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Manuherikia Catchment Water Strategy Group
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MANUHERIKIA: FALLS DAM RECOMMENDED OPTION

Dear Kate and Allan

Background

We are entering a critical phase in the project which involves completing a preliminary dam design for a raise at Falls Dam. Based on the available options of either raising the existing concrete faced rockfill dam or to construct a new roller compacted concrete dam downstream, our recommended option is a Roller Compacted Concrete (RCC) Dam capable of storing 50 M m^3 i.e., an approximately 15 m raise of the existing full storage level. The rationale supporting this recommendation is described in this letter[1].

To ensure we are aligned regarding the next steps and expectations, we offer the following summary and are seeking confirmation that this is our common understanding. The reason for doing this is that:

- Golder Associates (NZ) Limited (Golder) is contracted to develop feasibility assessments for the raised Falls Dam options (low, medium and high).
- MCSWG needs to provide meaningful options to allow landowners to reach a cost-benefit decision.
- A *preliminary dam design* requires stability assessments (both static and seismic), leakage assessments, preliminary spillway and offtake design, grout curtain assessments, quantity estimates, construction methodology, and preliminary design drawings.
- Golder did not envisage completing preliminary level designs for 3 alternatives at Falls Dam (nor do we believe Manuherikia Catchment Water Strategy Group (MCWSG) did to the level described above). In our Scope Summary and Clarification document dated 28 March 2014 which was discussed with MCWSG it states:

  "Design Criteria and Assessment of Dam Types: ***Deliverable*** (G5) – Report section discussing results of the assessment and recommending a dam type to carry through to preliminary design. *Note that the Scope is for a single (preferred) option."*
- Golder therefore proposes a preliminary dam design for the 15 m raise and comparative designs and cost estimates for the 6 m and 27 m raises. The comparative designs will be based on the preliminary 15 m dam design but scaled up or down to the additional heights.

[1] This letter is provided subject to the limitations in Attachment A.

Purpose

This letter provides a comprehensive description of the geotechnical, engineering and construction issues relating to the selection of a dam type to accommodate increased storage at Falls Dam. The purpose of this letter is to provide an updated understanding such that decisions can be made to reduce the number of available options to allow more focus on the selected dam that will progress further.

Summary

Work undertaken by Golder as part of the current study has included assessments of seismic hazard, foundation conditions, dam break effects and dam stability. Key findings include:

- The seismic hazard at Falls Dam site is high due to the close proximity of the active Blue Lake Fault. Design ground motions for the dam are substantially higher than previously assumed. This assessment is summarised in the Stage 1 Geotechnical and Engineering Report (Golder 2014a).
- Foundation conditions at Falls Dam are good and suitable for the concrete faced rockfill dam (CFRD) and RCC options under consideration (Golder 2014a).
- The 27 m raise of the reservoir (assuming an RCC dam) is confirmed to be a high potential impact category (PIC) dam (Golder 2014b). Smaller RCC options are also likely to be high PIC. It is likely that the existing dam is at least Medium PIC and that any raised CFRD option would be high PIC.
- Preliminary stability analysis indicates that, a new RCC dam that achieves a 15 m raise of the reservoir is likely to meet stability criteria with slopes of approximately 1H:1V downstream and a vertical to near vertical upstream face. However, a new RCC dam that achieves a 27 m raise of the reservoir is likely to require flatter slopes and additional design measures, such as anchoring, to meet stability criteria.
- New survey data indicates that the 27 m reservoir raise will require construction of a substantial saddle dam, while the 15 m raise option will not require a saddle dam. Evaluation of foundation conditions at the saddle dam site indicates poor foundation conditions and consequently a high cost is expected for the saddle dam.

Irrespective of whether the full reservoir storage level is increased by 6 m, 15 m or 27 m, a new RCC dam is the preferred dam type to progress to feasibility level design due to several factors summarised below and discussed more fully in subsequent sections:

- The existing reservoir can stay full during construction.
- The existing dam can be used as a coffer dam during construction.
- RCC has a rapid construction time.
- A new dam does not have to address the deficiencies of the existing dam.
- A 15 m RCC dam would likely be able to accommodate the anticipated high seismic ground motions without requiring a very wide cross section (and consequent increased cost).

A NEW CFRD may provide greater seismic stability and be more cost effective than an RCC dam. However, this option is not currently being moved forward due to having been ruled out previously (OPUS 2012) and being outside the current contracted scope of work. If an RCC dam is determined to be too costly or not stable in future analyses, a new CFRD embankment could be considered.

Proposed Reservoir Storage Volumes

The current feasibility study is assessing the suitability of increasing the storage at Falls Dam by either raising the CFRD with additional rockfill or constructing a new RCC dam downstream. The three proposed storage volumes are presented in Table 1 below.

Table 1: Proposed Supply Levels for Modified Falls Dam.

Raise Height	Fully Supply Level (m)	Approximate Dam Height (m)	Estimated Storage (Mm^3)
Current Condition	RL561.4 (565.2)	33.5	10
6 m	RL567.4 (571.2)	39.5	20
15.6 m	RL577 (580.8)	49.1	50
26.6 m	RL588 (591.8)	60.1	100

Notes: Values taken form pre-feasibility investigations Opus 2012 and Aqualinc 2012c. Levels provided in brackets are based on recent survey data.

Existing Falls Dam

The current CFRD Falls Dam, constructed in the 1930's, is approximately 33.5 metres (m) high with a crest length of approximately 155 m. The dam comprises a relatively homogenous embankment consisting of dumped rockfill quarried around the dam site with 1.5H:1V downstream slopes and 1.25H:1V upstream slopes down to elevation RL539.5 at which point the upstream slope becomes 1.5H:1V. The upstream slope of the dam consists of a concrete liner underlain by one meter thick layer of rock in mortar underlain by 3.0 m of packed rock. The upstream toe consists of a 14 m wide section of rock in mortar and a 3.0 m wide section of packed rock. The concrete liner consists of 44 feet square (13.4 m^2) concrete panels ranging from 15 inches (0.4 m) thick near the crest of the dam to 24 inches (0.6 m) thick near the toe of the dam. The slabs are reinforced with 1-inch (0.025 m) diameter rods placed in a mat format with rods placed from 12 inches (0.3 m) to 18 inches (0.45 m) on centre. The facing was made watertight by placing copper strips between the panels. A cutoff wall was constructed at the upstream toe which ties the upstream concrete liner into the bedrock. The cutoff wall was reportedly excavated down to solid rock which was located at depths of up to 6 m. The width of the cutoff wall was limited to about 0.6 m when the depth was relatively shallow and upwards of 1.5 m when the cutoff wall extended to deeper depths (Gilkison 1937).

Grouting was completed beneath the cutoff wall. Three meter deep grout holes spaced at 0.6 m on centre to within 26 m of the dam crest and then spaced at 1.2 m on centre were drilled prior to concrete placement in the cutoff wall. The holes were grouted after the cutoff wall was backfilled with concrete. The 196 grout holes took 222 bags of cement, an average of 1.13 bags per hole. The five worst holes took 87 bags of cement. Additional water pipes, spaced at 1.2 m on centre, were brought up during construction of the concrete cutoff wall and were cored out at to a depth of 12 m deep and also grouted. Difficulty in drilling of the weathered bedrock was encountered and some holes were abandoned due to collapsing holes and stuck equipment. Some areas of broken rock were grouted and then cored to reach the maximum depth which resulted in a slow and expensive grouting program (Gilkison 1937).

The morning glory spillway exits through the original construction stream diversion tunnel through the left abutment. The entry lip of the spillway was raised 0.61 m in 1955 to improve storage capacity and the 2012 OPUS study has reported that the existing spillway has a discharge capacity of approximately 430 m^3/sec, which is when choking begins. The spillway entry lip is at elevation RL561.4.

The original offtake works were modified in 2003 during the construction of the hydropower scheme. The original offtake consisted of a 33-inch (0.8 m) diameter cast iron pipe through the concrete bulkhead into the valve chamber. This original pipe was coupled with a 1100NB steel pipe leading to the new hydropower plant. The new 1100NB steel pipe was placed in a trench constructed into the bottom of the original access adit. An additional 1200NB steel pipe was laid over the crest of the dam which intersects with the 1100NB pipe where they both then discharge into a 1400NB steel penstock pipe near the powerhouse (OPUS 2012).

Current Condition of Falls Dam

Sporadic visual inspections have been performed at the dam and some instrumentation data has been recorded. An assessment of the current condition of the dam has been made based on this information along with site visits, site investigations, and other pertinent design and construction reports.

The 1937 construction report had some information on crest settlement immediately after construction and during first filling. During the first 6 months after construction, and before the reservoir was filled, settlement at the crest was measured to be 0.68 feet (0.2 m). After first filling, 11 months after construction completion, settlement was measured at 1.19 feet (0.36 m). The total settlement was projected to be about 1.56 feet (0.48 m) at the dam crest. The dam also had some downstream movement, measured to be 0.48 feet (0.15 m) 6 months after construction completion and 0.87 feet (0.27 m) 11 months after construction completion and first filling. The total project movement was 1.25 feet (0.38 m) (Gilkison 1937). The 1937 (Gilkison) design report indicates that this projected settlement is less than the crest camber. If the projected 1.56 feet (0.48 m) of settlement is accurate, the total settlement would be about 1.5 percent of the total height which is an acceptable amount of settlement. An attempt to locate additional settlement data was unsuccessful and as a result the actual total settlements are unknown.

A 1984 sedimentation report estimated that there was a total volume of 610,000 m^3 of sediment in Falls Dam reservoir (Bishop et al. 1984). The sediment is mostly silty clay and clay and the deposit is thickest at the upstream toe of the dam. The estimated accumulation rate is about 10-15 mm a year (Bishop et al. 1984). This current volume of sediment at the site has likely increased since 1984 which will impact the storage capacity of the reservoir. The reservoir may be dredged to remove the sediment if the loss of capacity is determined to be unacceptable. The sediment at the upstream toe may also prove to be an obstacle for repairing or replacing the upstream liner in the future.

A visual inspection of the entire spillway was performed in 1984 by the Ministry of Works and Development (Richards 1985). The inspection report noted areas of spalling, exposed steel, and deteriorated concrete along the interior panels of the morning glory spillway. Repairs were recommended to the most deteriorated panels and joints but also noted that all poor quality concrete should be repaired. The report also noted that the nosing cone of the needle valve required immediate replacement. The response from the Alexandra residency indicated that repairs had to be postponed until October / November 1985 when the reservoir was expected to be lower (Richards 1985). The inspection team also performed a cursory inspection of the concrete liner. The liner was in “sound” condition but noted that there is general deterioration of the concrete below the normal water level with exposure of aggregate, pitting, horizontal cracking, and erosion of concrete around the cracks. A follow up inspection was recommended as well as repairing the pitted and deteriorated concrete liner (Richards 1985). Some local repairs of the concrete joints in the spillway were completed but the majority of the deteriorated concrete was not repaired (OPUS 2012). The needle valve was not replaced and is still in need of repair.

The 2012 OPUS report noted that a 1989 serviceability report recommended six repairs which consisted of:

- Control valve and associated equipment
- Leakage measuring weir
- Access tracks
- Facing panels and joint deterioration
- Spillway crest control
- Repair of tunnel lining concrete.

The first three items in the list were completed during the installation of the hydropower scheme in 2003 but the rest of the items are outstanding (OPUS 2012).

The 2012 OPUS report also noted that in the 1980’s a significant seepage area was observed at the dam which was traced back to a membrane joint. The joint was temporarily repaired by placing butynol sheeting over the joint and securing it with steel angles. Permanent repairs were never implemented (OPUS 2012).

Aqualinc (2012a) reported seepage weir readings between May 2011 and 15 July 2012. The weir collects water from a known seepage location at the right abutment toe. The water is collected in a pipe and discharged at the v-notch weir outside of the power house. The weir readings are recorded as depths over the v-notch weir and do not exactly correlate to flow rate but as the flow depth increases, the flow rate increases exponentially.

The spillway spilled approximately three times between May 2011 and 15 July 2012 and each time there was a significant spike in the seepage weir readings, increasing from about 90 mm (depth over the weir) to approximately 725 mm in May 2011, approximately 800 mm in October 2011 and approximately 425 mm in

March 2012 (Aqualinc 2012a). Based on personal communication with Omakau Area Irrigation Scheme, the spike in readings is due to the v-notch weir getting flooded by water backing up the river when the dam is spilling (Williams 2014). There is still likely a small spike in the weir flows when the spillway is flowing but not as significant as the readings indicate.

There are a few smaller spikes in weir readings when there are sudden increases in the reservoir water elevation. These small spikes may be due to rainfall or they may indicate the presence of a connection between the reservoir, either through an embankment or foundation flaw, and the seepage collection system. Omakau Area Irrigation Scheme also noted that after the 2011 Christchurch earthquakes, the seepage area migrated towards the left abutment. The known seepage area at the right abutment toe dried up and a wet area was observed and running water was heard near the left abutment toe. This seepage dried up over time and the seepage migrated back towards the original seepage area at the right abutment toe (Williams 2014). The migration of seepage areas likely indicates the dam is still susceptible to movements and settlements.

Golder undertook site visits to Falls Dam in 2013 and 2014 and was also supplied with a series of photographs taken in 2010 when the reservoir was low. The site visits included walking around the dam and documenting the condition through photographs. The observations of the dam crest indicate some settlement and downstream movement (offset and crooked handrails and distortion of the facing panels). There is also deterioration, cracking, pitting, and erosion of the concrete liner, especially between the panel joints. The seals over the joints are ripped and no longer functioning as designed. The concrete above the normal water surface elevation also appears to be spalling and deteriorated possibly due to freeze thaw impacts. The coping wall on the dam crest is severely cracked and broken compromising the design width. The spillway concrete also appears to be deteriorated with slaking and spalling of the concrete with some exposed steel near the intake. The concrete joints in the spillway appear to be open and the concrete appears deteriorated. There also appears to be seepage through the spillway neck as water was observed entering the spillway through joints as the reservoir level was below the spillway intake elevation. There is no known documentation of the observed condition of the plinth at the upstream toe.

Options for Increasing Reservoir Storage

Raising the existing Concrete Faced Rockfill Dam

One option being considered during the current study for increasing the reservoir storage is to raise the existing Falls Dam between 6 and 27 meters. The proposed concept would consist of a downstream raise by adding additional rockfill to the downstream slope and the crest would be shifted downstream to accommodate the raised crest elevation. The upstream slope would be protected by a water tight barrier possibly consisting of concrete, asphaltic concrete, or a geomembrane. Before the raise can be completed, maintenance and modifications at the existing dam should be addressed.

Potential Repairs to the Existing Falls Dam

The upstream concrete liner has been in operation for over 70 years without any substantial repairs or replacements. There are also no known observation reports on the condition of the concrete plinth at the upstream toe. The condition of the connection between the upstream liner and plinth is also unknown and any deficiencies that currently exist will likely be magnified by increasing the full supply level of the reservoir. The current condition of the upper portion of liner based on visual observations is deteriorated, especially at the joints between the concrete panels. These joints may not be critical during the current operation of the dam but if the reservoir is raised these joints would be submerged under a larger head and for longer durations. As demonstrated in the 1980s, openings in the concrete liner result in seepage through the embankment. The spalling, pitting, erosion, exposed steel, and cracking of the concrete liner panels are compromising the strength and water barrier properties of the liner. Larger settlements or reservoir loads than what was designed for may result in a rupture of the liner allowing water to freely exit between the reservoir and downstream slope. The liner will likely need extensive repairs or replacement before a dam raise could be constructed.

The rockfill portion of the dam was constructed by dumping rock into place rather than the current conventional method of compaction in layers.

The currently accepted method of construction of CFRD embankments recommends compaction of the rockfill and recommends upstream and downstream slopes ranging from 1.3H:1V to 1.4H:1V (USBR 1987).

The main reason for compaction is reducing settlement of the rockfill as excessive settlement may lead to rupture of the upstream liner. Settlement of rockfill may occur due to increasing heights, application of additional water, and due to earthquakes. The existing rockfill dam may experience additional settlements due to any of these reasons and the existing upstream liner may need modifications to account for these settlements and potential damage to the liner.

Construction alternatives for repairing the upstream liner may include individual repairs to the concrete liner with new seals over joints, epoxy filling of cracks, or replacing concrete in areas. Repairs of this type will require the reservoir to be drawn down to access all areas that need repair. Replacement of the existing liner is also an option for repair. The existing concrete panels can be removed and replaced with either concrete or asphaltic concrete which would require a drawdown of the reservoir but would provide a more complete and robust repair. The reservoir would likely need to be drawn down for a few months under ideal conditions to repair the liner, but the drawdown may extend for many months depending on the condition of the liner and the required repairs. A geomembrane may also be placed over the existing concrete panels. The geomembrane could provide the water barrier that the deteriorated concrete liner can no longer reliably supply and the geomembrane can be placed in the wet or the dry. Placing the geomembrane in the dry would allow for thorough inspection and cleaning of the existing concrete liner and plinth, allow for better connection between the new geomembrane and plinth, and allow for testing of the seal between the geomembrane panels. However, the reservoir would have to be drained to allow for this construction. The geomembrane could be place in the wet with divers doing the installation but inspection of the plinth would be limited and the seal between the geomembrane and plinth would be more difficult to construct. Also, the sedimentation at the upstream toe may need to be removed before installation of the liner which may be difficult and costly. Based on personal conversations with Carpi, a manufacturer of geomembranes, the cost for installation of the geomembrane underwater is many times higher than if it was completed in the dry (2014).

The prevention of seepage beneath a rockfill dam and the watertight seal between the upstream concrete liner and the foundation is critical to the performance of a rockfill dam (USBR 1987). The grout curtain is intended to prevent seepage beneath the rockfill dam. The current grout curtain has performed adequately for the current reservoir operations but the performance of the grout curtain under a larger reservoir with a higher full supply level is unknown. Increasing the full supply level of the reservoir will apply higher heads on the foundation defects, potentially expose more defects to the reservoir, and could increase seepage beneath the dam. If it is determined that additional grouting is needed beneath the dam, the work will likely require the reservoir to be drained to perform drilling and grouting. It may be possible to perform grouting from a barge in the reservoir but the costs are expected to be high.

Other Design Considerations for Raised CFRD

The raised CFRD will likely require a relatively large spillway cut into the left abutment regardless of the raise height. The sizing of the spillway will be based on the inflow design flood (IDF) which will be based on the potential impact category of the dam. The spillway can be constructed with a full reservoir.

The current configuration of the hydropower scheme and offtake structures will likely need to be modified to accommodate a dam raise. Based on the 2012 OPUS report, the peak discharge capacity required for a 6 m raise is 4.0 m^3/s, which is the current peak capacity. The peak discharge capacity required for a 15 m raise is 6.5 m^3/s and the peak discharge capacity required for the 26 m raise is 11.0 m^3/s (OPUS 2012). The 2012 OPUS report estimated that the diameter of the required offtake pipe for the 6, 15, and 26 m raises are 1.5, 1.9, and 2.3 m, respectively. These discharges may increase based on reservoir drawdown requirements and irrigator needs downstream. The current offtake pipe has a diameter of 0.8 meters. This pipe, along with the tunnel that it is housed in, does not appear to have been inspected in the past and it is likely that this tunnel and pipe have experienced deterioration and may need replacement or repairs. The concrete bulkhead consists of low strength concrete and a more robust bulkhead will likely be required for a dam raise. The current size of the offtake pipe will likely be inadequate to accommodate the required offtake discharge flows. It may be possible to utilize the current morning glory spillway tunnel excavation and place new offtake pipes through the excavation but to have the ability to use the water stored below the current spillway invert elevation of RL561.5, the intake will have to be lowered, likely requiring excavation around the inlet upstream of the dam.

Lowering the inlet elevation of the offtake structure will also allow for lowering of the reservoir during inspection, repairs, and unexpected emergencies. The excavation required around the inlet of the tunnel will need to be done in the dry and will require lowering the reservoir down to dead storage (near empty) or a cofferdam may be constructed upstream of the excavation work but the reservoir will still likely need to be drawn down significantly to complete the work. There may be a foundation defect through the spillway tunnel, as noted in the weir readings, and an understanding of this potential flaw will be required before use of the tunnel. An alternative to using the existing spillway tunnel may be to construct a new tunnel through the left abutment to accommodate the offtake structure.

The majority of the new tunnel excavation can be completed with the reservoir full but the construction of the tunnel intake will require the reservoir to be near empty, accomplished either by emptying the reservoir or installing a cofferdam upstream of the work area but this may still require some lowering of the reservoir.

During construction of a CFRD raise, the reservoir may also need to be restricted to a predetermined elevation to allow for flood protection during construction.

Earthquake Loadings

Based on GeoNet's Quake Search website, the dam has not experienced significant shaking based on the magnitudes and distances of the nearby earthquakes. Based on our preliminary analysis the peak ground accelerations (PGA) at the site are expected to be about 1.0 g during the 10,000 year return period event. This seismic load is very high and few dams have been designed to withstand such high ground motions. Historical references indicate that even fewer dams have experienced such high ground motions (USSD 2014). Understanding additional design measures required to accommodate the high ground motions, regardless of the dam type and height, will need to be carefully understood and designed in future work.

In areas where seismic activity is expected, it is recommended that CFRD embankments be constructed with large downstream zones of compacted rockfill with downstream slopes of 1.7H:1V, which is considerably flatter than the current dam (USBR 1987), and preliminary stability analysis indicates that dam slopes may have to be substantially flatter to meet stability design criteria. The existing CFRD embankment is likely to experience additional settlement due to a large earthquake (in excess of that experienced at the site to date) and the settlements may result in severe deformations or displacement of the upstream concrete liner. The large seismic loads will have to be accounted for in the CFRD raised portion as well and the upstream liner may need to be flexible enough to accommodate significant movement so as to prevent catastrophic failure of the dam. In our opinion, the existing liner is unlikely to satisfactorily accommodate the expected deformations associated with 10,000 year return period ground shaking given the steep dam slopes and uncompacted rockfill.

RCC

A second option for increasing the reservoir storage is to build a new RCC dam downstream of the current CFRD embankment. An advantage of constructing a new dam downstream is that there is no reliance on existing infrastructure. It is our opinion that incorporating the existing CFRD into a dam raise is technically more challenging than constructing a new dam. By eliminating the reliance on the existing CFRD in the new dam design, there are fewer uncertainties with future performance.

Design Considerations for a New RCC Dam

The new RCC dam will consist of a gravity dam with an overtopping spillway which will reduce excavation quantities during construction compared to a CFRD option. The bedrock at the site is strong greywacke which is considered to be a suitable foundation and will likely be suitable as an RCC aggregate. The area just downstream of the existing dam is relatively narrow and will not require a long embankment. There will be some excavation into the underlying bedrock for the dam foundation to address stability but there will be no excavation for a spillway. The offtake structure will likely consist of a tower located upstream of the RCC structure and may allow for water extraction from different levels within the reservoir. The construction of the offtake structure may require additional excavation into the bedrock but tunnelling will likely not be required.

The current CFRD embankment and appurtenant structures will likely be used for stream diversion and flood protection during construction of the RCC dam.

Having the existing CFRD embankment upstream from the new proposed footprint of the RCC dam will likely allow for the reservoir to stay full during the duration of construction.

The RCC dam is expected to have easier construction logistics as the dam and spillway are combined into a single structure. Rapid placement of RCC often results in shorter construction durations as compared to other dam construction methods.

The RCC will require significant amounts of cement, and / or flyash if an adequate source can be identified, which may have significant impact on the costs. Protecting the RCC from potential freeze thaw will also need to be evaluated. For protection, the RCC may require conventional concrete facing or it may be possible to overbuild the RCC with the assumption that there will be weathering and erosion of the overbuilt RCC.

Earthquake Loadings

RCC dams have been designed in the past to accommodate large ground accelerations by incorporating robust defensive measures and some of these measures may be required at this site to accommodate the high seismic loads required for design. The preliminary seismic stability analysis indicates that a 15 m RCC dam raise (total dam height of approximately 50 m) is likely to accommodate the high seismic loads with slopes of approximately 1H:1V downstream and vertical to near vertical upstream face. A detailed analysis and design will likely result in additional design features to accommodate the seismic loads but such design features cannot be predicted at this early stage. The screening level seismic stability analysis also indicated that the 27 m RCC dam raise (total dam height of approximately 60 m) is unlikely to accommodate the high seismic loads without significant additional design measures, such as flattened slopes, tendon anchoring, etc.

RCC dams do not accommodate foundation fault displacements, but based on current site investigations, a fault is not expected under the footprint of the dam.

A new CFRD embankment may also be constructed downstream of the existing dam. Modern rockfill dams are typically able to withstand high seismic loads due to their flexibility. Initial stability analyses indicate that a CFRD downstream of the existing Falls Dam would likely require flat slopes to accommodate the high seismic loads. The footprint of the CFRD embankments will be much larger than the RCC footprints and a large quantity of rockfill and a large spillway excavation will be required. The reservoir may also need to be lowered during part of the construction effort to accommodate upgrades or new offtake structure facilities. New CFRD embankments are not being considered as part of this feasibility study as they were ruled out in the prefeasibility study (OPUS 2012) and are not part of the existing scope of work. However, a new CFRD could be considered if the current study indicates that the RCC options are too expensive.

Preferred Height

New survey data indicates that a raise of the reservoir by 27 m will require construction of a considerable saddle dam at Shamrock Gully. Poor foundation conditions have been identified in the footprint of the saddle dam and it is predicted that cost of constructing such a structure will significantly impact on the 27 m raise option. No saddle dam would be required for the 15 m raise option.

Moving one dam type and height forward to feasibility level designs will allow for a more developed design with more accurate quantities and cost estimates. The preliminary seismic stability analysis indicated that the anticipated seismic loads can be accommodated into a 15 m RCC dam raise using fairly conventional design features. Larger RCC dams and particularly a 27 m RCC dam raise are likely to require additional design features such as flattened slopes, tendon anchoring, etc. However, it should be noted that the seismic stability of any RCC dam height will have to be verified during more rigorous analysis as part of final design.

The larger the storage at Falls Dam the greater the potentially irrigated area and the greater the potential benefits and as a result the benefit of a 6 m RCC dam raise may not outweigh the costs of the project. The 15 m RCC dam raise is the recommended dam height to move forward as it represents the maximum dam height which can be accommodated using fairly conventional design features. Hydrological modelling completed by Aqualinc (2013) suggests that a 15 m raise in the full supply level of Falls Dam would result in approximately 50 Mm^3 of storage and allow approximately 12,000 ha of land to be reliably spray irrigated in the Manuherikia Valley above Ophir.

Preferred Dam Type

A new RCC dam is the preferred option moving forward as the new RCC dam will not have to address deficiencies of the existing dam and the initial stability analysis indicates that a 15 m RCC dam will be able to accommodate the high seismic loads required for design. An RCC dam also has relatively rapid construction time, the reservoir can stay full during construction, and the existing dam can be used as a coffer dam during construction.

Conclusion and Recommendation

Based on our current evaluation, we believe that raising the existing Falls Dam as a concrete faced rockfill dam is not practical due to the substantial engineering effort that will be required to meet modern dam safety guidelines.

We recommend that raised storage is achieved by construction of a new RCC dam constructed immediately downstream of the existing dam. Due to seismic stability considerations, we believe the dam option that raises the reservoir by 15 m should be carried through to preliminary design. In addition, a substantially higher option will require construction of a saddle dam that will significantly impact on the construction cost for the project. The cost for larger or smaller dam configurations can then be estimated by scaling the various cost elements relative to the 15 m raise option.

Closing Remarks

We trust this letter provides an agreeable preferred dam option to move forward to preliminary design. If you wish to discuss any of the above please contact Rebecca Allen or Tim McMorran (reallen@golder.co.nz, tmcmorran@golder.co.nz or telephone 03 377 5696).

Kind Regards

GOLDER ASSOCIATES (NZ) LIMITED

Rebecca Allen
Geological Engineer

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RAA/TM/sb

Attachments:
A) Report Limitations.
B) References.

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Attachment A: Report Limitations

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iii) Conditions may exist which were undetectable given the limited nature of the enquiry Golder was retained to undertake with respect to the site. Variations in conditions may occur between investigatory locations, and there may be special conditions pertaining to the site which have not been revealed by the investigation and which have not therefore been taken into account in the Report / Document. Accordingly, if information in addition to that contained in this report is sought, additional studies and actions may be required.

iv) The passage of time affects the information and assessment provided in this Report / Document. Golder's opinions are based upon information that existed at the time of the production of the Report / Document. The Services provided allowed Golder to form no more than an opinion of the actual conditions of the site at the time the site was visited and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings, or any laws or regulations.

v) Any assessments, designs and advice made in this Report / Document are based on the conditions indicated from published sources and the investigation described. No warranty is included, either express or implied, that the actual conditions will conform exactly to the assessments contained in this Report / Document.

vi) Where data supplied by the client or other external sources, including previous site investigation data, have been used, it has been assumed that the information is correct unless otherwise stated. No responsibility is accepted by Golder for incomplete or inaccurate data supplied by others.

vii) The Client acknowledges that Golder may have retained subconsultants affiliated with Golder to provide Services for the benefit of Golder. Golder will be fully responsible to the Client for the Services and work done by all of its subconsultants and subcontractors. The Client agrees that it will only assert claims against and seek to recover losses, damages or other liabilities from Golder and not Golder's affiliated companies. To the maximum extent allowed by law, the Client acknowledges and agrees it will not have any legal recourse, and waives any expense, loss, claim, demand, or cause of action, against Golder's affiliated companies, and their employees, officers and directors.

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Attachment B: References

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