type) will not be confirmed by the MCWSG until after the current feasibility study. This dam breach assessment has been completed using standard methodologies based on the potentially worst case scenario of a maximum storage volume and a dam type (RCC) that results in a rapid failure mode.

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APPENDIX A

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APPENDIX C

Stability Analysis

Stability Analysis

Introduction

This Appendix summarises the results of the simplified stability analyses performed for Falls Dam with full supply levels of 592.2 m, 580.4 and 570.6 m. The stability analyses were performed for usual, unusual, and extreme loading conditions. Usual conditions are under static loading conditions, unusual are based on hydrologic loadings, and extreme conditions are based on seismic loadings. The analyses were performed using the computer analysis program CADAM (version 1.4.14, dated 23 July 2004). CADAM is based on the gravity method (rigid body equilibrium and beam theory). The purpose of this stability analysis is to make an estimate of a stable dam configuration under the static, peak flood and peak earthquake loading conditions. More detailed and rigorous analyses will be required to determine the actual dam configuration under multiple loading conditions and these types of analyses are not included in the current scope of work. Outputs from CADAM are attached at the end of this Appendix.

RCC Gravity Dam Geometry

The seismic stability analysis analysed the maximum height section of the embankment for full supply levels of 592.2 m, 580.4 m and 570.6 m. The associated dam heights are 71.2 m, 59.4 m and 49.6 m, respectively. The dam crests are all 8 m wide and the distance from the dam crest to the full supply level is 4 m. The downstream slope is relatively shallow, 1H:1V, for an RCC dam to account for the large seismic loadings. The reservoir water surface elevation during the inflow design flood (IDF) is estimated to increase above the full supply level by 3 m. A typical section is presented in Figure C1 below and also included in Appendix F.

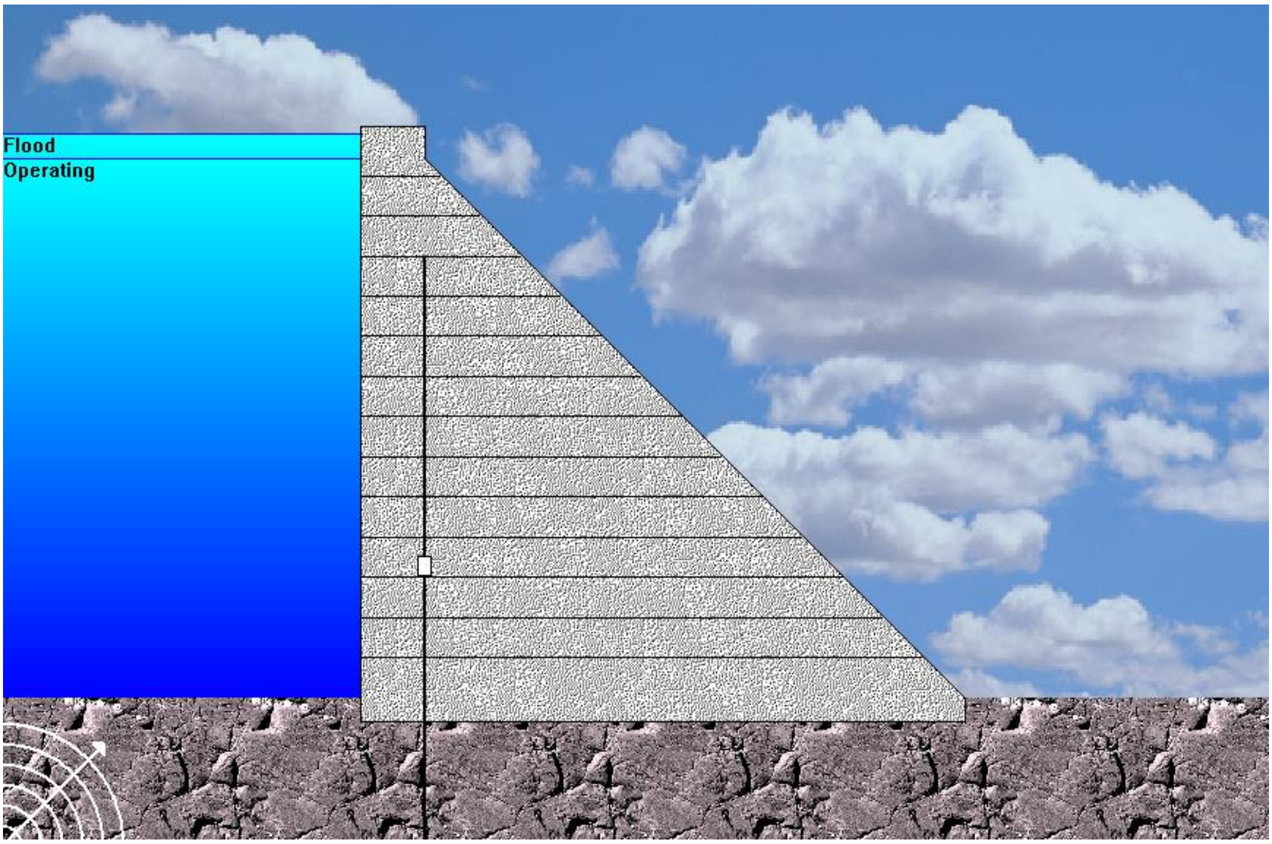

Figure C1: Typical Maximum Section.

A drainage gallery is located 8 m from the upstream toe at an elevation of 545 m. The drains extend up to within 10 m of the full supply level and drain effectiveness is estimated to be 50 percent (USACE 1995), but will have to confirmed in final design.

The lift joints were spaced at 5 m to simplify the model but lift joints will be spaced much closer during construction. The lifts have been assumed to be flat lying and each is assumed to have bedding mortar. Only the maximum section of the embankment was modelled as part of this preliminary analysis.

Passive shear strength from the rock below the downstream ground level was included in the analysis. The rock was assumed to have a unit mass of 2,400 kg/m^3, cohesion of 100 kPa and a friction angle of 40 degrees. Laboratory testing will be required to confirm these strengths.

RCC Gravity Dam Properties

The RCC at Falls Dam will likely require a high compressive strength to achieve higher tensile and shear strength as required due to the high seismic ground motions. For the preliminary analysis it is assumed that the compressive strength of the RCC is 20,000 kPa and the tensile strength, typically between 5 and 15 percent of the compressive strength, will be about 2,000 kPa (USACE 2000). The mass of the RCC is assumed to be 2,400 kg/m^3. Silt loads were ignored as part of the preliminary analysis as sedimentation surveys indicated that sedimentation was not a significant issue (Golder 2014a).

Typical strength properties of the RCC lift joints and the base joint are presented in Table C1 below. Most of the preliminary strengths are based on guidance provided by the US Army Corps of Engineers (USACE) (2000). The values in Table C1 are subject to confirmation by actual RCC test results that are recommended for the next project phase.

Table C1. RCC Joint and Base Joint Strength Properties.

Material	Compressive Strength (f'c)	Tensile Strength (ft)[1]	Peak Shear Strength		Residual Shear Strength		Minimum Normal Compressive Stress for Cohesion (σ_n)
			Cohesion (c)[1]	Friction Angle (φ)	Cohesion (c)[1]	Friction Angle (φ)[1]	
	kPa	kPa	kPa	degree	kPa	degree	kPa
RCC Joint	20 000	2 000	1 000	48[2]	0	45	36
Base Joint	20 000	2 000	1 000	50	0	45	36

1. Properties based on US Army Corps of Engineers, EM 1110-2-2006, 15 January 2000. Guidance recommends c = 0.05f'c for preliminary design for RCC lift joint receiving mortar.
2. Properties from typical test results of bedded RCC at Saluda Dam (Schrader and Rizzo 2000).
3. Represents a nominal amount.

The seismic stability analysis was only performed for the maximum design earthquake (MDE). A seismic stability analysis for the operating basis earthquake (OBE) and during aftershock shaking will likely be required as part of future final design work. To meet accepted dam safety standards, the dam should be designed to withstand the MDE without severe damage and without uncontrolled release of the reservoir. Since the new Falls Dam is estimated to be a high potential impact category (PIC), the MDE is either the controlling maximum earthquake (CME) or maximum credible earthquake (MCE), which is the largest earthquake demand that can be reasonably expected at the site given the tectonic setting. Where determined probabilistically, the MDE is usually the event having an annual exceedance probability of 1 in 10,000. Where deterministically evaluated, the MDE should be the 84th percentile ground motions for the CME, but need not have an annual exceedance probability of less than 1 in 10,000.

At this early preliminary stage, a deterministic seismic hazard assessment was undertaken, understanding that a probabilistic seismic hazard assessment will need to be undertaken as part of final design. The deterministic seismic hazard resulted in a peak horizontal ground acceleration of 1.0 g for the 1 in 10,000 year ground motion. The Blue Lake Fault's surface trace is within 6 km of the site.

The seismic coefficient method was used to evaluate the dynamic response of the RCC dam. This method is a simplified analytical method and a dynamic analysis will be required in final design. The seismic coefficient method suggests using a peak horizontal ground acceleration equal to ⅔ the effective peak ground acceleration (EPGA). The EPGA is equal to dividing the 0.3 second spectral acceleration, for the design event, by 2.5 (USACE 2005). The 0.3 second spectral acceleration is presented in *Geotechnical Stage One Report: Background Review and Investigations* by Golder (2014b) and is equal to 1.917 g which results in an EPGA of 0.7688 g. The peak horizontal seismic coefficient to be used in the seismic analysis is therefore equal to 0.51 g. The peak vertical seismic coefficient used in this preliminary analysis is equal to 0.7 of the peak horizontal seismic coefficient, or 0.36 g (USACE 2007). The stability of the embankment was analysed under these peak ground accelerations and also under sustained accelerations due to the seismic event. The sustained accelerations are equal to 0.67 of the peak values (Ecole 2001).

A summary of the applied seismic loadings are presented in Table C2 below.

Table C2: Loading Conditions.

Loading Case	Event Return Period (yrs)	Horizontal Peak Ground Accel. (g)	Horizontal Sustained Ground Accel. (g)	Vertical Peak Ground Accel. (g)	Vertical Sustained Ground Accel. (g)
Pseudo-static	10,000	0.51	0.34	0.36	0.24

Flood Loading

The 10,000 year return period flood is modelled for the flood loading condition. Based on the flood inflows from Aqualinc (2013), the reservoir is expected to rise 3 m above the full supply level during the inflow design flood (IDF). Details of the flood loading and anticipated increase in the reservoir pool are described in the main body of the report.

Loading Conditions

Falls Dam was analysed under three different loading conditions for sliding, uplift and overturning types of failures. The loading conditions are:

- Usual.
- Unusual – IDF Flood Loading.
- Extreme – MDE Earthquake, peak and sustained ground motions (Pseudo-Static Analysis).

A pseudo-dynamic analysis was not performed as part of this preliminary analysis as the required inputs involve a better understanding of the design earthquake and dam properties and behaviour which are outside the current scope of work. Due to the large seismic event anticipated at the site, a dynamic analysis will be required during final design.

The loading conditions for each of these cases are presented in Table C3 below.

Table C3: Loading Conditions.

Loading Case	Reservoir Elevation (m)	Tailwater Elevation (m)	Horizontal Peak Ground Accel. (g)	Vertical Peak Ground Accel. (g)
Usual – Static	Full Supply Level	525	0	0
Unusual – IDF Loading	Full Supply Level +3 m	525	0	0
Extreme – MDE (pseudo-static)	Full Supply Level (Westergaard Procedure)	525	0.51	0.36

Results

The basic stability requirements for gravity dams for each loading condition are:

- Be safe against overturning at any horizontal plane within the structure, at the base, or at a plane below the base.
- Safe against sliding on any horizontal or near-horizontal plane within the structure at the base or on any rock seam in the foundation.
- All allowable unit stresses in the concrete or foundation material shall not be exceeded (USACE 1995).

The stability of the dam in sliding and uplift was assessed through calculation of factors of safety (against sliding). The stability of the dam in overturning was based on force resultant location and the check of allowable stresses was done by comparing the normal stresses developed at the upstream and downstream sides of the dam against allowable stress for the different loading conditions. The minimum factors of safety on the sliding stability accepted for safety of the dam are based on New Zealand Society on Large Dams (NZSOLD) guidelines. Golder also assessed the factor of safety against overturning based on the resultant location and concrete stresses, NZSOLD does not provide guidelines for acceptable factors of safety for these conditions so guidance from the US Army Corps of Engineers was used (1995). The minimum factors of safety as recommended by NZSOLD and USACE (1995) are summarised in Table C4.

Table C4: Required Factors of Safety.[1]

Loading Case	Sliding		Resultant Location, % of Base in Compression (Overturning)	Concrete Stress	
	Peak	Residual		Compressive	Tensile
Usual – Static	3.0	1.5	Middle 1/3, 100 %	$0.3 f'_c$	0
Unusual – IDF Loading	2.0	1.3	Middle ½, 75 %	$0.5 f'_c$	$0.6 f'_c{}^{2/3}$
Extreme – MDE (pseudo-static)	1.3	1.0	Within Base, N/A	$0.9 f'_c$	$1.5 f'_c{}^{2/3}$

1. Required factors of safety based on New Zealand Society on Large Dams (NZSOLD) guidelines and resultant location, % of base in compression, and concrete stresses are based on guidelines from the US Army Corps of Engineers (1995).

The results of the analyses are presented in Tables C5, C6 and C7.

Table C5: Results of Stability Analysis – Full Supply Level 592.2 m.

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	5.9	2.4	OK	OK	OK
Unusual – IDF Loading	5.4	2.2	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.1[2]	0.4	FAIL	OK	FAIL[4]
Extreme – MDE Sustained Ground Accel. (pseudo-static)	1.8	0.7[3]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. Only applies to from base to el. 535, FS above 1.3 above El 535.
3. If peak factors of safety are met, materials not expected to reduce to residual strengths.
4. Only the base joint failed.

Table C6: Results of Stability Analysis – Full Supply Level 580.4 m.

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	7.1	2.6	OK	OK	OK
Unusual – IDF Loading	6.4	2.3	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.3	0.5[2]	FAIL[3]	OK	OK
Extreme – MDE Sustained Ground Accel. (pseudo-static)	2.1	0.7[2]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. If peak factors of safety are met, materials not expected to reduce to residual strengths.
3. The resultants for most lift layers are acceptable; the resultant is only outside of the base below El. 535 m.

Table C7: Results of Stability Analysis – Full Supply Level 570.6 m.

Loading Case	Sliding[1]		Resultant Location, % Base in Compression	Concrete Stress[1]	
	Peak	Residual		Compressive	Tensile
Usual – Static	7.7	2.6	OK	OK	OK
Unusual – IDF Loading	6.8	2.3	OK	OK	OK
Extreme – MDE Peak Ground Accel. (pseudo-static)	1.4	0.5[2]	FAIL[3]	OK	OK
Extreme – MDE Sustained Ground Accel. (pseudo-static)	2.3	0.8[2]	OK	OK	OK

1. The lowest factor of safety and lowest stress reported.
2. If peak factors of safety are met, materials not expected to reduce to residual strengths.
3. The resultants for most lift layers are acceptable; the resultant is only outside of the base below El. 535 m.

The model has been shown to be sensitive to the strength of the RCC lift joints and base joints. A slight increase in cohesion along the base increases the factor of safety significantly at the base lift and increases in lift cohesion increases factors of safety at each lift. A better understanding of the RCC mix design and strength properties, likely to be evaluated through a test section and laboratory testing, will be required during final design.

Conclusions

The dam meets the minimum required factors of safety under usual and unusual loading conditions and during the peak loadings of the extreme event for the low and medium height dams. The peak loadings of the extreme event for the high dam are not met and the factors of safety are not met under the peak and sustained residual strengths of the extreme loading event for any dam height. Even though the factors of safety were not met for all loading conditions, it does not mean the dam will fail catastrophically but that there may be some movement or cracking along lift lines. A better understanding of the amount and direction of movement will be required to estimate the response of the dam during the seismic event.

The percentage of the base in compression and the ratio of normal compressive strength to RCC strength were met for all cases. The tensile forces did not exceed the tensile strength of the RCC, except along the base lift of the tallest dam under peak seismic loading, and therefore cracks are not expected to form under the majority of the loading conditions. The resultant location falls within the required percentage of the base except under peak loadings of the extreme event where it falls up to 20 percent of the base width outside of the footprint along the bottom lifts. Further understanding and estimates of overturning under the peak loadings of the extreme event will be required in the next design phase. The factors of safety against sliding considering residual strength parameters drop below minimum safety factors for both the peak and sustained ground motions loading cases for all analysed dam heights. However, as the factors of safety for peak strength are acceptable for all but the highest dam, it is anticipated that there will not be sufficient deterioration or damage to the lift and base joint strengths to induce decreases in the strengths to the residual values for the two lower dam height options. As a result, the computed low factor of safety values with residual strengths may not be a representative loading condition. This simplified pseudo-static analysis is a screening tool that indicates a more rigorous dynamic analysis will be required in the next phase of design to verify stability. Time histories and dynamic responses of the dam will have to be assessed while also taking into account three dimensional effects of the canyon. Further refinement of the dam design and geometry should be expected as a result of these more rigorous analyses.

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CADAM Static Results

CADAM 2000 - Result report

Project:	Manuherikia Dam Raise	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	570.6m	Date:	12 August 14
Dam location:		Units:	Metric

LOAD COMBINATION FACTORS

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic
Self-weight	1.000	1.000	1.000		
Hydrostatic (upstream)	1.000	1.000	1.000		
Hydrostatic (downstream)	1.000	1.000	1.000		
Uplift pressures	1.000	1.000	1.000		
Seismic (horizontal)			-1.000		
Seismic (vertical)			-1.000		

USUAL COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	570.000					-115.070	-81.522	0.000	-6660.000	0.000	-26.900	33.249	81.522
2	565.000					-223.361	-41.901	0.000	-6660.000	0.000	-1.098	13.779	41.901
3	560.000					-299.624	-63.938	0.000	-6660.000	0.000	63.938	100.000	63.938
4	555.000					-358.347	-94.835	0.000	-6660.000	0.000	94.835	100.000	94.835
5	550.000					-411.482	-132.569	0.000	-6660.000	0.000	132.569	100.000	132.569
6	545.000					-461.712	-173.902	0.000	-6660.000	0.000	173.902	100.000	173.902
7	540.000					-510.242	-217.356	0.000	-6660.000	0.000	217.356	100.000	217.356
8	535.000					-537.670	-257.666	0.000	-6660.000	0.000	257.666	100.000	257.666
9	530.000					-563.447	-299.818	0.000	-6660.000	0.000	299.818	100.000	299.818
10	Base					-561.143	-452.593	0.000	-6660.000	0.000	322.983	50.000	0.000

USUAL COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S							
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	570.000	> 100	> 100	48.250	27.411	34.400	-845.3	1.8	-206.8	47.156	25.3	0.000
2	565.000	> 100	11.727	6.591	5.100	5.829	-1803.8	153.8	-2796.9	38.599	373.6	0.000
3	560.000	40.563	6.135	7.939	4.397	5.890	-3381.1	551.1	-6794.8	39.196	691.5	0.000
4	555.000	24.746	4.480	7.424	3.809	5.423	-5347.5	1193.7	-12230.4	40.309	1209.0	0.000
5	550.000	17.891	3.738	7.210	3.490	5.207	-7779.9	2081.5	-19011.6	41.456	1849.1	0.000
6	545.000	14.142	3.322	7.101	3.289	5.088	-10678.3	3214.5	-27077.1	42.453	2611.8	0.000
7	540.000	11.800	3.058	7.036	3.150	5.015	-14042.6	4592.8	-36365.6	43.291	3497.2	0.000
8	535.000	10.111	2.789	6.026	2.860	4.440	-17338.3	6216.4	-44356.4	44.132	5039.8	0.000
9	530.000	8.892	2.595	5.445	2.660	4.072	-20977.3	8085.2	-51890.2	44.910	6827.7	0.000
10	Base	7.586	2.490	4.651	2.359	3.603	-27168.1	11541.3	-25988.4	48.215	10438.2	1570.931

FLOOD COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	570.000					-79.480	-87.682	1000.000	-10000.000	0.000	-19.622	29.953	87.682
2	565.000					-169.510	-66.322	1000.000	-10000.000	0.000	66.322	100.000	66.322
3	560.000					-237.418	-105.101	1000.000	-10000.000	0.000	105.101	100.000	105.101
4	555.000					-288.874	-144.605	1000.000	-10000.000	0.000	144.605	100.000	144.605
5	550.000					-336.887	-188.334	1000.000	-10000.000	0.000	188.334	100.000	188.334
6	545.000					-383.331	-234.064	1000.000	-10000.000	0.000	234.064	100.000	234.064
7	540.000					-428.956	-280.877	1000.000	-10000.000	0.000	280.877	100.000	280.877
8	535.000					-454.088	-323.832	1000.000	-10000.000	0.000	323.832	100.000	323.832
9	530.000					-478.006	-368.122	1000.000	-10000.000	0.000	368.122	100.000	368.122
10	Base					-465.204	-531.621	1000.000	-10000.000	0.000	364.246	50.000	0.000

FLOOD COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S							
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	570.000	> 100	11.307	8.216	4.211	5.733	-718.8	63.6	50.5	50.818	151.9	0.000
2	565.000	42.398	4.421	4.581	3.002	3.795	-1603.7	362.8	-1590.5	42.708	573.7	0.000
3	560.000	24.402	3.511	6.663	3.083	4.590	-3185.4	907.2	-3814.7	43.562	887.2	0.000
4	555.000	17.255	3.014	6.683	2.910	4.548	-5115.1	1696.9	-6696.0	44.453	1441.5	0.000
5	550.000	13.522	2.749	6.717	2.806	4.545	-7510.7	2731.9	-10125.9	45.286	2118.4	0.000
6	545.000	11.246	2.585	6.744	2.737	4.555	-10372.2	4012.1	-14043.0	45.971	2917.9	0.000
7	540.000	9.718	2.474	6.764	2.688	4.568	-13699.8	5537.5	-18386.0	46.523	3840.0	0.000
8	535.000	8.543	2.320	5.903	2.506	4.129	-16958.7	7308.3	-20634.3	47.209	5419.4	0.000
9	530.000	7.661	2.205	5.387	2.377	3.838	-20560.9	9324.2	-21628.5	47.836	7244.1	0.000
10	Base	6.685	2.173	4.669	2.150	3.453	-26714.9	13015.7	15900.9	51.110	10891.5	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam Raise	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	580.400	Date:	12 August 14
Dam location:		Units:	Metric

LOAD COMBINATION FACTORS

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic
Self-weight	1.000	1.000	1.000		
Hydrostatic (upstream)	1.000	1.000	1.000		
Hydrostatic (downstream)	1.000	1.000	1.000		
Uplift pressures	1.000	1.000	1.000		
Seismic (horizontal)			-1.000		
Seismic (vertical)			-1.000		

USUAL COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	580.000					-108.968	-84.232	0.000	-6660.000	0.000	-29.981	33.299	90.314
2	575.000					-220.461	-36.729	0.000	-6660.000	0.000	-1.222	14.782	39.381
3	570.000					-297.658	-55.844	0.000	-6660.000	0.000	59.877	100.000	59.877
4	565.000					-358.714	-83.482	0.000	-6660.000	0.000	89.509	100.000	89.509
5	560.000					-415.290	-117.214	0.000	-6660.000	0.000	125.677	100.000	125.677
6	555.000					-469.703	-153.988	0.000	-6660.000	0.000	165.106	100.000	165.106
7	550.000					-522.921	-192.487	0.000	-6660.000	0.000	206.385	100.000	206.385
8	545.000					-575.414	-232.056	0.000	-6660.000	0.000	248.811	100.000	248.811
9	540.000					-627.437	-272.332	0.000	-6660.000	0.000	291.994	100.000	291.994
10	535.000					-658.076	-309.637	0.000	-6660.000	0.000	331.993	100.000	331.993
11	530.000					-687.887	-347.894	0.000	-6660.000	0.000	373.013	100.000	373.013
12	Base					-695.396	-490.954	0.000	-6660.000	0.000	371.320	50.000	0.000

USUAL COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	580.000	> 100	> 100	71.701	39.459	50.236	-814.2	0.8	-146.4	47.866	16.5	0.000
2	575.000	> 100	12.398	6.548	5.187	5.855	-1773.3	143.0	-2911.5	38.094	365.3	0.000
3	570.000	43.184	6.380	7.832	4.480	5.888	-3384.9	530.5	-7390.6	38.599	692.5	0.000
4	565.000	26.246	4.659	7.289	3.895	5.414	-5419.5	1163.3	-13780.7	39.626	1227.9	0.000
5	560.000	18.962	3.896	7.059	3.585	5.198	-7953.7	2041.3	-22166.7	40.671	1894.8	0.000
6	555.000	14.990	3.472	6.936	3.392	5.080	-10987.5	3164.5	-32661.6	41.563	2693.2	0.000
7	550.000	12.513	3.203	6.861	3.260	5.008	-14521.0	4533.0	-45378.2	42.302	3623.0	0.000
8	545.000	10.829	3.019	6.811	3.165	4.961	-18554.0	6146.7	-60429.7	42.913	4684.4	0.000
9	540.000	9.613	2.884	6.775	3.093	4.928	-23086.7	8005.7	-77928.9	43.422	5877.2	0.000
10	535.000	8.619	2.713	6.007	2.876	4.473	-27424.0	10110.0	-93277.0	43.999	7896.5	0.000
11	530.000	7.843	2.579	5.512	2.717	4.157	-32129.4	12459.5	-109047.9	44.529	10178.7	0.000
12	Base	6.989	2.490	4.759	2.455	3.718	-39980.0	16684.7	-77394.0	47.128	14707.5	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam Raise	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	580.400	Date:	12 August 14
Dam location:		Units:	Metric

FLOOD COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	580.000					-74.120	-89.651	1000.000	-10000.000	0.000	-23.254	30.621	96.124
2	575.000					-168.578	-59.182	1000.000	-10000.000	0.000	63.456	100.000	63.456
3	570.000					-238.947	-93.694	1000.000	-10000.000	0.000	100.459	100.000	100.459
4	565.000					-293.882	-128.796	1000.000	-10000.000	0.000	138.096	100.000	138.096
5	560.000					-346.224	-167.624	1000.000	-10000.000	0.000	179.727	100.000	179.727
6	555.000					-397.548	-208.086	1000.000	-10000.000	0.000	223.111	100.000	223.111
7	550.000					-448.419	-249.375	1000.000	-10000.000	0.000	267.380	100.000	267.380
8	545.000					-499.069	-291.125	1000.000	-10000.000	0.000	312.145	100.000	312.145
9	540.000					-549.607	-333.153	1000.000	-10000.000	0.000	357.207	100.000	357.207
10	535.000					-579.025	-371.896	1000.000	-10000.000	0.000	398.747	100.000	398.747
11	530.000					-607.814	-411.354	1000.000	-10000.000	0.000	441.055	100.000	441.055
12	Base					-607.646	-562.242	1000.000	-10000.000	0.000	410.553	50.000	0.000

FLOOD COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	580.000	> 100	12.172	8.598	4.298	5.910	-690.2	56.7	92.0	51.581	140.6	0.000
2	575.000	44.883	4.537	4.482	3.029	3.764	-1570.4	346.1	-1733.6	41.995	568.2	0.000
3	570.000	25.761	3.616	6.523	3.143	4.570	-3185.2	880.7	-4439.4	42.722	892.3	0.000
4	565.000	18.225	3.119	6.522	2.985	4.531	-5180.3	1660.6	-8265.7	43.490	1467.1	0.000
5	560.000	14.296	2.858	6.540	2.893	4.531	-7675.1	2685.8	-13281.7	44.207	2173.5	0.000
6	555.000	11.901	2.697	6.555	2.833	4.543	-10669.4	3956.2	-19600.3	44.786	3011.3	0.000
7	550.000	10.294	2.588	6.565	2.791	4.558	-14163.4	5471.8	-27334.5	45.246	3980.6	0.000
8	545.000	9.142	2.510	6.572	2.760	4.573	-18157.1	7232.7	-36597.3	45.614	5081.4	0.000
9	540.000	8.277	2.452	6.576	2.737	4.588	-22650.3	9238.9	-47501.5	45.913	6313.6	0.000
10	535.000	7.537	2.345	5.897	2.590	4.219	-26948.1	11490.3	-55448.4	46.370	8372.4	0.000
11	530.000	6.946	2.260	5.447	2.478	3.956	-31614.1	13986.9	-63011.9	46.787	10694.1	0.000
12	Base	6.286	2.222	4.756	2.272	3.583	-39425.2	18447.5	-17188.2	49.353	15262.3	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	592.2m	Date:	12 August 14
Dam location:		Units:	Metric

LOAD COMBINATION FACTORS

	Usual	Flood	Seismic #1	Seismic #2	Post-seismic
Self-weight	1.000	1.000	1.000		
Hydrostatic (upstream)	1.000	1.000	1.000		
Hydrostatic (downstream)	1.000	1.000	1.000		
Uplift pressures	1.000	1.000	1.000		
Seismic (horizontal)			-1.000		
Seismic (vertical)			-1.000		

USUAL COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	590.000					-159.631	-58.936	0.000	-6660.000	0.000	-16.575	31.904	58.936
2	585.000					-245.301	-41.936	0.000	-6660.000	0.000	41.936	100.000	41.936
3	580.000					-319.307	-72.741	0.000	-6660.000	0.000	72.741	100.000	72.741
4	575.000					-375.783	-106.377	0.000	-6660.000	0.000	106.377	100.000	106.377
5	570.000					-427.797	-145.496	0.000	-6660.000	0.000	145.496	100.000	145.496
6	565.000					-477.389	-187.622	0.000	-6660.000	0.000	187.622	100.000	187.622
7	560.000					-525.523	-231.573	0.000	-6660.000	0.000	231.573	100.000	231.573
8	555.000					-572.714	-276.713	0.000	-6660.000	0.000	276.713	100.000	276.713
9	550.000					-619.260	-322.671	0.000	-6660.000	0.000	322.671	100.000	322.671
10	545.000					-665.346	-369.214	0.000	-6660.000	0.000	369.214	100.000	369.214
11	540.000					-711.092	-416.191	0.000	-6660.000	0.000	416.191	100.000	416.191
12	535.000					-735.064	-460.489	0.000	-6660.000	0.000	460.489	100.000	460.489
13	530.000					-758.407	-505.473	0.000	-6660.000	0.000	505.473	100.000	505.473
14	Base					-748.798	-665.858	0.000	-6660.000	0.000	481.274	50.000	0.000

USUAL COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	590.000	> 100	46.954	13.858	9.563	11.127	-1114.7	23.7	-873.0	42.322	110.1	0.000
2	585.000	69.313	8.585	5.660	4.390	5.067	-2183.0	254.3	-3915.5	38.200	536.8	0.000
3	580.000	33.693	5.424	7.714	4.161	5.693	-3959.7	730.1	-8384.1	39.518	843.8	0.000
4	575.000	22.016	4.187	7.337	3.688	5.338	-6075.2	1451.1	-14257.0	40.688	1400.5	0.000
5	570.000	16.470	3.581	7.168	3.416	5.162	-8656.7	2417.4	-21455.8	41.793	2079.8	0.000
6	565.000	13.282	3.225	7.077	3.239	5.061	-11704.2	3628.9	-29919.3	42.738	2881.8	0.000
7	560.000	11.228	2.992	7.021	3.114	4.998	-15217.6	5085.7	-39586.2	43.529	3806.4	0.000
8	555.000	9.800	2.828	6.984	3.022	4.955	-19197.1	6787.7	-50395.2	44.192	4853.6	0.000
9	550.000	8.753	2.707	6.958	2.951	4.925	-23642.5	8735.0	-62284.8	44.752	6023.4	0.000
10	545.000	7.953	2.613	6.939	2.894	4.903	-28553.8	10927.6	-75193.9	45.229	7315.9	0.000
11	540.000	7.324	2.539	6.924	2.848	4.886	-33931.2	13365.3	-89061.1	45.640	8731.0	0.000
12	535.000	6.760	2.429	6.302	2.696	4.521	-38975.0	16048.4	-97269.1	46.172	11068.2	0.000
13	530.000	6.296	2.338	5.865	2.577	4.250	-44362.2	18976.7	-103872.3	46.665	13650.7	0.000
14	Base	5.809	2.270	5.136	2.362	3.846	-53191.1	24127.9	-39085.8	49.023	18691.6	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	592.2m	Date:	12 August 14
Dam location:		Units:	Metric

FLOOD COMBINATION (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	590.000					-117.947	-71.190	1000.000	-10000.000	0.000	-7.640	23.741	71.190
2	585.000					-186.660	-71.147	1000.000	-10000.000	0.000	71.147	100.000	71.147
3	580.000					-254.471	-117.035	1000.000	-10000.000	0.000	117.035	100.000	117.035
4	575.000					-304.486	-158.287	1000.000	-10000.000	0.000	158.287	100.000	158.287
5	570.000					-351.871	-202.808	1000.000	-10000.000	0.000	202.808	100.000	202.808
6	565.000					-397.998	-248.954	1000.000	-10000.000	0.000	248.954	100.000	248.954
7	560.000					-443.447	-296.007	1000.000	-10000.000	0.000	296.007	100.000	296.007
8	555.000					-488.496	-343.612	1000.000	-10000.000	0.000	343.612	100.000	343.612
9	550.000					-533.297	-391.575	1000.000	-10000.000	0.000	391.575	100.000	391.575
10	545.000					-577.932	-439.780	1000.000	-10000.000	0.000	439.780	100.000	439.780
11	540.000					-622.454	-488.158	1000.000	-10000.000	0.000	488.158	100.000	488.158
12	535.000					-645.380	-533.652	1000.000	-10000.000	0.000	533.652	100.000	533.652
13	530.000					-667.819	-579.669	1000.000	-10000.000	0.000	579.669	100.000	579.669
14	Base					-650.844	-747.531	1000.000	-10000.000	0.000	523.365	50.000	0.000

FLOOD COMBINATION (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	590.000	84.982	7.273	6.103	3.664	4.708	-964.6	132.6	-405.4	45.880	260.2	0.000
2	585.000	34.050	3.839	4.287	2.813	3.576	-1959.3	510.3	-2224.0	42.532	760.5	0.000
3	580.000	21.502	3.311	6.664	3.016	4.569	-3752.2	1133.3	-4673.3	43.834	1051.2	0.000
4	575.000	15.827	2.913	6.694	2.872	4.545	-5830.9	2001.4	-7736.8	44.735	1644.7	0.000
5	570.000	12.682	2.689	6.726	2.781	4.548	-8375.7	3114.9	-11329.3	45.521	2360.9	0.000
6	565.000	10.695	2.545	6.751	2.720	4.559	-11386.3	4473.6	-15389.4	46.160	3199.6	0.000
7	560.000	9.331	2.446	6.769	2.675	4.572	-14863.0	6077.5	-19855.7	46.677	4161.0	0.000
8	555.000	8.337	2.372	6.782	2.641	4.585	-18805.7	7926.7	-24667.1	47.098	5245.0	0.000
9	550.000	7.582	2.317	6.791	2.614	4.598	-23214.3	10021.1	-29762.1	47.446	6451.6	0.000
10	545.000	6.990	2.272	6.797	2.593	4.610	-28088.9	12360.8	-35079.5	47.738	7780.9	0.000
11	540.000	6.512	2.237	6.802	2.576	4.621	-33429.4	14945.7	-40557.9	47.985	9232.8	0.000
12	535.000	6.069	2.162	6.233	2.466	4.312	-38436.4	17775.9	-39580.0	48.421	11606.8	0.000
13	530.000	5.699	2.100	5.826	2.379	4.078	-43786.8	20851.4	-36200.4	48.822	14226.1	0.000
14	Base	5.314	2.064	5.143	2.204	3.724	-52578.9	26238.0	45564.1	51.152	19303.7	1570.931

CADAM Seismic Results

CADAM 2000 - Result report

Project:	Manuherikia Dam Raise	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	570.6m	Date:	12 August 14
Dam location:		Units:	Metric

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	570.000					11.135	-134.835	1818.000	-18180.000	0.000	134.835	100.000	134.835
2	565.000					52.980	-202.971	1818.000	-18180.000	0.000	202.971	100.000	202.971
3	560.000					117.585	-323.498	1818.000	-18180.000	0.000	323.498	100.000	323.498
4	555.000					195.007	-448.159	1818.000	-18180.000	0.000	448.159	100.000	448.159
5	550.000					274.327	-575.970	1818.000	-18180.000	0.000	575.970	100.000	575.970
6	545.000					353.615	-704.440	1818.000	-18180.000	0.000	704.440	100.000	704.440
7	540.000					432.233	-832.664	1818.000	-18180.000	0.000	832.664	100.000	832.664
8	535.000					530.019	-955.808	1818.000	-18180.000	0.000	955.808	100.000	955.808
9	530.000					627.851	-1079.190	1818.000	-18180.000	0.000	1079.190	100.000	1079.190
10	Base					927.636	-1436.212	1818.000	-18180.000	0.000	1019.092	50.000	0.000

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	570.000	*16.449*	*1.171*	*3.391*	*1.534*	*2.570*	-531.9	454.4	899.7	69.667	25.3	0.000
2	565.000	*7.262*	*0.676*	*2.940*	*1.190*	*1.881*	-1019.9	1509.4	3945.1	78.441	373.6	0.000
3	560.000	*4.595*	*0.586*	*3.347*	*1.115*	*1.888*	-1915.0	3266.4	12716.4	85.702	691.5	0.000
4	555.000	*3.365*	*0.526*	*3.425*	*1.056*	*1.837*	-2987.2	5677.1	29851.5	92.344	1209.0	0.000
5	550.000	*2.701*	*0.495*	*3.479*	*1.021*	*1.811*	-4313.5	8721.5	57959.1	96.981	1849.1	0.000
6	545.000	*2.288*	*0.476*	*3.516*	*0.998*	*1.797*	-5893.9	12388.1	99541.7	100.265	2611.8	0.000
7	540.000	*2.006*	*0.464*	*3.542*	*0.983*	*1.788*	-7728.3	16668.8	157053.9	102.647	3497.2	0.000
8	535.000	*1.755*	*0.431*	*3.381*	*0.952*	*1.709*	-9282.2	21557.7	235374.9	108.160	5039.8	0.000
9	530.000	*1.567*	*0.405*	*3.269*	*0.928*	*1.651*	-10967.5	27050.3	335996.8	113.036	6827.7	0.000
10	Base	*1.372*	*0.417*	*3.085*	*0.867*	*1.568*	-13629.8	36415.5	565936.9	127.466	10438.2	1570.931

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation (m)	Crack length (%)	(m)	Crack length (%)	(m)	Upstream (kPa)	Downstream (kPa)	tension (kPa)	Compression (kPa)	Upstream (kPa)	Maximum (kPa)	Maximum at l-axis (% of joint)	Downstream (kPa)
1	570.000					-30.934	-117.064	1818.000	-18180.000	0.000	-0.897	8.022	117.064
2	565.000					-39.134	-149.281	1818.000	-18180.000	0.000	149.281	100.000	149.281
3	560.000					-21.485	-236.978	1818.000	-18180.000	0.000	236.978	100.000	236.978
4	555.000					10.556	-330.385	1818.000	-18180.000	0.000	330.385	100.000	330.385
5	550.000					45.724	-428.170	1818.000	-18180.000	0.000	428.170	100.000	428.170
6	545.000					81.839	-527.594	1818.000	-18180.000	0.000	527.594	100.000	527.594
7	540.000					118.075	-627.561	1818.000	-18180.000	0.000	627.561	100.000	627.561
8	535.000					174.123	-723.094	1818.000	-18180.000	0.000	723.094	100.000	723.094
9	530.000					230.752	-819.399	1818.000	-18180.000	0.000	819.399	100.000	819.399
10	Base					431.376	-1108.339	1818.000	-18180.000	0.000	787.056	50.000	0.000

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation (m)	Peak	Residual	Toward U/S	Toward D/S		(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	570.000	30.546	2.097	4.580	2.238	3.717	-636.4	303.5	530.9	59.700	25.3	0.000
2	565.000	14.206	1.212	3.419	1.598	2.430	-1281.2	1057.5	1697.7	59.743	373.6	0.000
3	560.000	8.900	1.018	3.899	1.484	2.440	-2403.7	2361.3	6212.7	63.896	691.5	0.000
4	555.000	6.172	0.902	3.939	1.391	2.356	-3774.0	4182.6	15824.2	67.767	1209.0	0.000
5	550.000	4.737	0.840	3.977	1.336	2.315	-5469.0	6508.2	32302.2	70.652	1849.1	0.000
6	545.000	3.903	0.803	4.006	1.300	2.291	-7488.7	9330.2	57335.4	72.787	2611.8	0.000
7	540.000	3.360	0.778	4.027	1.275	2.276	-9833.1	12643.5	92580.7	74.392	3497.2	0.000
8	535.000	2.892	0.728	3.786	1.224	2.150	-11967.6	16443.9	142131.1	77.239	5039.8	0.000
9	530.000	2.556	0.690	3.623	1.186	2.059	-14304.1	20728.6	206701.1	79.733	6827.7	0.000
10	Base	2.174	0.701	3.362	1.099	1.932	-18142.6	28124.1	368628.5	87.907	10438.2	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam Raise	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	580.400	Date:	12 August 14
Dam location:		Units:	Metric

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	580.000					10.286	-132.522	1818.000	-18180.000	0.000	142.090	100.000	142.090
2	575.000					42.968	-188.499	1818.000	-18180.000	0.000	202.109	100.000	202.109
3	570.000					99.965	-300.171	1818.000	-18180.000	0.000	321.844	100.000	321.844
4	565.000					168.792	-415.729	1818.000	-18180.000	0.000	445.745	100.000	445.745
5	560.000					238.737	-533.871	1818.000	-18180.000	0.000	572.417	100.000	572.417
6	555.000					308.151	-652.278	1818.000	-18180.000	0.000	699.374	100.000	699.374
7	550.000					376.581	-770.183	1818.000	-18180.000	0.000	825.792	100.000	825.792
8	545.000					443.949	-887.339	1818.000	-18180.000	0.000	951.407	100.000	951.407
9	540.000					510.302	-1003.694	1818.000	-18180.000	0.000	1076.163	100.000	1076.163
10	535.000					596.787	-1115.811	1818.000	-18180.000	0.000	1196.375	100.000	1196.375
11	530.000					683.029	-1227.799	1818.000	-18180.000	0.000	1316.448	100.000	1316.448
12	Base					955.216	-1557.367	1818.000	-18180.000	0.000	1179.841	50.000	0.000

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final	Passive
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S						Force	wedge resistance
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	580.000	*17.064*	*1.199*	*3.421*	*1.565*	*2.632*	-515.2	429.7	845.5	69.472	16.5	0.000
2	575.000	*7.563*	*0.673*	*2.909*	*1.210*	*1.884*	-1003.4	1490.5	3668.0	76.508	365.3	0.000
3	570.000	*4.743*	*0.582*	*3.295*	*1.136*	*1.887*	-1917.1	3296.4	12229.4	83.310	692.5	0.000
4	565.000	*3.461*	*0.523*	*3.354*	*1.078*	*1.836*	-3026.4	5790.3	29266.7	89.452	1227.9	0.000
5	560.000	*2.776*	*0.493*	*3.395*	*1.045*	*1.810*	-4408.2	8949.7	57455.7	93.630	1894.8	0.000
6	555.000	*2.351*	*0.475*	*3.421*	*1.024*	*1.796*	-6062.5	12761.5	99358.8	96.515	2693.2	0.000
7	550.000	*2.062*	*0.464*	*3.438*	*1.010*	*1.787*	-7989.1	17216.8	157484.1	98.558	3623.0	0.000
8	545.000	*1.852*	*0.457*	*3.449*	*1.000*	*1.781*	-10188.2	22308.8	234301.1	100.042	4684.4	0.000
9	540.000	*1.693*	*0.452*	*3.456*	*0.993*	*1.776*	-12659.7	28032.6	332250.4	101.142	5877.2	0.000
10	535.000	*1.532*	*0.428*	*3.322*	*0.970*	*1.714*	-14708.6	34383.9	458461.5	104.994	7896.5	0.000
11	530.000	*1.403*	*0.409*	*3.222*	*0.952*	*1.665*	-16898.5	41359.3	612871.9	108.460	10178.7	0.000
12	Base	*1.265*	*0.412*	*3.045*	*0.900*	*1.590*	-20292.5	53014.2	951172.0	119.545	14707.5	1570.931

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		allowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	580.000					-29.465	-116.425	1818.000	-18180.000	0.000	-3.048	13.374	124.832
2	575.000					-44.842	-137.909	1818.000	-18180.000	0.000	147.866	100.000	147.866
3	570.000					-32.576	-218.729	1818.000	-18180.000	0.000	234.522	100.000	234.522
4	565.000					-7.043	-304.980	1818.000	-18180.000	0.000	327.000	100.000	327.000
5	560.000					20.728	-394.985	1818.000	-18180.000	0.000	423.504	100.000	423.504
6	555.000					48.866	-486.181	1818.000	-18180.000	0.000	521.285	100.000	521.285
7	550.000					76.747	-577.618	1818.000	-18180.000	0.000	619.323	100.000	619.323
8	545.000					104.161	-668.911	1818.000	-18180.000	0.000	717.208	100.000	717.208
9	540.000					131.055	-759.907	1818.000	-18180.000	0.000	814.774	100.000	814.774
10	535.000					178.499	-847.086	1818.000	-18180.000	0.000	908.248	100.000	908.248
11	530.000					226.057	-934.497	1818.000	-18180.000	0.000	1001.970	100.000	1001.970
12	Base					405.012	-1201.896	1818.000	-18180.000	0.000	910.334	50.000	0.000

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final	Passive
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S						Force	wedge resistance
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	580.000	31.581	2.145	4.671	2.302	3.848	-614.8	286.7	514.8	59.934	16.5	0.000
2	575.000	14.586	1.210	3.388	1.625	2.434	-1260.1	1041.3	1474.8	58.488	365.3	0.000
3	570.000	9.184	1.013	3.843	1.512	2.440	-2406.4	2374.5	5689.4	62.346	692.5	0.000
4	565.000	6.545	0.900	3.863	1.421	2.354	-3824.1	4247.9	14917.5	65.914	1227.9	0.000
5	560.000	5.010	0.841	3.886	1.368	2.313	-5590.1	6646.9	30914.9	68.513	1894.8	0.000
6	555.000	4.119	0.806	3.904	1.334	2.289	-7704.2	9562.5	55352.0	70.391	2693.2	0.000
7	550.000	3.542	0.783	3.916	1.311	2.274	-10166.4	12988.8	89863.3	71.774	3623.0	0.000
8	545.000	3.138	0.767	3.924	1.295	2.265	-12976.8	16921.5	136057.5	72.815	4684.4	0.000
9	540.000	2.840	0.756	3.930	1.283	2.258	-16135.4	21357.0	195524.0	73.613	5877.2	0.000
10	535.000	2.543	0.721	3.734	1.245	2.157	-18947.1	26292.6	274548.7	75.566	7896.5	0.000
11	530.000	2.314	0.693	3.591	1.215	2.081	-21975.5	31726.0	372232.0	77.303	10178.7	0.000
12	Base	2.035	0.695	3.344	1.141	1.965	-26855.0	40904.3	608316.6	83.608	14707.5	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	592.2m	Date:	12 August 14
Dam location:		Units:	Metric

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	590.000					17.359	-149.472	1818.000	-18180.000	0.000	149.472	100.000	149.472
2	585.000					82.180	-240.584	1818.000	-18180.000	0.000	240.584	100.000	240.584
3	580.000					153.468	-374.304	1818.000	-18180.000	0.000	374.304	100.000	374.304
4	575.000					237.824	-506.393	1818.000	-18180.000	0.000	506.393	100.000	506.393
5	570.000					322.817	-640.139	1818.000	-18180.000	0.000	640.139	100.000	640.139
6	565.000					407.077	-773.738	1818.000	-18180.000	0.000	773.738	100.000	773.738
7	560.000					490.202	-906.570	1818.000	-18180.000	0.000	906.570	100.000	906.570
8	555.000					572.124	-1038.444	1818.000	-18180.000	0.000	1038.444	100.000	1038.444
9	550.000					652.892	-1169.336	1818.000	-18180.000	0.000	1169.336	100.000	1169.336
10	545.000					732.594	-1299.288	1818.000	-18180.000	0.000	1299.288	100.000	1299.288
11	540.000					811.326	-1428.363	1818.000	-18180.000	0.000	1428.363	100.000	1428.363
12	535.000					910.695	-1553.623	1818.000	-18180.000	0.000	1553.623	100.000	1553.623
13	530.000					1009.698	-1678.573	1818.000	-18180.000	0.000	1678.573	100.000	1678.573
14	Base					1317.627	-2044.045	1818.000	-18180.000	0.000	1457.237	50.000	0.000

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S							
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	590.000	12.186	0.934	3.185	1.373	2.223	-673.8	721.0	1446.4	71.046	110.1	0.000
2	585.000	5.705	0.581	2.910	1.124	1.794	-1203.9	2071.6	6214.3	83.960	536.8	0.000
3	580.000	3.903	0.540	3.422	1.078	1.867	-2230.4	4128.8	17946.0	89.831	843.8	0.000
4	575.000	2.962	0.494	3.501	1.027	1.827	-3384.0	6852.3	39384.0	96.184	1400.5	0.000
5	570.000	2.429	0.469	3.554	0.996	1.806	-4791.6	10222.5	73187.9	100.577	2079.8	0.000
6	565.000	2.087	0.454	3.589	0.976	1.793	-6453.2	14227.0	121923.0	103.674	2881.8	0.000
7	560.000	1.849	0.444	3.613	0.962	1.785	-8369.0	18857.1	188103.2	105.911	3806.4	0.000
8	555.000	1.674	0.437	3.629	0.953	1.780	-10538.8	24106.4	274204.5	107.563	4853.6	0.000
9	550.000	1.539	0.433	3.640	0.945	1.776	-12962.7	29969.7	382674.0	108.807	6023.4	0.000
10	545.000	1.432	0.429	3.647	0.940	1.773	-15640.7	36442.8	515935.5	109.758	7315.9	0.000
11	540.000	1.345	0.427	3.652	0.936	1.771	-18572.8	43522.1	676393.7	110.496	8731.0	0.000
12	535.000	1.248	0.409	3.538	0.920	1.721	-20959.4	51204.7	872993.1	113.883	11068.2	0.000
13	530.000	1.167	0.395	3.447	0.907	1.680	-23477.5	59487.9	1103992.3	116.985	13650.7	0.000
14	Base	1.087	0.395	3.276	0.864	1.613	-27313.3	73056.2	1584198.9	127.129	18691.6	1570.931

CADAM 2000 - Result report

Project:	Manuherikia Dam	Project engineer:	
Dam:	Falls Dam	Analysis performed by:	
Full Supply Level:	592.2m	Date:	12 August 14
Dam location:		Units:	Metric

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)

Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable stresses		Shear			
ID	Upstream elevation	Crack length		Crack length		Upstream	Downstream	tension	Compression	Upstream	Maximum	Maximum at l-axis	Downstream
	(m)	(%)	(m)	(%)	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(% of joint)	(kPa)
1	590.000					-41.638	-119.293	1818.000	-18180.000	0.000	119.293	100.000	119.293
2	585.000					-26.980	-174.368	1818.000	-18180.000	0.000	174.368	100.000	174.368
3	580.000					-4.124	-273.783	1818.000	-18180.000	0.000	273.783	100.000	273.783
4	575.000					33.288	-373.054	1818.000	-18180.000	0.000	373.054	100.000	373.054
5	570.000					72.612	-475.258	1818.000	-18180.000	0.000	475.258	100.000	475.258
6	565.000					112.255	-578.366	1818.000	-18180.000	0.000	578.366	100.000	578.366
7	560.000					151.627	-681.571	1818.000	-18180.000	0.000	681.571	100.000	681.571
8	555.000					190.511	-784.534	1818.000	-18180.000	0.000	784.534	100.000	784.534
9	550.000					228.841	-887.114	1818.000	-18180.000	0.000	887.114	100.000	887.114
10	545.000					266.614	-989.263	1818.000	-18180.000	0.000	989.263	100.000	989.263
11	540.000					303.854	-1090.972	1818.000	-18180.000	0.000	1090.972	100.000	1090.972
12	535.000					362.109	-1189.245	1818.000	-18180.000	0.000	1189.245	100.000	1189.245
13	530.000					420.330	-1287.540	1818.000	-18180.000	0.000	1287.540	100.000	1287.540
14	Base					628.818	-1584.649	1818.000	-18180.000	0.000	1131.916	50.000	0.000

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)

Joint		Safety factors					Resultants				Uplift	Rock
		Sliding		Overturning		Uplifting	Normal	Shear	Moment	Position	Final Force	Passive wedge resistance
ID	Upstream elevation	Peak	Residual	Toward U/S	Toward D/S							
	(m)						(kN)	(kN)	(kN·m)	(% of joint)	(kN)	(kN)
1	590.000	22.741	1.680	4.009	1.922	3.032	-820.7	488.6	673.3	58.042	110.1	0.000
2	585.000	11.449	1.044	3.308	1.495	2.286	-1530.2	1465.8	2837.7	62.200	536.8	0.000
3	580.000	7.430	0.937	3.952	1.431	2.406	-2806.9	2995.9	9169.3	66.172	843.8	0.000
4	575.000	5.300	0.847	4.001	1.352	2.340	-4281.0	5051.9	21503.7	69.932	1400.5	0.000
5	570.000	4.193	0.798	4.040	1.304	2.306	-6079.9	7620.8	41640.0	72.678	2079.8	0.000
6	565.000	3.523	0.767	4.069	1.272	2.285	-8203.6	10694.3	71308.9	74.694	2881.8	0.000
7	560.000	3.073	0.747	4.089	1.250	2.272	-10651.9	14266.7	112206.7	76.204	3806.4	0.000
8	555.000	2.751	0.732	4.103	1.234	2.263	-13424.9	18333.5	166004.6	77.357	4853.6	0.000
9	550.000	2.510	0.722	4.113	1.222	2.257	-16522.7	22891.5	234354.4	78.255	6023.4	0.000
10	545.000	2.321	0.714	4.120	1.213	2.252	-19945.1	27937.7	318892.4	78.965	7315.9	0.000
11	540.000	2.170	0.708	4.124	1.206	2.249	-23692.3	33469.8	421242.1	79.534	8731.0	0.000
12	535.000	2.005	0.683	3.961	1.179	2.168	-26964.6	39485.9	549572.4	81.260	11068.2	0.000
13	530.000	1.870	0.662	3.833	1.156	2.104	-30439.1	45984.2	701370.8	82.823	13650.7	0.000
14	Base	1.720	0.661	3.595	1.095	2.000	-35939.2	56746.7	1043104.0	88.596	18691.6	1570.931

APPENDIX D

Cost Estimate for Full Supply Level 592.2m Option

Full Supply Level 592.2 m

Item No.	Description	Unit	Quantity	Unit Rate	Amount	Comment	Assumptions
1	**Project Management**						
1.1	Construction Management	7% of BCS		7%	$ 8,137,871		
1.2	Engineering and Design	10% of BCS		10%	$ 11,625,530		
			Item 1 Costs		$ 19,763,401		
2	**Consents and Permits**						
2.1	Consenting	2% of BCS		2%	$ 2,325,106		
2.2	Bonds and Insurance	5% of BCS		5%	$ 5,812,765		
			Item 2 Costs		$ 8,137,871		
3	**Site Establishement**						
3.1	Site Access	LS	1	$520,000	$ 520,000	Improve road from highway 85. Road is 9km and majority will need widening and some blasting and widening will be required around bluffs. Likely need to blast and widen approximately 1km. Include cost for site security, signage.	General road upgrade over 9km @ $40,000/km + widen 1km @ $100,000 + site secuity fence and signage LS $60,000
3.2	Site Construction Roads	km	5	$220,000	$ 1,100,000	Will likely need about 3km of construction roads on site, some areas very steep and may require switchbacks, blasting likely required for construction of about 1km of road. An additional 2km of road will need to be constructed during construction (as the dam height increases), assume 1km will be permenant site access.	4km construction road @ $200,000 + 1km permanent @ $300,000. Add $80,000 if seal required
3.3	Field Explorations	LS	1	$500,000	$ 500,000	Golder Estimate	
3.4	Survey	LS	1	$230,000	$ 230,000	Survey required for duration of project, likely need full site survey at least once a week. (assume duration of project 1 year)	Initial survey control $30,000 + $16,500 /mth
3.5	Construction Offices	LS	1	$130,000	$ 130,000		Office,smoko, ablution, crew +workshop/lab = 4 No- 12x4 portacom + container/canvas cover. Hire @ $2500/wk
3.6	Stream Diversion						
3.6.1	Extension of Offtake Outlet	LS	1	$3,310,000	$ 3,310,000	Extend existing 5.2m dia offtake an additional 160 m, include backfill concrete below extenstion (likely a few meters thick by a few meters wide) and foundation cleaning as concrete and pipe extension to be permenant structure beneath dam.	Foundation clean up LS $30,000 plus concrete 15m3/m @ $800/m3 x 160m = $1,920,000 + 5m dia precast units (16.3m circum x 0.3m = 5m3/lm @ $1500/m3 = $7,500/lm plus install @ $1,000/lm)
3.6.2	Improvement to Existing Offtake Gates	LS	1	$150,000	$ 150,000	Will likely require 2 concrete bulkheads in existing offtake shafts to power station, may also require gate upgrade at existing intake	Details unknown, bulkhead size/gate works. Allow $50,000 each
3.6.3	Bulkhead and close off stream diversion and existing dam outlets once construction completed	LS	1	$290,000	$ 290,000	Bulkhead offtake gate pipes and spillway after construction, spillway tunnel has 5.2m diameter	Bulkhead 2.5 time diameter, say 13m long = 260m3 @ $1000/m3 + contact grouting LS $30,000
3.7	Environmental Controls	LS	1	$320,000	$ 320,000	Include cost for site re-establishment post construction (planting and grading)	12mths construction cost @ $10,000/mths + rehabilitation of site LS $200,000
3.8	Quarry Establishment	LS	1	$100,000	$ 100,000	Quarry will be used for RCC aggregate, saddle dam rockfill, roads, concrete, riprap	Allow LS $100,000 to strip an open up site
3.8.1	Drill and Blast	LS	1	$4,140,000	$ 4,140,000	Produce enough material for RCC, saddle dam, roads, riprap	RCC 250,000m3+Concrete agg 35,000m3+Riprap 500m3+rockfill 10,000m3=300,000m3+15%=345,000m3 average primary production to stockpile @$12=$4,140,000
3.9	Access Bridge	LS	1	$1,070,000	$ 1,070,000	2 lane bridge, 80m span, permanent site access	Type of bridge unknown, will require 2 abuts, 2 piers, 3 spans LS=2x80k+3x250k+2x80k
3.10	Install Sediment Controls	LS	1	$200,000	$ 200,000	Controls to prevent excessive sediment from entering river	Details unknown,assume small dam or tank ssytem and flocking plus monitoting
3.11	RCC Test Section	LS	1	$500,000	$ 500,000	Include construction of test section, multiple RCC mix combinations, coring and laboratory testing	Trials and testing, say 500m3 @ $1000/m3
3.12	Establish power and phone to construction offices	LS	1	$200,000	$ 200,000	There is power at existing power house, need to get on top to construction offices	Nominal cost for construction power reticulation LS 100,000. Need to establish 3G or similar signal, allow $100,000
3.13	Establish Laboratory	LS	1	$290,000	$ 290,000	Capabilities will need to include strength testing of concrete cores, soil gradations, vebe testing. Will be onsite for duration of project and will need a full time employee	Building included in 3.5 above, allow to establish lab equipment, UCS testing probably at CTS in Alexandra, allow $50,000 plus technician $20,000/mth
			Item 3 Costs		$ 13,050,000		
4	**Foundation Treatment**						
4.1	Stripping	m3	1800	$20	$36,000	Assume 0.15m of overburden stripping along entire foundation and energy dissipator	150mm of stripping appears minimal
4.2	Excavation	m3	46600	$40	$1,864,000	Excavate 3m into hard rock along entire foundation. Also includes excavation into rock for energy dissipator	Excavation will be slow, assume no blasting, and include mechanical and final cleaning of foundation
4.3	Dental Concrete	m3	1800	$600	$1,080,000	Assume half of the foundation will require at least 0.3m of dental concrete for shaping and removing overhangs	Include final air water cleaning in preparation for concrete, foundation anchors, bulk concrete placement
4.4	Foundation Cleanup	m2	12050	$3.60	$43,380	Air and Water cleaning by hand	Include construction joint preparation of dental concrete
4.5	Foundation Grouting	m2	22600	$16	$361,600	Grout curtain extend to a depth equal to height of reservoir, one row of vertical holes and one row of angled holes, secondary/tertiary holes where needed	Assume down stage grouting using 2 rigs, 22600m2/100m2/hole= 226 holes @ 6 holes/day=40days. Drills 40dx2x$250/hrx12hrs=$240,000+Cement 226holesx10bagx 25kg+waste @15%=65tx$500=$32,500+Grout plant 40dx10hrx$150=$60,000+consumeables=$20,000>>>>>Total $352,000/22600=
			Item 4 Costs		$3,384,980		
5	**RCC and Spillway**						
5.1	Establish crushing and screening plant	LS	1	$200,000	$200,000	Assume screening aggregate, including sand, for production. Will need lots of sand	Mob and set up plant including water supply, power, sediment ponds and carry out trial production
5.2	Establish RCC production line	LS	1	$300,000	$300,000	Includes conveyor, hopper, and other plant associated with RCC production	Purchase or hire in pug mill, set up and trial
5.3	Produce and Place RCC	m3	235000	$250	$58,750,000	Assume blasting and screening required for all aggregate, cement is imported, 20MPa RCC. Dam will include gallery, drainage conduit, one low level outlet, overtopping spillway,bedding grout treatment between each lift. Do not have a mix design so difficult to estimate quantities of each material but will be cement heavy mix	RCC mix coares agg 1150kg, sand 850kg., cement 220kg produce at $200/m3. 65m/0.3=216lifts. 250000/216=1160m3/lift …$40/m3 to place+$10/m3 grout enriched joint=$250/m3
5.4	Produce and Place Formed Facing Concrete	m3	30800	$1,000	$30,800,000	Pricing includes transport, placement and compaction. 25MPa concrete with reinforcing	Jump formed concrete at $1000/m3
5.5	Produce and Place Formed Spillway Walls	m3	380	$1,500	$570,000	40 MPa reinforced concrete, 4m high walls on downstream face (1H:1V) of dam	Formed reinforced concrete
5.6	Produce and Place Formed Ogee Spillway Crest	m3	105	$2,000	$210,000	40 MPa reinforced concrete, detailed curvature	As above
5.7	Produce and Place Spillway Dissipation Structure	m3	1350	$1,500	$2,025,000	40 MPa reinforced concrete, quantity includes 7m tall walls, ~50 chute blocks at inlet, ~20 baffle blocks at outlet	
5.8	Produce and Place Bedding Mortar	m3		$0	$0		included in 5.3 above
5.9	Produce and Place Formed Intake Tower	m3	400	$2,000	$800,000	40 MPa reinforced concrete, dowel into dam (connect tower to dam face), keep interior open	
5.10	Produce and Place Riprap Downstream of Dissipation Structure	m3	350	$120	$42,000	Assume D50 equal to 0.3m, can come from onsite quarry	Rock size may need to be larger
			Item 5 Costs		$93,697,000		
6	**Offtake Structure**						
6.1	Supply and Install 2m Diamter Conduit through dam	LS	1	$1,376,000	$1,376,000	Concrete encased high pressure rated steel pipe, length is 52m to daylight at toe, additional 120m to discharge into river	Allow 172m @$8,000
6.2	Supply and Install 3 Trashracks on upstram face of Tower	EA	3	$110,000	$330,000	Cover 1mx2m gates	Assume galv or epoxy coated
6.3	Supply and Install 3 gates	EA	3	$30,000	$90,000	1mx2m wide sealing slide gates, including rails anchored to intake tower and connection cable	
6.4	Supply and install 1 gate	EA	1	$50,000	$50,000	2mx2m wide sealing slide gate, including rails anchored to intake tower and connection cable	
6.5	Supply and install instrumentation panel and controls	LS	1	$40,000	$40,000	Control for 4 hydraulic gates on site, incorporation of site-wide instrumentation allowing for real-time monitoring and control	Nominal figures
6.6	Supply and install Remote operating system	LS	1	$60,000	$60,000	Radio or cellular equipment to remotely control 4 offtake gates and remotely monitor site	
6.7	Backup Power Supplies	LS	1	$100,000	$100,000	Backup generator(s) to control gates in case of power loss	
6.8	Supply and install/construct control house	LS	1	$48,000	$48,000	Building on top of intake tower to house the electrical and controls equipment, maybe prefabricated	Assume 40m2 at $1200/m2
6.9	Supply and install gate hoist	LS	1	$100,000	$100,000	Framing, hydraulics, motors, and associated equipment required to raise and lower gates	Nominal figures
6.10	Supply and install electrical gear	LS	1	$80,000	$80,000	3 phase power to control house, medium voltage motor control center for gates, low voltage connections for lights, etc.	
			Item 6 Costs		$2,274,000		
7	**Drainage**						
7.1	Supply and Install Internal Drainage System	LS	1	$1,800,000	$1,800,000	Drain holes are spaced 5m apart, extend to a depth of 2/3 grout curtain depth into rock and up to El. About 580 in dam. Drainage gallery extends full length of dam, assume at El 545 (and up to 560 at abutments) and offset from upstream face by 8m (drainage gallery construction likely to be accounted for in RCC placement costs)	Assume 80 holes at 45m average = 3600lm @ $500m=
7.2	Install Waterstops	EA	35	$1,000	$35,000	Waterstops are grouted into foundation and terminate at the top of the dam. Assume spaced at 6m on center, 75lm per waterstop, PVC	Allow provisional sum $1,000/ea
			Item 7 Costs		$1,835,000		
8	**Demolition**						
8.1	Demolish existing dam to El. 545	m3	160200	$5	$801,000	Demolition consists of rockfill and concrete panels (concrete makes up about 2% of total demolition)	Assume side cast into existing reservoir dead storage, possible dozer push
8.2	Demolish existing power plant and substation	LS	1	$20,000	$20,000	Remove structure, no reuse of material all to waste	
			Item 8 Costs		$821,000		
9	**Instrumentation**						
9.1	Piezometer	EA	40	$3,500	$140,000	vibrating weir piezometers at crest and downstream slope, remote readings	
9.2	Seepage Weirs	EA	10	$500	$5,000	Most exit into drainage gallery	
9.3	Stuctural Monitoring Points	EA	25	$500	$12,500	Permanent monument that can be surveyed	
9.4	Early Warning System	LS	1	$30,000	$30,000	Includes alarms downstream, water level sensors upstream and downstream, water level sensor of reservoir level, and all will be remote system	Allow provisional sum
			Item 9 Costs		$187,500		
10	**Saddle Dam**						
10.1	Obtain and Place Low Permeability Core Material	m3	7850	$25	$196,250	Assume borrow source will be located within 1km of dam site, place in 0.15m thick lifts, compact	Moisture condition critical and placement subject to weather conditions
10.2	Obtain and PlaceSand Filter Material	m3	3800	$140	$532,000	Assume material is imported from Alexandra 70km haul distance	PSD critical
10.3	Obtain and Place Riprap	m3	415	$40	$16,600	Assume will be blasted from RCC quarry, D50 approximately 0.3m	Placement cost only
10.4	Obtain and Place Rockfill	m3	9650	$15	$144,750	Assume will be obtained from onsite quarry, placed in 0.3m thick lifts and compacted	Placement cost only
10.5	Foundation Excavation	m3	7590	$8	$60,720	Assume all excavation is soil (assume wasted) and the deepest excavation is 3m.	
10.6	construct access road to saddle dam site	LS	1	$50,000	$50,000	Assume road will be 1km long, no blasting required	
10.7	Strip Foundation Material	m3	1100	$5	$5,500	Assume 0.15m of overburden stripping along entire foundation	
			Item 10 Costs		$1,005,820		
				BCS Cost	$ 116,255,300		
				DCS Cost	$ 144,156,572	BCS+1+2	
11	**Uncosted Items**						
11.1	Uncosted Items	35% of DCS			$ 50,454,800		
			Preliminary Project Cost		**$ 194,611,372**	DCS+Uncosted Items	

APPENDIX E

Cost Estimate for Full Supply Level 580.4 m Option and 570.6 m Option

Full Supply Level 580.4 m

Item No.	Description	Unit	Quantity	Unit Rate	Amount	Comment	Assumptions
1	**Project Management**						
1.1	Construction Management	7% of BCS		7%	$ 5,940,724		
1.2	Engineering and Design	10% of BCS		10%	$ 8,486,748		
			Item 1 Costs		$ 14,427,472		
2	**Consents and Permits**						
2.1	Consenting	2% of BCS		2%	$ 1,697,350		
2.2	Bonds and Insurance	5% of BCS		5%	$ 4,243,374		
			Item 2 Costs		$ 5,940,724		
3	**Site Establishement**						
3.1	Site Access	LS	1	$520,000	$ 520,000	Improve road from highway 85. Road is 9km and majority will need widening and some blasting and widening will be required around bluffs. Likely need to blast and widen approximately 1km. Include cost for site security, signage.	General road upgrade over 9km @ $40,000/km + widen 1km @ $100,000 + site secuity fence and signage LS $60,000
3.2	Site Construction Roads	km	5	$220,000	$ 1,100,000	Will likely need about 3km of construction roads on site, some areas very steep and may require switchbacks, blasting likely required for construction of about 1km of road. An additional 2km of road will need to be constructed during construction (as the dam height increases), assume 1km will be permenant site access.	4km construction road @ $200,000 + 1km permanent @ $300,000. Add $80,000 if seal required
3.3	Field Explorations	LS	1	$500,000	$ 500,000	Golder Estimate	
3.4	Survey	LS	1	$211,500	$ 211,500	Survey required for duration of project, likely need full site survey at least once a week. (assume duration of project 11 months)	Initial survey control $30,000 + $16,500 /mth
3.5	Construction Offices	LS	1	$120,000	$ 120,000		Office,smoko, ablution, crew +workshop/lab = 4 No- 12x4 portacom + container/canvas cover. Hire @ $2500/wk
3.6	Stream Diversion						
3.6.1	Extension of Offtake Outlet	LS	1	$2,572,000	$ 2,572,000	Extend existing 5.2m dia offtake an additional 124 m, include backfill concrete below extenstion (likely a few meters thick by a few meters wide) and foundation cleaning as concrete and pipe extension to be permenant structure beneath dam.	Foundation clean up LS $30,000 plus concrete 15m3/m @ $800/m3 x 124m = $129,200 + 5m dia precast units (16.3m circum x 0.3m = 5m3/lm @ $1500/m3 = $7,500/lm plus install @ $1,000/lm)
3.6.2	Improvement to Existing Offtake Gates	LS	1	$150,000	$ 150,000	Will likely require 2 concrete bulkheads in existing offtake shafts to power station, may also require gate upgrade at existing intake	Details unknown, bulkhead size/gate works. Allow $50,000 each
3.6.3	Bulkhead and close off stream diversion and existing dam outlets once construction completed	LS	1	$290,000	$ 290,000	Bulkhead offtake gate pipes and spillway after construction, spillway tunnel has 5.2m diameter	Bulkhead 2.5 time diameter, say 13m long = 260m3 @ $1000/m3 + contact grouting LS $30,000
3.7	Environmental Controls	LS	1	$310,000	$ 310,000	Include cost for site re-establishment post construction (planting and grading)	11mths construction cost @ $10,000/mths + rehabilitation of site LS $200,000
3.8	Quarry Establishment	LS	1	$100,000	$ 100,000	Quarry will be used for RCC aggregate, roads, concrete, riprap	Allow LS $100,000 to strip an open up site
3.8.1	Drill and Blast	LS	1	$2,532,000	$ 2,532,000	Produce enough material for RCC, roads, riprap	RCC 155,600m3+Concrete agg 27,000m3+riprap 500m3=183,100m3+15%=211,000m3 average primary production to stockpile @$12=$2532000
3.9	Access Bridge	LS	1	$1,070,000	$ 1,070,000	2 lane bridge, 80m span, permanent site access	Type of bridge unknown, will require 2 abuts, 2 piers, 3 spans LS=2x80k+3x250k+2x80k
3.10	Install Sediment Controls	LS	1	$200,000	$ 200,000	Controls to prevent excessive sediment from entering river	Details unknown,assume small dam or tank ssytem and flocking plus monitoting
3.11	RCC Test Section	LS	1	$500,000	$ 500,000	Include construction of test section, multiple RCC mix combinations, coring and laboratory testing	Trials and testing, say 500m3 @ $1000/m3
3.12	Establish power and phone to construction offices	LS	1	$200,000	$ 200,000	There is power at existing power house, need to get on top to construction offices	Nominal cost for construction power reticulation LS 100,000. Need to establish 3G or similar signal, allow $100,000
3.13	Establish Laboratory	LS	1	$270,000	$ 270,000	Capabilities will need to include soil gradations, vebe testing. Will be onsite for duration of project and will need a full time employee	Building included in 3.5 above, allow to establish lab equipment, UCS testing probably at CTS in Alexandra, allow $50,000 plus technician $20,000/mth
			Item 3 Costs		$ 10,645,500		
4	**Foundation Treatment**						
4.1	Stripping	m3	1400	$20	$28,000	Assume 0.15m of overburden stripping along entire foundation and energy dissipator	150mm of stripping appears minimal
4.2	Excavation	m3	32636	$40	$1,305,440	Excavate 3m into hard rock along entire foundation. Also includes excavation into rock for energy dissipator	Excavation will be slow, assume no blasting, and include mechanical and final cleaning of foundation
4.3	Dental Concrete	m3	1400	$600	$840,000	Assume half of the foundation will require at least 0.3m of dental concrete for shaping and removing overhangs	Include final air water cleaning in preparation for concrete, foundation anchors, bulk concrete placement
4.4	Foundation Cleanup	m2	9400	$3.6	$33,840	Air and Water cleaning by hand	Include construction joint preparation of dental concrete
4.5	Foundation Grouting	m2	16200	$16	$259,200	Grout curtain extend to a depth equal to height of reservoir, one row of vertical holes and one row of angled holes, secondary/tertiary holes where needed	Assume down stage grouting using 2 rigs, 16200m2/100m2/hole= 162 holes @ 6 holes/day=27days. Drills 27dx2x$250/hrx12hrs=$162,000+Cement 162holesx10bagx 25kg+waste @15%=47tx$500=$23,500+Grout plant 27dx10hrx$150=$40,500+consumeables=$20,000>>>>>Total $246,000/16200=
			Item 4 Costs		$2,466,480		
5	**RCC and Spillway**						
5.1	Establish crushing and screening plant	LS	1	$200,000	$200,000	Assume screening aggregate, including sand, for production. Will need lots of sand	Mob and set up plant including water supply, power, sediment ponds and carry out trial production
5.2	Establish RCC production line	LS	1	$300,000	$300,000	Includes conveyor, hopper, and other plant associated with RCC production	Purchase or hire in pug mill, set up and trial
5.3	Produce and Place RCC	m3	155600	$250	$38,900,000	Assume blasting and screening required for all aggregate, cement is imported, 20MPa RCC. Dam will include gallery, drainage conduit, one low level outlet, overtopping spillway,bedding grout treatment between each lift. Do not have a mix design so difficult to estimate quantities of each material but will be cement heavy mix	RCC mix coares agg 1150kg, sand 850kg., cement 220kg produce at $200/m3. $40/m3 to place+$10/m3 grout enriched joint=$250/m3
5.4	Produce and Place Formed Facing Concrete	m3	22150	$1,000	$22,150,000	Pricing includes transport, placement and compaction. 25MPa concrete with reinforcing	Jump formed concrete at $1000/m3
5.5	Produce and Place Formed Spillway Walls	m3	300	$1,500	$450,000	40 MPa reinforced concrete, 4m high walls on downstream face (1H:1V) of dam	Formed reinforced concrete
5.6	Produce and Place Formed Ogee Spillway Crest	m3	125	$20,000	$2,500,000	40 MPa reinforced concrete, detailed curvature	As above
5.7	Produce and Place Spillway Dissipation Structure	m3	1550	$1,500	$2,325,000	40 MPa reinforced concrete, quantity includes 7m tall walls, ~50 chute blocks at inlet, ~20 baffle blocks at outlet	
5.8	Produce and Place Bedding Mortar	m3			$0		included in 5.3 above
5.9	Produce and Place Formed Intake Tower	m3	300	$2,000	$600,000	40 MPa reinforced concrete, dowel into dam (connect tower to dam face), keep interior open	
5.10	Produce and Place Riprap Downstream of Dissipation Structure	m3	450	$120	$54,000	Assume D50 equal to 0.3m, can come from onsite quarry	Rock size may need to be larger
			Item 5 Costs		$67,479,000		
6	**Offtake Structure**						
6.1	Supply and Install 2m Diamter Conduit through dam	LS	1	$968,000	$968,000	Concrete encased high pressure rated steel pipe, length is 45m to daylight at toe, additional 76m to discharge into river	Allow 121m @$8,000
6.2	Supply and Install 3 Trashracks on upstram face of Tower	EA	3	$110,000	$330,000	Cover 1mx2m gates	Assume galv or epoxy coated
6.3	Supply and Install 3 gates	EA	3	$30,000	$90,000	1mx2m wide sealing slide gates, including rails anchored to intake tower and connection cable	
6.4	Supply and install 1 gate	EA	1	$50,000	$50,000	2mx2m wide sealing slide gate, including rails anchored to intake tower and connection cable	
6.5	Supply and install instrumentation panel and controls	LS	1	$40,000	$40,000	Control for 4 hydraulic gates on site, incorporation of site-wide instrumentation allowing for real-time monitoring and control	Nominal figures
6.6	Supply and install Remote operating system	LS	1	$60,000	$60,000	Radio or cellular equipment to remotely control 4 offtake gates and remotely monitor site	
6.7	Backup Power Supplies	LS	1	$100,000	$100,000	Backup generator(s) to control gates in case of power loss	
6.8	Supply and install/construct control house	LS	1	$48,000	$48,000	Building on top of intake tower to house the electrical and controls equipment, maybe prefabricated	Assume 40m2 at $1200/m2
6.9	Supply and install gate hoist	LS	1	$100,000	$100,000	Framing, hydraulics, motors, and associated equipment required to raise and lower gates	Nominal figures
6.10	Supply and install electrical gear	LS	1	$80,000	$80,000	3 phase power to control house, medium voltage motor control center for gates, low voltage connections for lights, etc.	
			Item 6 Costs		$1,866,000		
7	**Drainage**						
7.1	Supply and Install Internal Drainage System	LS	1	$1,400,000	$1,400,000	Drain holes are spaced 5m apart, extend to a depth of 2/3 grout curtain depth into rock and up to El. About 570 in dam. Drainage gallery extends full length of dam, assume at El 545 (and up to 560 at abutments) and offset from upstream face by 8m (drainage gallery construction likely to be accounted for in RCC placement costs)	Assume 70 holes at 40m average = 2800lm @ $500m=
7.2	Install Waterstops	EA	30	$1,000	$30,000	Waterstops are grouted into foundation and terminate at the top of the dam. Assume spaced at 6m on center, 65 lm, PVC	Allow provisional sum $1,000/ea
			Item 7 Costs		$1,430,000		
8	**Demolition**						
8.1	Demolish existing dam to El. 545	m3	160200	$5	$801,000	Demolition consists of rockfill and concrete panels (concrete makes up about 2% of total demolition)	Assume side cast into existing reservoir dead storage, possible dozer push
8.2	Demolish existing power plant and substation	LS	1	$20,000	$20,000	Remove structure, no reuse of material all to waste	
			Item 8 Costs		$821,000		
9	**Instrumentation**						
9.1	Piezometer	EA	33	$3,500	$115,500	vibrating weir piezometers at crest and downstream slope, remote readings	
9.2	Seepage Weirs	EA	8	$500	$4,000	Most exit into drainage gallery	
9.3	Stuctural Monitoring Points	EA	20	$500	$10,000	Permanent monument that can be surveyed	
9.4	Early Warning System	LS	1	$30,000	$30,000	Includes alarms downstream, water level sensors upstream and downstream, water level sensor of reservoir level, and all will be remote system	Allow provisional sum
			Item 9 Costs		$159,500		
				BCS Cost	$ 84,867,480		
				DCS Cost	$ 105,235,675	BCS+1+2	
11	**Uncosted Items**						
11.1	Uncosted Items	35% of DCS			$ 36,832,486		
				Preliminary Project Cost	$ 142,068,162	DCS+Uncosted Items	

Full Supply Level 570.6 m

Item No.	Description	Unit	Quantity	Unit Rate	Amount	Comment	Assumptions
1	**Project Management**						
1.1	Construction Management	7% of BCS		7%	$ 4,350,248		
1.2	Engineering and Design	10% of BCS		10%	$ 6,214,640		
			Item 1 Costs		$ 10,564,888		
2	**Consents and Permits**						
2.1	Consenting	2% of BCS		2%	$ 1,242,928		
2.2	Bonds and Insurance	5% of BCS		5%	$ 3,107,320		
			Item 2 Costs		$ 4,350,248		
3	**Site Establishement**						
3.1	Site Access	LS	1	$520,000	$ 520,000	Improve road from highway 85. Road is 9km and majority will need widening and some blasting and widening will be required around bluffs. Likely need to blast and widen approximately 1km. Include cost for site security, signage.	General road upgrade over 9km @ $40,000/km + widen 1km @ $100,000 + site secuity fence and signage LS $60,000
3.2	Site Construction Roads	km	5	$220,000	$ 1,100,000	Will likely need about 3km of construction roads on site, some areas very steep and may require switchbacks, blasting likely required for construction of about 1km of road. An additional 2km of road will need to be constructed during construction (as the dam height increases), assume 1km will be permenant site access.	4km construction road @ $200,000 + 1km permanent @ $300,000. Add $80,000 if seal required
3.3	Field Explorations	LS	1	$500,000	$ 500,000	Golder Estimate	
3.4	Survey	LS	1	$195,000	$ 195,000	Survey required for duration of project, likely need full site survey at least once a week. (assume duration of project 10 months)	Initial survey control $30,000 + $16,500 /mth
3.5	Construction Offices	LS	1	$100,000	$ 100,000		Office,smoko, ablution, crew +workshop/lab = 4 No- 12x4 portacom + container/canvas cover. Hire @ $2500/wk
3.6	Stream Diversion						
3.6.1	Extension of Offtake Outlet	LS	1	$2,285,000	$ 2,285,000	Extend existing 5.2m dia offtake an additional 110 m, include backfill concrete below extenstion (likely a few meters thick by a few meters wide) and foundation cleaning as concrete and pipe extension to be permenant structure beneath dam.	Foundation clean up LS $30,000 plus concrete 15m3/m @ $800/m3 x 110m = $118,000 + 5m dia precast units (16.3m circum x 0.3m = 5m3/lm @ $1500/m3 = $7,500/lm plus install @ $1,000/lm)
3.6.2	Improvement to Existing Offtake Gates	LS	1	$150,000	$ 150,000	Will likely require 2 concrete bulkheads in existing offtake shafts to power station, may also require gate upgrade at existing intake	Details unknown, bulkhead size/gate works. Allow $50,000 each
3.6.3	Bulkhead and close off stream diversion and existing dam outlets once construction completed	LS	1	$290,000	$ 290,000	Bulkhead offtake gate pipes and spillway after construction, spillway tunnel has 5.2m diameter	Bulkhead 2.5 time diameter, say 13m long = 260m3 @ $1000/m3 + contact grouting LS $30,000
3.7	Environmental Controls	LS	1	$300,000	$ 300,000	Include cost for site re-establishment post construction (planting and grading)	10mths construction cost @ $10,000/mths + rehabilitation of site LS $200,000
3.8	Quarry Establishment	LS	1	$100,000	$ 100,000	Quarry will be used for RCC aggregate, roads, concrete, riprap	Allow LS $100,000 to strip an open up site
3.8.1	Drill and Blast	LS	1	$1,692,000	$ 1,692,000	Produce enough material for RCC, roads, riprap	RCC 101,150m3+Concrete agg 20,000m3+Riprap 500m3=122,000m3+15%=141,000m3 average primary production to stockpile @$12=$1,692,000
3.9	Access Bridge	LS	1	$1,070,000	$ 1,070,000	2 lane bridge, 80m span, permanent site access	Type of bridge unknown, will require 2 abuts, 2 piers, 3 spans LS=2x80k+3x250k+2x80k
3.10	Install Sediment Controls	LS	1	$200,000	$ 200,000	Controls to prevent excessive sediment from entering river	Details unknown,assume small dam or tank system and flocking plus monitoting
3.11	RCC Test Section	LS	1	$500,000	$ 500,000	Include construction of test section, multiple RCC mix combinations, coring and laboratory testing	Trials and testing, say 500m3 @ $1000/m3
3.12	Establish power and phone to construction offices	LS	1	$200,000	$ 200,000	There is power at existing power house, need to get on top to construction offices	Nominal cost for construction power reticulation LS 100,000. Need to establish 3G or similar signal, allow $100,000
3.13	Establish Laboratory	LS	1	$250,000	$ 250,000	Capabilities will need to include soil gradations, vebe testing. Will be onsite for duration of project and will need a full time employee	Building included in 3.5 above, allow to establish lab equipment, UCS testing probably at CTS in Alexandra, allow $50,000 plus technician $20,000/mth
			Item 3 Costs		$ 9,452,000		
4	**Foundation Treatment**						
4.1	Stripping	m3	1200	$20	$24,000	Assume 0.15m of overburden stripping along entire foundation and energy dissipator	150mm of stripping appears minimal
4.2	Excavation	m3	34200	$40	$1,368,000	Excavate 3m into hard rock along entire foundation. Also includes excavation into rock for energy dissipator	Excavation will be slow, assume no blasting, and include mechanical and final cleaning of foundation
4.3	Dental Concrete	m3	1200	$600	$720,000	Assume half of the foundation will require at least 0.3m of dental concrete for shaping and removing overhangs	Include final air water cleaning in preparation for concrete, foundation anchors, bulk concrete placement
4.4	Foundation Cleanup	m2	8000	$3.6	$28,800	Air and Water cleaning by hand	Include construction joint preparation of dental concrete
4.5	Foundation Grouting	m2	11600	$16	$185,600	Grout curtain extend to a depth equal to height of reservoir, one row of vertical holes and one row of angled holes, secondary/tertiary holes where needed	Assume down stage grouting using 2 rigs, 11600m2/100m2/hole= 116 holes @ 6 holes/day=20days. Drills 20dx2x$250/hrx12hrs=$120,000+Cement 116holesx10bagx 25kg+waste @15%=34tx$500=$17,000+Grout plant 20dx10hrx$150=$30,000+consumeables=$20,000>>>>Total $187,000/11600=
			Item 4 Costs		$2,326,400		
5	**RCC and Spillway**						
5.1	Establish crushing and screening plant	LS	1	$200,000	$200,000	Assume screening aggregate, including sand, for production. Will need lots of sand	Mob and set up plant including water supply, power, sediment ponds and carry out trial production
5.2	Establish RCC production line	LS	1	$300,000	$300,000	Includes conveyor, hopper, and other plant associated with RCC production	Purchase or hire in pug mill, set up and trial
5.3	Produce and Place RCC	m3	101150	$250	$25,287,500	Assume blasting and screening required for all aggregate, cement is imported, 20MPa RCC. Dam will include gallery, drainage conduit, one low level outlet, overtopping spillway,bedding grout treatment between each lift. Do not have a mix design so difficult to estimate quantities of each material but will be cement heavy mix	RCC mix coares agg 1150kg, sand 850kg., cement 220kg produce at $200/m3.$40/m3 to place+$10/m3 grout enriched joint=$250/m3
5.4	Produce and Place Formed Facing Concrete	m3	17000	$1,000	$17,000,000	Pricing includes transport, placement and compaction. 25MPa concrete with reinforcing	Jump formed concrete at $1000/m3
5.5	Produce and Place Formed Spillway Walls	m3	240	$1,500	$360,000	40 MPa reinforced concrete, 4m high walls on downstream face (1H:1V) of dam	Formed reinforced concrete
5.6	Produce and Place Formed Ogee Spillway Crest	m3	150	$2,000	$300,000	40 MPa reinforced concrete, detailed curvature	As above
5.7	Produce and Place Spillway Dissipation Structure	m3	1800	$1,500	$2,700,000	40 MPa reinforced concrete, quantity includes 7m tall walls, ~50 chute blocks at inlet, ~20 baffle blocks at outlet	
5.8	Produce and Place Bedding Mortar	m3					included in 5.3 above
5.9	Produce and Place Formed Intake Tower	m3	240	$2,000	$480,000	40 MPa reinforced concrete, dowel into dam (connect tower to dam face), keep interior open	
5.10	Produce and Place Riprap Downstream of Dissipation Structure	m3	500	$120	$60,000	Assume D50 equal to 0.3m, can come from onsite quarry	Rock size may need to be larger
			Item 5 Costs		$46,687,500		
6	**Offtake Structure**						
6.1	Supply and Install 2m Diamter Conduit through dam	LS	1	$896,000	$896,000	Concrete encased high pressure rated steel pipe, length is 34m to daylight at toe, additional 78m to discharge into river	Allow 112m @$8,000
6.2	Supply and Install 3 Trashracks on upstram face of Tower	EA	3	$110,000	$330,000	Cover 1mx2m gates	Assume galv or epoxy coated
6.3	Supply and Install 3 gates	EA	3	$30,000	$90,000	1mx2m wide sealing slide gates, including rails anchored to intake tower and connection cable	Nominal figures
6.4	Supply and install 1 gate	EA	1	$50,000	$50,000	2mx2m wide sealing slide gate, including rails anchored to intake tower and connection cable	
6.5	Supply and install instrumentation panel and controls	LS	1	$40,000	$40,000	Control for 4 hydraulic gates on site, incorporation of site-wide instrumentation allowing for real-time monitoring and control	
6.6	Supply and install Remote operating system	LS	1	$60,000	$60,000	Radio or cellular equipment to remotely control 4 offtake gates and remotely monitor site	
6.7	Backup Power Supplies	LS	1	$100,000	$100,000	Backup generator(s) to control gates in case of power loss	
6.8	Supply and install/construct control house	LS	1	$48,000	$48,000	Building on top of intake tower to house the electrical and controls equipment, maybe prefabricated	Assume 40m2 at $1200/m2
6.9	Supply and install gate hoist	LS	1	$100,000	$100,000	Framing, hydraulics, motors, and associated equipment required to raise and lower gates	Nominal figures
6.10	Supply and install electrical gear	LS	1	$80,000	$80,000	3 phase power to control house, medium voltage motor control center for gates, low voltage connections for lights, etc.	
			Item 6 Costs		$1,794,000		
7	**Drainage**						
7.1	Supply and Install Internal Drainage System	LS	1	$900,000	$900,000	Drain holes are spaced 5m apart, extend to a depth of 2/3 grout curtain depth into rock and up to El. About 560 in dam. Drainage gallery extends full length of dam, assume at El 545 (and up to 550 at abutments) and offset from upstream face by 8m (drainage gallery construction likely to be accounted for in RCC placement costs)	Assume 60 holes at 30m average = 1800lm @ $500m=
7.2	Install Waterstops	EA	25	$1,000	$25,000	Waterstops are grouted into foundation and terminate at the top of the dam. Assume spaced at 6m on center, 55lm, PVC	Allow provisional sum $1,000/ea
			Item 7 Costs		$925,000		
8	**Demolition**						
8.1	Demolish existing dam to El. 545	m3	160200	$5	$801,000	Demolition consists of rockfill and concrete panels (concrete makes up about 2% of total demolition)	Assume side cast into existing reservoir dead storage, possible dozer push
8.2	Demolish existing power plant and substation	LS	1	$20,000	$20,000	Remove structure, no reuse of material all to waste	
			Item 8 Costs		$821,000		
9	**Instrumentation**						
9.1	Piezometer	EA	28	$3,500	$98,000	vibrating weir piezometers at crest and downstream slope, remote readings	
9.2	Seepage Weirs	EA	7	$500	$3,500	Most exit into drainage gallery	
9.3	Stuctural Monitoring Points	EA	18	$500	$9,000	Permanent monument that can be surveyed	
9.4	Early Warning System	LS	1	$30,000	$30,000	Includes alarms downstream, water level sensors upstream and downstream, water level sensor of reservoir level, and all will be remote system	Allow provisional sum
			Item 9 Costs		$140,500		
				BCS Cost	$ 62,146,400		
				DCS Cost	$ 77,061,536	BCS+1+2	
11	**Uncosted Items**						
11.1	Uncosted Items	35% of DCS			$ 26,971,538		
			Preliminary Project Cost		$ 104,033,074	DCS+Uncosted Items	

APPENDIX F

Preliminary Design Drawings

MANUHERIKIA CATCHMENT FEASIBILITY STUDY

FALLS DAM RAISE OPTIONS

REFERENCES

1. EXISTING GROUND SURVEY WAS PROVIDED BY BTW SOUTH ON 27 JUNE 2014. COORDINATE SYSTEM, SURVEY, AND ALL ELEVATIONS ARE IN REFERENCE TO LINZ LISTED TRIG N No 2 AND DUNEDIN VERTICAL DATUM 1958.

SITE PHOTO
N.T.S.

INDEX OF FIGURES

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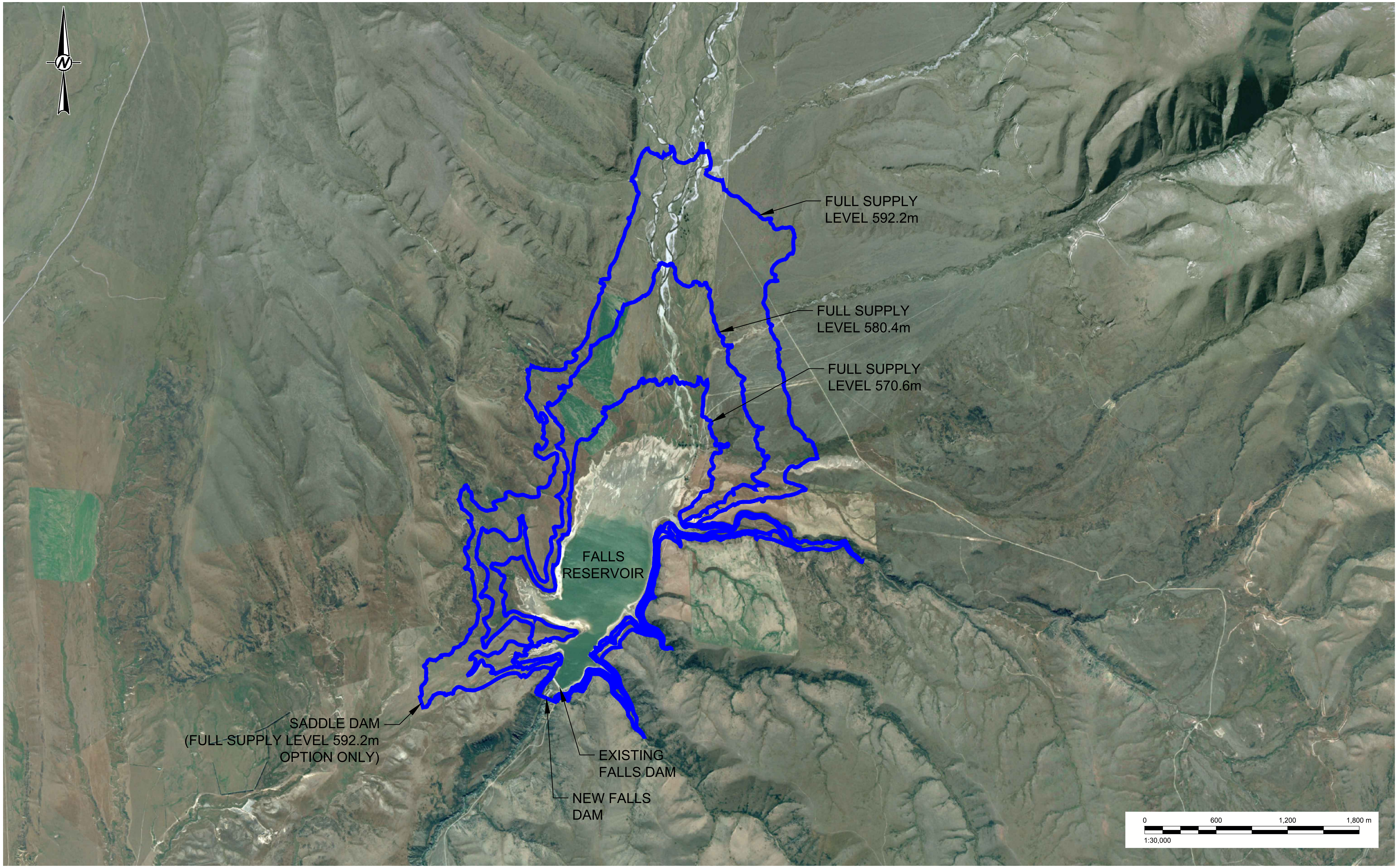
CLIENT	PROJECT
MANUHERIKIA CATCHMENT WATER STRATEGY GROUP	MANUHERIKIA CATCHMENT FEASIBILITY STUDY
CONSULTANT	TITLE
Golder Associates	**TITLE SHEET AND SITE MAP**

PROJECT No.	Rev.	Figure
1378110270	A	1



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CLIENT MANUHERIKIA CATCHMENT WATER STRATEGY GROUP	PROJECT MANUHERIKIA CATCHMENT FEASIBILITY STUDY
CONSULTANT Golder Associates	TITLE **RESERVOIR AREA**
	PROJECT No. 1378110270 · Rev. A · Figure 2

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TITLE
PLAN VIEW OF FULL SUPPLY LEVEL 570.6 m DAM RAISE OPTION

PROJECT No.	Rev.	Figure
1378110270	A	3

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MANUHERIKIA CATCHMENT FEASIBILITY STUDY

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TITLE
PLAN VIEW OF FULL SUPPLY LEVEL 580.4 m DAM RAISE OPTION

PROJECT No.	Rev.	Figure
1378110270	A	4

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MANUHERIKIA CATCHMENT FEASIBILITY STUDY

CONSULTANT
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TITLE
PLAN VIEW OF FULL SUPPLY LEVEL 592.2 m DAM RAISE OPTION

PROJECT No. 1378110270
Rev. A
Figure 5

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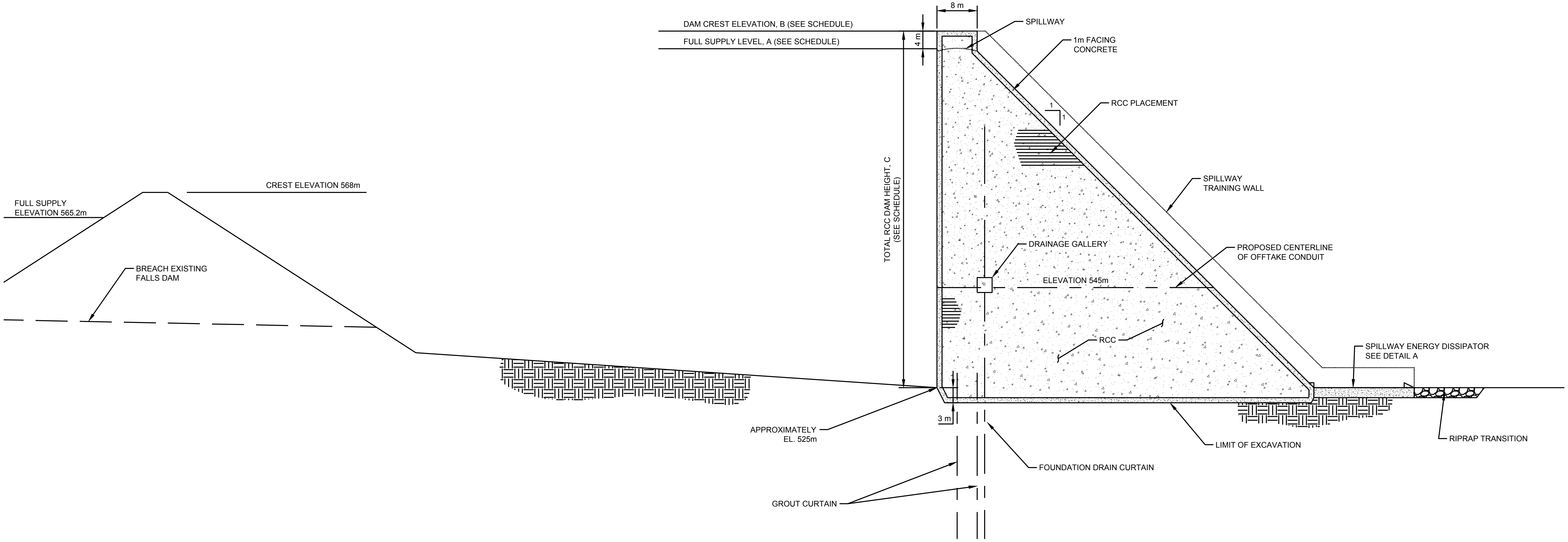
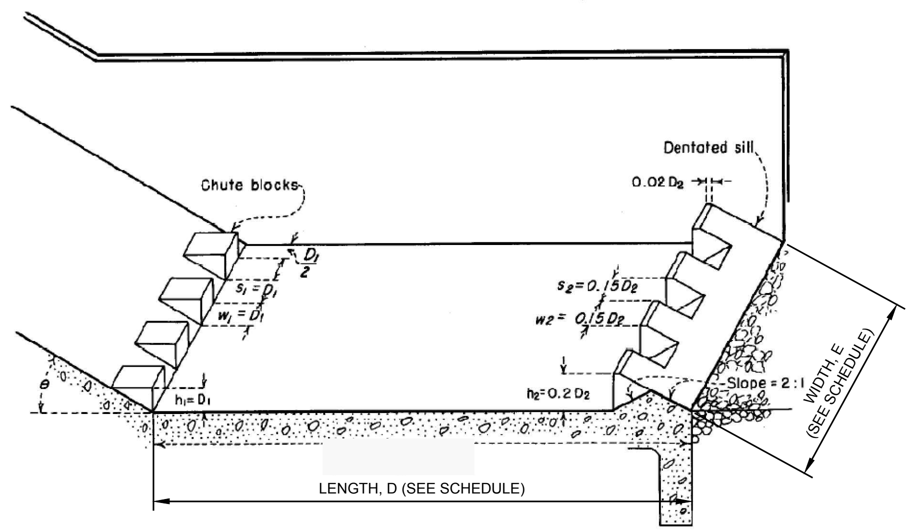


1 TYPICAL MAXIMUM CROSS SECTION OF NEW RCC DAM

0 16 32 48 m

1:800



NOT TO SCALE A ENERGY DISSIPATOR TYPICAL DETAIL (USBR TYPE II)

NEW FALLS RCC DAM SCHEDULE

FULL SUPPLY LEVEL, A	DAM CREST ELEVATION, B	TOTAL DAM HEIGHT, C	ENERGY DISSIPATOR LENGTH, D	ENERGY DISSIPATOR WIDTH, E
570.6m	574.6m	49.6m	30m	50m
580.4m	584.4m	59.4m	30m	42m
592.2m	596.2m	71.2m	30m	35m

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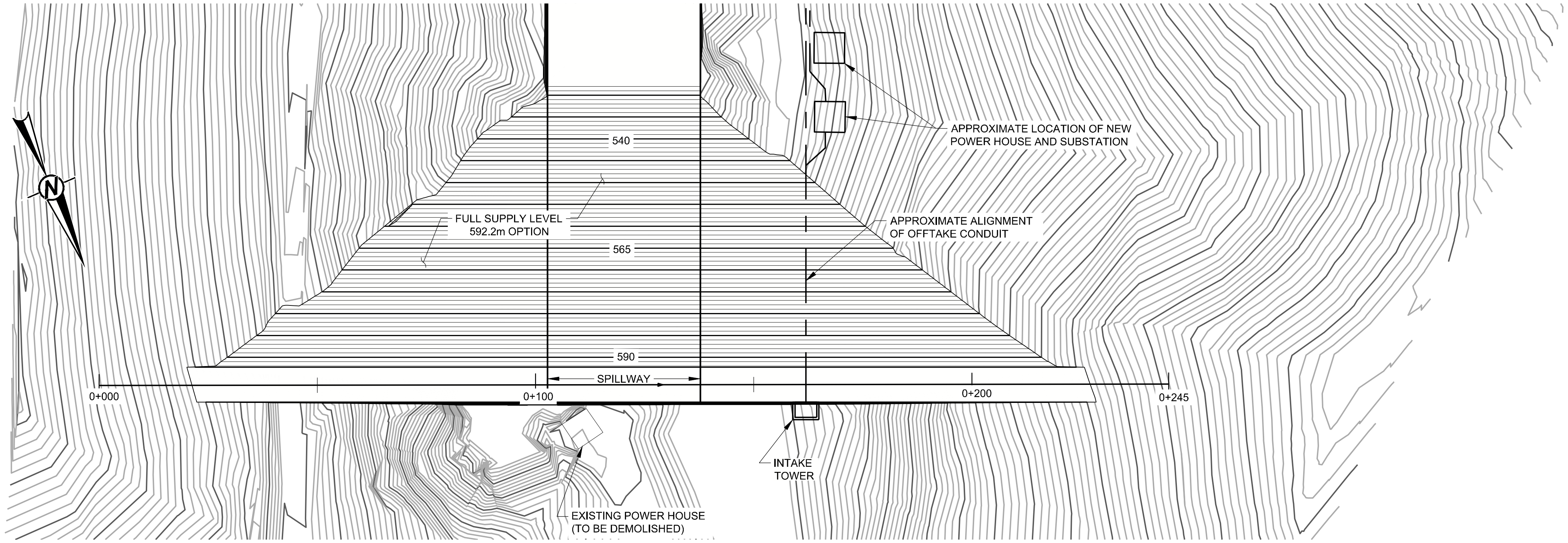
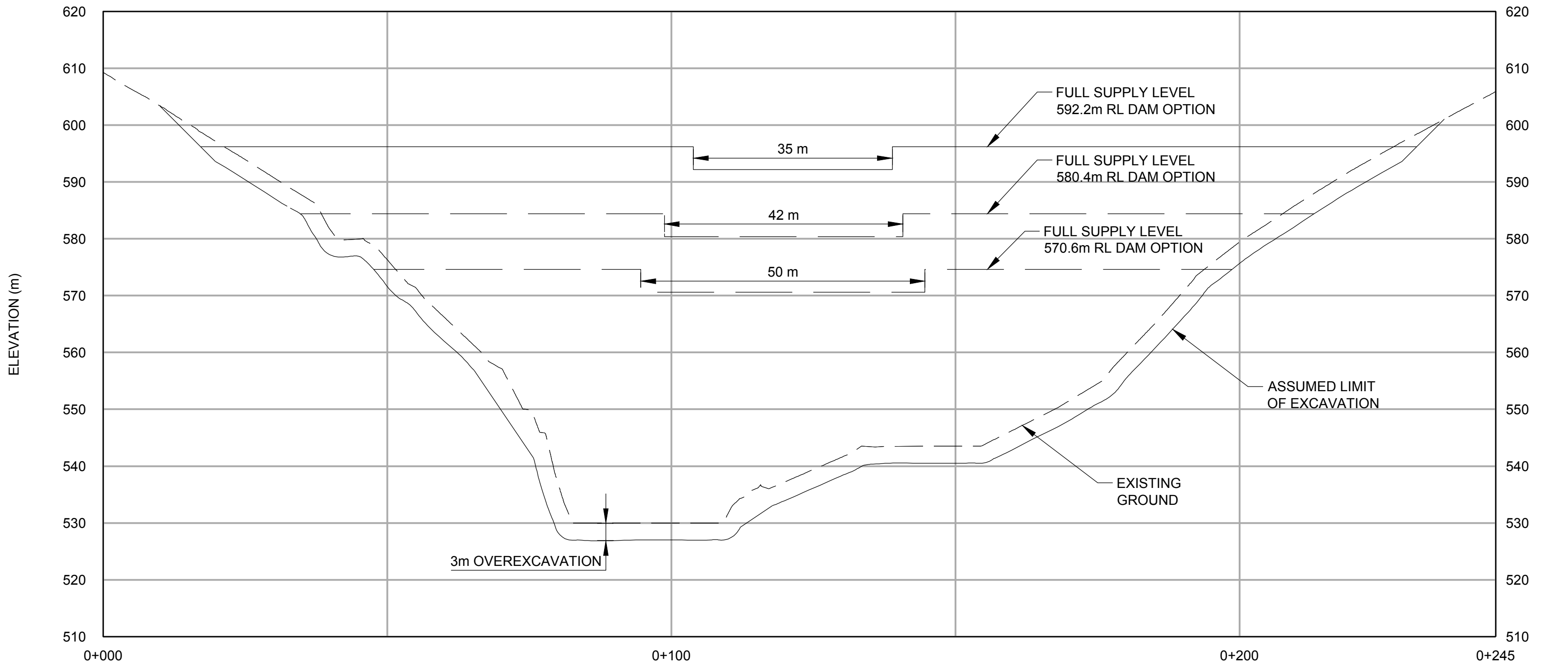
TITLE
TYPICAL MAXIMUM CROSS SECTION OF NEW RCC DAM

PROJECT No. 1378110270 | Rev. A | Figure 6



0 25 50 75 m

1:1,250



1 PROFILE ALONG DAM CREST

0 25 50 75 m

1:1,250

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TITLE
PROFILE ALONG DAM CREST

PROJECT No.	Rev.	Figure
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25 mm

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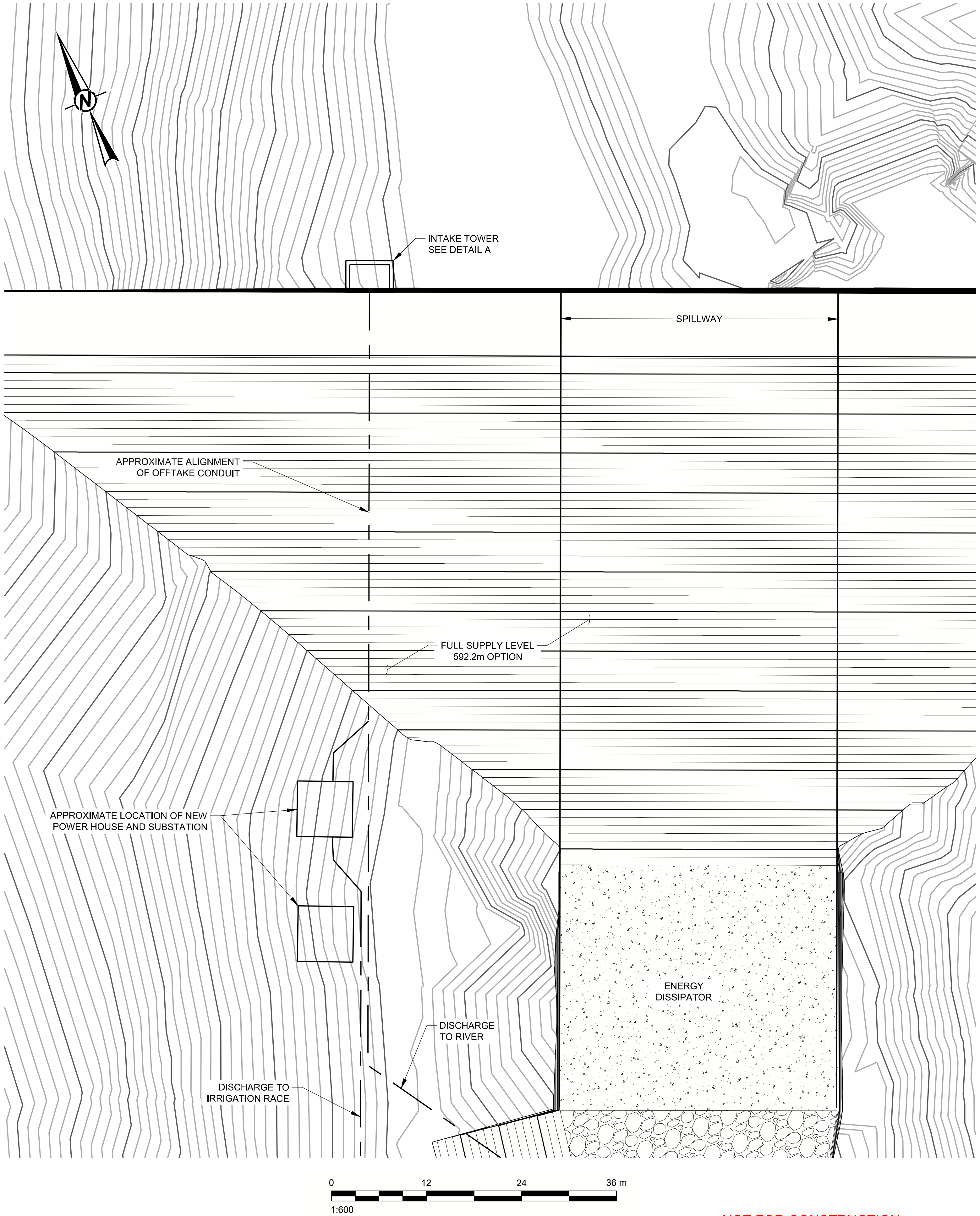

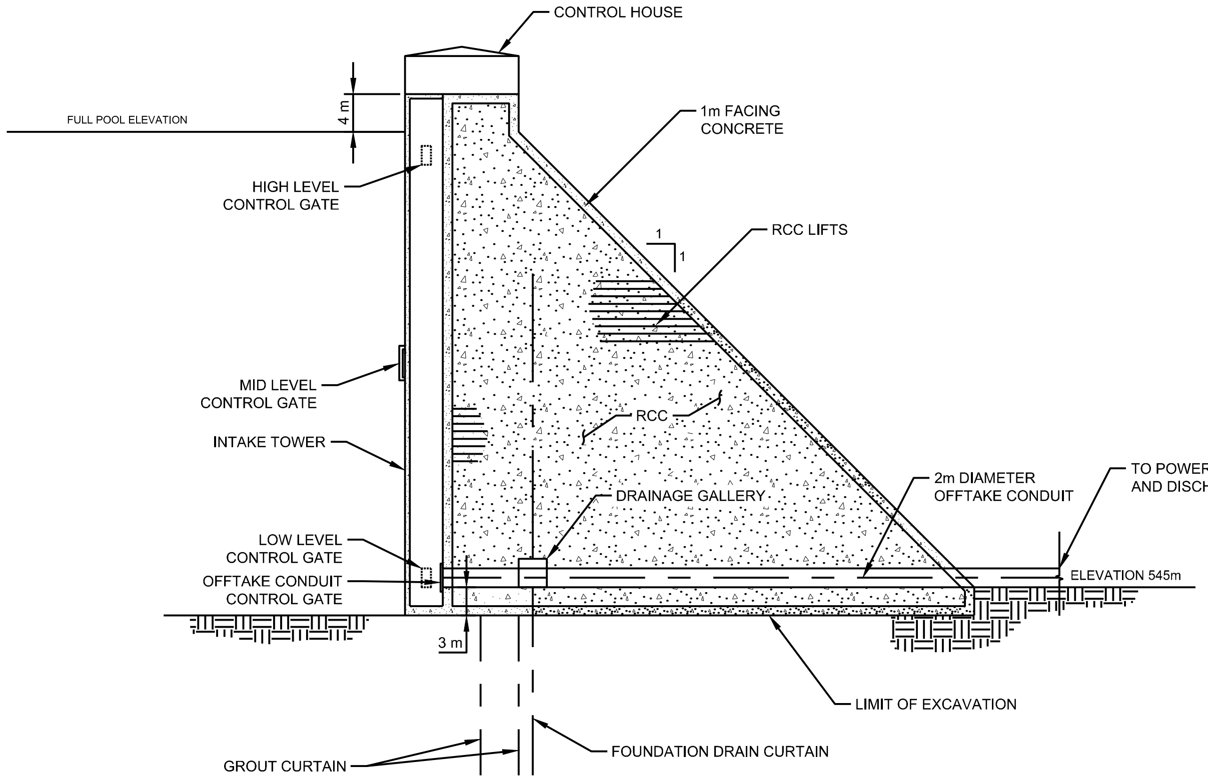

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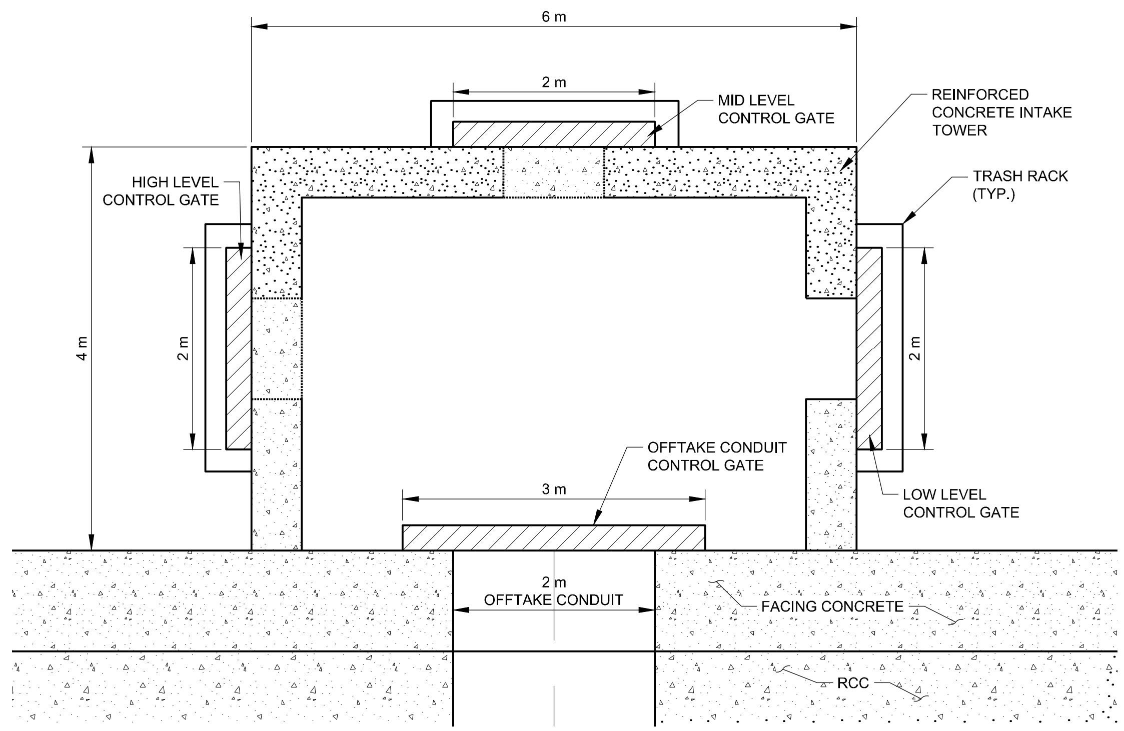

NOT TO SCALE A INTAKE TOWER DETAIL

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	PROJECT No. 1378110270 · Rev. A · Figure 8

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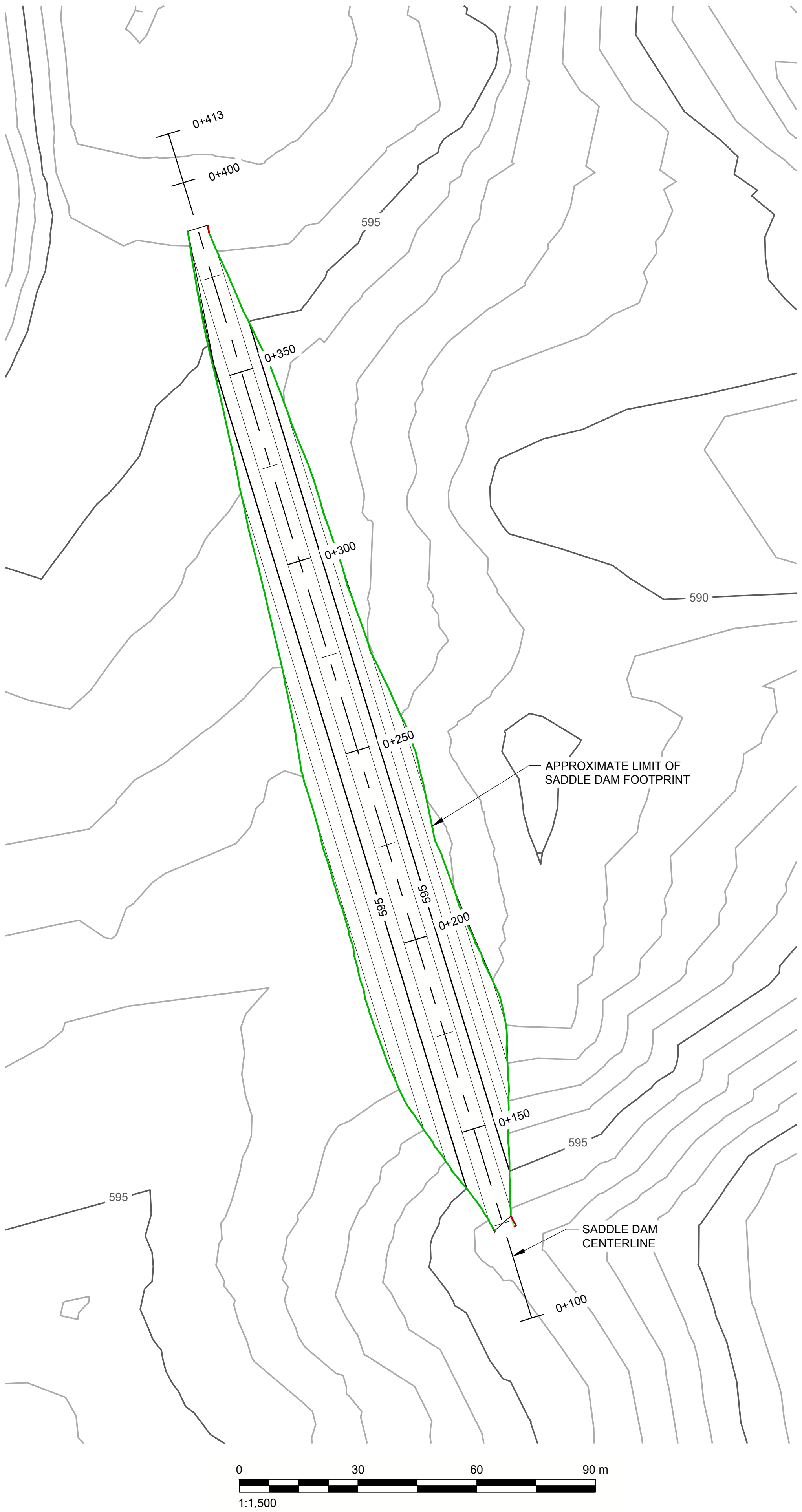
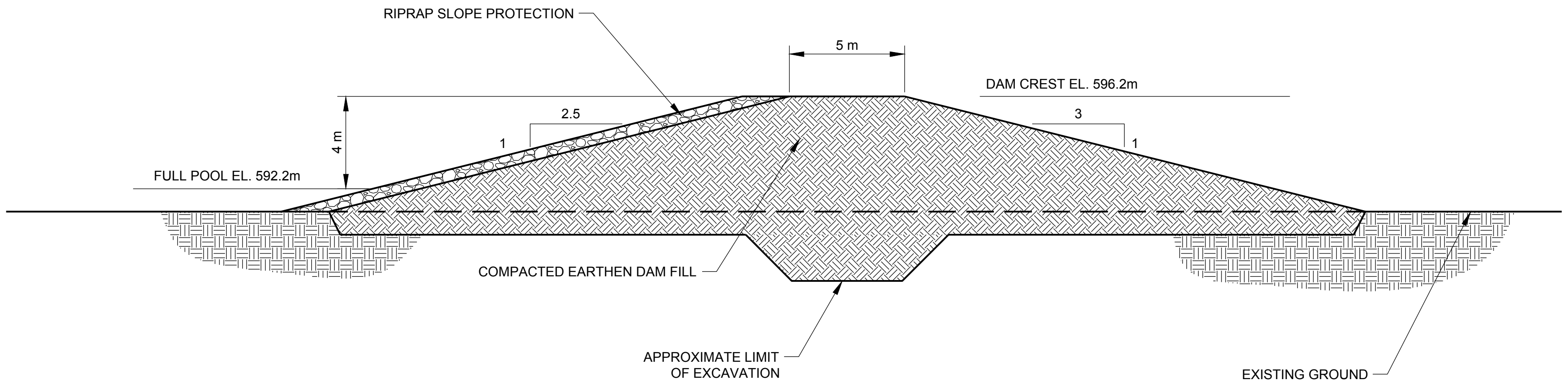
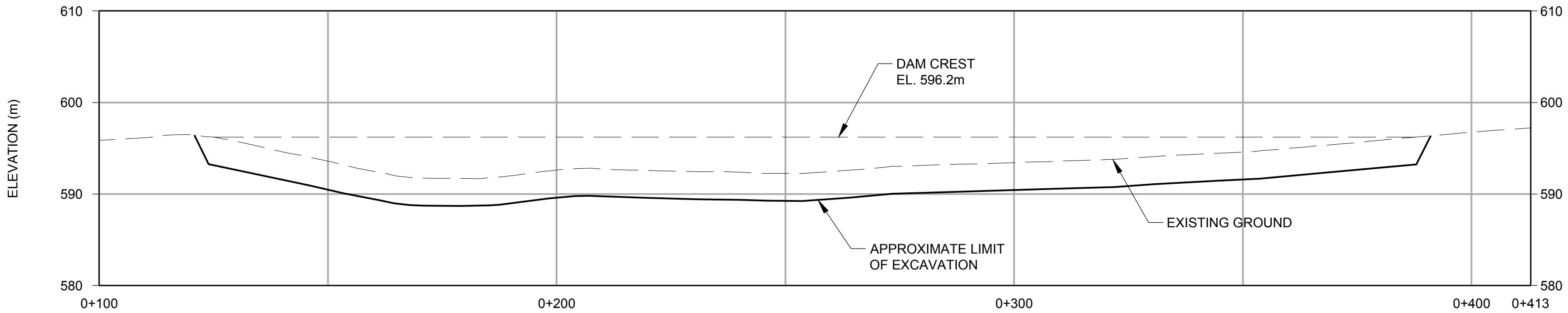




NOT TO SCALE 1 SADDLE DAM - MAXIMUM SECTION



2X VERTICAL EXAGGERATION 2 SADDLE DAM PROFILE

0 30 60 90 m

1:1,500

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TITLE
SADDLE DAM PLAN VIEW AND MAXIMUM SECTION

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TITLE
PROPOSED CONSTRUCTION LAYOUT

PROJECT No.	Rev.	Figure
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