Manuherikia Valley Irrigation Strategy XX

Falls Dam Redevelopment Prefeasibility Study

Embankment Dam

Concept Options Update

Manuherikia Catchment Water Strategy Group

Falls Dam Redevelopment

Prefeasibility Study

Embankment Dam Redevelopment
Concept Options Update

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Contents

This report has been prepared for MCWSG in the context of the comprehensive full catchment water strategy study for the Manuherikia Valley. Its use for other purposes and by other parties is not authorised. Some content has been sourced from work undertaken previously for the Falls Dam Company and Pioneer Generation Ltd, and from work undertaken by others on alternative development options for this site and for other similar developments, but the views expressed herein are those of the author and should not be assumed to be those of the directors of those organisations. As this report is preliminary in nature, it should not be relied upon for making commitment decisions other than those for which it is intended; namely for progressing to feasibility study level investigations of selected development options. As is the nature of such development projects, it is possible that further investigations, design development, or other factors may lead to a need to review some of the opinions expressed herein that have been based on the current state of knowledge reached. The author would appreciate being formally informed of any such significant changes that may be relevant to the opinion expressed.

1 Introduction

1.1 Background

The Manuherikia Valley Water Strategy Group are seeking to facilitate further irrigation development in the Manuherikia Valley. The existing Falls Dam reservoir has been identified as a key site for increased storage through developed of a high dam at the site. Prefeasibility reporting (Opus 2012) has previously covered both redevelopment of the concrete faced rockfill embankment dam (CFRD) at the Falls Dam site, along with roller compacted concrete (RCC) options. While the roller compacted concrete (RCC) dam development concept addressed in the prefeasibility reporting was shown to be more expensive than the equivalent concrete faced concrete dam (CFRD) concept, the RCC approach was favoured for high dam development options as being better suited to the site characteristics and risks. This selection was particularly related to the tighter construction footprint of the RCC dam over the CFRD equivalent, insofar as the potential dam site between the existing dam and the original falls could be utilised. This presented the potential to utilise the existing dam and spillway facilities in essentially unmodified form to provide for construction diversion and dewatering.

A feasibility study on the preferred roller compacted concrete (RCC) high dam development potential for the site has recently been completed (Golders 2015), leading to an improved state of knowledge on some of the site characteristics, and updated construction costs. The feasibility study findings revealed substantially more challenging design conditions than anticipated in the prefeasibility study, leading to more conservative preliminary design decisions that resulted in the dam footprint being increased to the point that it was longer able to be accommodated within the area above the falls. The resulting increased scope of RCC construction has led to substantial increases in construction cost estimates for high RCC development options that include and exclude a saddle dam at the head of Shamrock Gully.

Further to the high dam development potential of the site, the owners of the current dam have been considering investing in moderate raising of the existing embankment to improve the security of supply to existing users.

1.1 Purpose

Given the RCC feasibility report finding of substantially increased construction costs for this development concept, the Manuherikia Valley Water Strategy Group have commissioned Opus to review and update their previous prefeasibility reporting covering redevelopment of the rockfill embankment dam at the Falls Dam site.

The purpose of this update is not to duplicate or reproduce previous work, as the relevant reports are available for direct reference. However, the changing state of knowledge on the site characteristics, along with changes in dam technology and the regulatory environment do present the opportunity to review the earlier work in the context of determining if a high embankment dam concept may be worthy of further investigation and development in a feasibility study phase.

The primary objective of this update is therefore to identify the likely form, scope and construction cost of high embankment dam development, given the current state of knowledge reached.

There is also a secondary objective of updating existing water users of the likely form, scope and construction cost of the moderate raising redevelopment option, based on the same current state of knowledge.

As the hydrology and topography of the Falls Dam site presents a wide range of possible impoundment volumes, it is also intended that the current update should give an indication as to what dam height (if any) may be a scoping "sweet spot", where a specific dam height may be expected to offer maximum investment value. This objective has been anticipated to some degree in the commission brief, insofar as the highest potential development requiring construction of the Shamrock Gully saddle dam is excluded, and the dam height is limited to those options involving maximum flood level below the saddle point.

1.2 Scope of this Study

This report is intended to be read in conjunction with previous relevant reporting, as the only selected information is represented herein. The reader is assumed to be generally familiar with the various studies previously undertaken in connection with this development potential. The update findings are presented in summarised form, with supporting background and detail held on file.

This update is limited to consideration of membrane faced rockfill embankment dam redevelopment options in the immediate vicinity of the current Falls Dam. No specific further investigations have been undertaken in support of this update, as reliance has been placed on desk review of existing information. Our focus has been on geotechnical factors, along with spillway hydraulic considerations. In looking forward to a possible feasibility study phase, we have broadly identified areas where significant risk or uncertainty appears to remain, and where further investigation and design development can be expected. However, any such feasibility study phase should be subject to detailed scoping at the time.

The preliminary designs presented in this update have been prepared with a view to scoping the construction work in order to meet the study objectives outlined above. The preliminary designs have not been subject to analysis that would satisfy normal design processes, as this is a function for later phases. In other words, the designs will be subject to change, so the information presented should not be taken outside of the context in which it has been prepared.

There are several exclusions from the current update, including the following:

- No further preliminary design consideration has been given to the offtake works as the distribution works interface is yet to be defined along with environmental release requirements.
- No further preliminary design consideration has been given to hydropower generation impacts and potential for future generation.
- No further consideration has been given to the deferred maintenance liabilities associated with the existing Falls Dam assets.
- No further consideration has been given to specific environmental conditions that may apply to site redevelopment activity.
- Land and property aspects have not been considered.
- Construction dewatering aspects have been generally qualitatively addressed rather than quantified at this stage.

- Construction methodology and site layout has not been subject to specific further consideration.

1.3 Other Factors

Further to the site specific information that has become available to assist with this update, there are some other factors that have been used to inform this study.

Notwithstanding the physical constraints and challenges involved, we have assumed that temporary substantial dewatering of the existing reservoir for redevelopment purposes is not ruled out in terms of either irrigation supply or environmental compliance considerations.

There is a similar embankment dam project proposed on the Lee River near Richmond as part of the Waimea Water Augmentation Project. This CFRD is of similar scope to the potential Falls Dam development, and the design has been progressed to 80% completion. Construction costs for this project have been subject to independent evaluation, and a relatively high level of confidence has been created in the risk adjusted figures arrived at. While we are not authorised to present detailed estimated rates, the overall costs are in the public domain and able to be compared with the similar nature and scope of work proposed at Falls Dam.

Rockfill embankment dam redevelopment examined at this site to date has utilised reinforced concrete facing as the upstream water retaining element. There are other membrane options, and recent use of a PVC composite geosynthetic membrane on the Tekapo Canal relining project has highlighted the potential benefits of using such a flexible membrane option. We are continuing to liaise with the specialist suppler of this product, and we have assumed that this material is likely to provide a cost effective and technically suitable design solution at Falls Dam.

The government has recently announced an intention to change the statutory dam safety management framework. We understand the Resource Management Act (RMA) will be used for this function rather than the Building Act as previously intended. The Building Act will continue to apply to the Building Consent process insofar as alterations or new dam construction works are concerned, but ongoing management of impoundment safety will be consolidated with the RMA compliance obligations for dam owners. This change is not expected to significantly alter the obligations faced by dam owners, as the accepted means of compliance generally applied is the NZSOLD Dam Safety Guidelines which are based on international good practice. These guidelines have been recently updated (May 2015), and although a lot more detail has been added, the philosophy is essentially similar to the c2000 version. This dam safety management factor is most relevant to ongoing operation of the existing ageing assets, insofar as there is an underlying expectation that all deficiencies relative to current engineering standards will be identified and mitigated. This expectation applies to both design and deterioration related deficiencies.

2 Methodology

We have adopted the following outline methodology in pursuing the update study.

1. Carry forward the previous CFRD options presented in the c2012 prefeasibility report to provide a linkage to this previous work.

2. Review relevant current site characteristics generally as presented in the recent feasibility reporting for the RCC dam concept, including topography, hydrology, geotechnical characterisation, seismicity, etc.

3. Compile a new digital terrain model for the dam site using latest survey data and some earlier local data. Note: elevation datum now consistent with the Golders feasibility reporting.

4. Develop a scoping design for the moderate raised height embankment dam **(option 1)** as an update to previous crest raising to improve security of supply to existing users, in light of current site characterisation.

5. Develop a scoping design for the mid height raised embankment dam **(option 2)** that cam maximise the use of the existing assets for construction facilitation purposes in light of current site characterisation. (i.e. utilise existing bellmouth spillway and tunnel and low level offtake potential for dewatering purposes, and effectively limit the dam footprint to above the falls)

6. Develop scoping designs for the maximum height embankment dam condition (excluding Shamrock Gully saddle dam) for both raised and new standalone developments **(options 3 & 5 respectively)** in light of current site characterisation to compare with the recently reported RCC dam optimisation option (Golders 2015b).

7. Develop scoping design for the mid height standalone embankment dam **(option 4)** in light of current site characterisation to provide direct comparison with option 2.

8. Assume a raised operational bell mouth spillway and new uncontrolled left abutment auxiliary rock cut spillway for Option 1.

9. Develop scoping concept design for new left abutment uncontrolled spillway sill and concrete lined spillway chute layout suited to dam raising options 2 & 3.

10. Develop scoping concept design for new left abutment uncontrolled spillway sill and concrete lined spillway chute layout suited to standalone embankment dam options 4 & 5

11. Refine scoping concept site layouts for both raising (options 1,2,& 3) and standalone (options 4 & 5) developments taking into account a possible staged development approach.

12. Briefly examine construction diversion implications for options 3, 4,&5 that would obstruct the existing tunnel outlet portal, and scope the layout of

13. Subject spillway scoping designs to challenge by a technical specialist to identify key design considerations and issues.

14. Quantify the key construction elements for each option including diversion works, rockwork and foundation preparation, membrane and associated plinth and coping wall, spillway lining, etc. Include key quantities from the c2012 prefeasibility CFRD options.
15. Index the previous c2012 prefeasibility construction cost option estimates to current values.
16. Compare the new concept designs key scoping quantities to the similar Lee River dam project.
17. Relate the robust Lee River risk adjusted project cost to the scoping concept options – principally the similar option 5 standalone option that is most comparable in form and scope.
18. Interpret the standalone high dam cost comparison in light of the scope of work associated with the other scoping concept options, and compile a preliminary costing update.
19. Identify any preferred option(s) for feasibility study.
20. Identify key considerations for investigation during a feasibility study, especially any potential "show stoppers" that have come to attention through the prefeasibility update, along with other factors presenting risk and/or uncertainty to the development.

3 Embankment Dam Conceptual Design Options

In presenting this update we have sought to generally refer to redevelopment options relative to current full supply level (FSL) as defined by the current bell mouth spillway sill. We trust that this approach enables easier visualisation of the proposals and comparison of the various options.

3.1 Previous Embankment Redevelopment Options

Previous CFRD prefeasibility options presented c2012 related to raising the existing embankment. Preliminary design and costing of the standalone embankment options were not developed at that time, as the RCC concept was seen to be preferable for standalone development on the restricted topographic footprint available.

The labels given to the options at that time were:

567.5 option **(existing FSL +6m)**

577 option **(existing FSL +15.5m)**

588 option **(existing FSL +26.5m)**

The option titles refer to the approximate redeveloped FSL in terms of the construction datum. This datum has changed with the topographic survey work undertaken during the recent RCC feasibility study, as described elsewhere in this update.

The nominal FSL elevations of these options when converted to the new datum become:

567.5 option (+6m) = 571.1m

577 option (+15.5m) = 580.6m

588 option (+26.5m) = 591.6m

The 567.5 option was intended to simply address the requirement to improve security of supply to existing users. The earlier auxiliary rock cut spillway concept with a fuse plug crest detail to complement a raised bell mouth spillway was modified in this report to comprise a simple uncontrolled auxiliary rock cutting spillway. This approach increased the freeboard requirement above FSL and resulted in greater rockfill demands, but it avoided the risk of unreliable fuse plug breach initiation. The spillway cut volume was nominally balanced against the embankment fill demands.

The 588 option included the saddle dam at the head of Shamrock Gully, and a new left abutment spillway, and was seen as maximum practical scope of embankment dam redevelopment.

The 577 option was an intermediate scope of development prepared to gain an understanding of the relationship between scope of work and embankment height at this site.

All three options adopted a design flood discharge requirement of 700m^3/s. Scoping concepts and costing for these options is presented in the 2012 prefeasibility report. For ease of reference, we have appended copies of selected drawings prepared at that time.

By way of further background, we note that the future potential to raise the embankment crest by some 14.2m was envisaged at the time of the original c1930's development of the current facilities. It is not clear as to how much detailed consideration was given at the time to making provision for increased loading on the various structural features etc., but it can be reasonably inferred that this potential was at least considered for the dam embankment if not for the spillway facilities.

3.2 Further Embankment Options

In implementing the study methodology outlined above for updating the CFRD redevelopment options, we have defined the new scoping concept reservoir raising as quantified below. All options are focussed upon rockfill embankment dam concepts that are broadly similar to the existing 9.5Mm^3 storage dam with progressively increasing gross storage up to the limit that is possible without the need for saddle dam construction. The means of achieving these storage volumes may be achieved by either raising the existing embankment or by constructing a new embankment immediately downstream. Each of these approaches have their respective pro's and con's. The 5m reservoir raising has only been considered in terms raising the existing embankment. The height of three storage options being examined are:

1. **Increase FSL by 5m** above existing to RL 570.1m **(Option 1 Raising)**
2. **Increase FSL by 12.5m** above existing to RL 577.6m **(Option 2 Raising, 4 Standalone)**
3. **Increase FSL by 20m** above existing to RL 585.1m **(Option 3 Raising, 5 Standalone)**

The 5m increase in water level aligns with improved security of supply to existing irrigators, and the 20m increase is intended to provide for substantial land use changes in the valley and it closely matches the optimised roller compacted concrete option (Golders 2015b) The intermediate 12.5m option may represent a limited development in its own right, or be envisaged as a possible staged development point that may satisfy demand arising from land use changes for say 15 or more years, thereby allowing capital expenditure to better match potential benefit cashflow.

The means of achieving these storage volumes may be achieved by either raising the existing embankment or by constructing a new embankment immediately downstream. Each of these approaches have their respective pro's and con's. The 5m reservoir raising has only been considered in terms raising the existing embankment. Limiting the degree of FSL raising of the existing embankment to 12.5m would allow the existing spillway and tunnel to be utilised directly as the construction diversion works, but use of the existing embankment effectively as the coffer dam would present construction dewatering and foundation treatment challenges, along with technical compliance difficulties associated with characterising the c1930's dumped rockfill. Constructing a new embankment immediately downstream would avoid these challenges, but substantial diversion works would be required due to the conflict with the existing tunnel layout.

4 Factors Influencing the Design

In this section we briefly outline the key factors we have considered when updating the embankment dam concept designs and for scoping update purposes. The focus in this section is on the preliminary design changes that have been made in response to the changing state of knowledge.

We are particularly interested in identifying matters requiring serious consideration at the feasibility phase, along with risk and uncertainty aspects that may affect commercial viability of the project.

Design at this early stage of project development are subject to a high degree of iteration as new information progressively comes to hand. We have sought to be moderately conservative in our approach to reflect the limited investigations so far undertaken, but not unduly so. Our methodology also entailed preparing the concept designs for systematic challenge, so the drawings and quantities presented do not necessarily represent the layout or dimensions of any final option that may be adopted for detailed design and construction.

4.1 Topography and Basin Storage Potential

The storage basin above Falls Dam has been subject to original imperial topographic mapping at the time of the original 1930's development, and that datum has been previously used in the prefeasibility study (Opus 2012) with the existing full supply reduced level of 1842 feet equating to RL 561.44m at that time, although there is some limitation in the datum accuracy for the photogrammetry undertaken from existing aerial photographs that were used in that study; assessed to be around 0.15m.

Further topographic definition has been undertaken in connection with the recent feasibility study (Golders 2015). That survey is in terms of the LINZ listed trig N No 2, which is located on the ridge at the south East corner of the Falls Dam site, and the work included updating the site datum to Dunedin Vertical Datum 1958. This updating increased the RL values by some 3.635m , such that the existing FSL became 565.075m, (rounded to RL 565.1m). This is the datum reference used in this report.

We have reviewed the Golders basin stage - storage curve and the earlier Opus stage - storage curve with the datum correction and have found a mismatch in the derived volumes. For the purposes of this update we have adopted a conservative interpretation of the available information by adjusting the higher level Golder curve to fit the lower elevation Opus model previously developed. This has involved a change of some 1.7Mm^3. We note that the higher elevation volumes after this adjustment still exceed volumes obtained from the earlier photogrammetry. Further detailed examination of the latest digital terrain model along with assumptions regarding sedimentation in the reservoir etc., will enable this matter to be clarified, but it is not material to the decisions being addressed in this update, provided the residual uncertainty in these figures is recognised.

The gross storage curve adopted for this update is illustrated in the following figure, along with the points on the curve representing the existing dam and the additional embankment redevelopment options. The relative gross storage volumes of the options are also shown on the figure overleaf.

Gross storage for the options are:

- @ existing FSL gross storage is around 9.5Mm³
- Increase FSL by 5m to increase gross storage to just under 18Mm³
- Increase FSL by 12.5m to increase gross storage to just under 40Mm³
- Increase FSL by 20m to increase gross storage to around 70Mm³

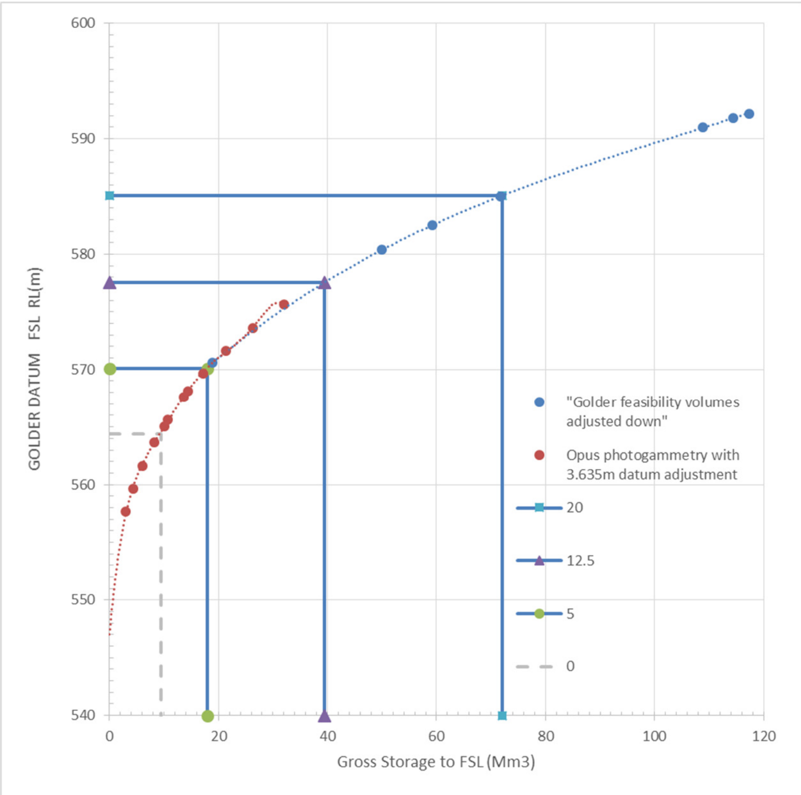

Figure 1 Basin Storage Characteristics and Redevelopment Option FSL

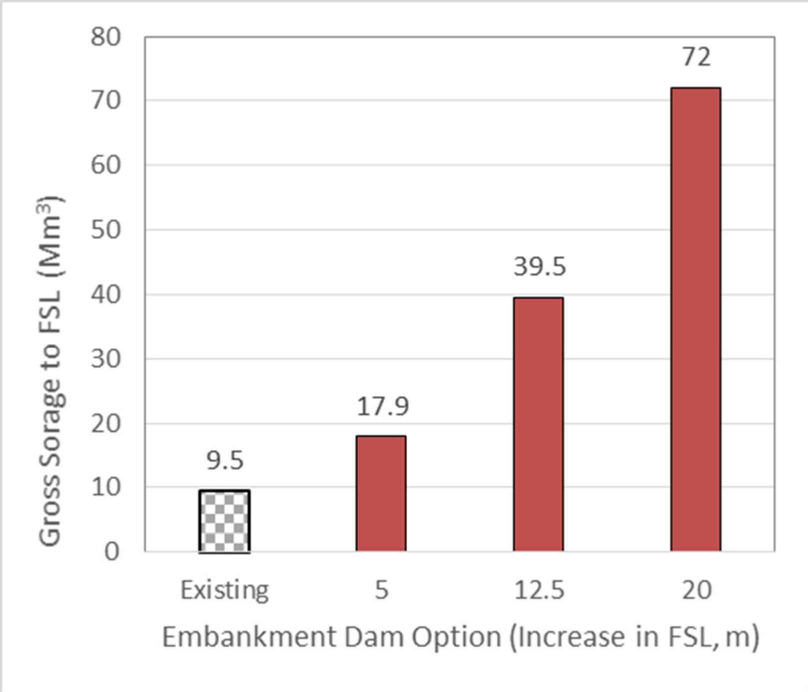

Figure 2 Relative Gross Storage of Redevelopment Options

4.2 Site Hydrology

This update exercise does not include consideration of the water balance, catchment yield, or other factors applicable to the normal operational management of the reservoir. These matters have been generally documented elsewhere and will be subject to further development in any specific embankment dam redevelopment feasibility study that may be commissioned.

However, the flood discharge capacity is relevant to this update insofar as it relates to both existing facilities (along with any design deficiencies that may be present), and to any new facilities that may be proposed.

A specialist review has been commissioned of the spillway concept designs adopted as part of this update. A copy of the review is appended to this report, and a status overview of the flood hydrology is included therein.

Key hydrology findings from the specialist review are:

- The extreme flood flows derived for the catchment by different parties vary considerably in magnitude, and this variability needs to be resolved for design to progress in any feasibility phase.
- An impoundment rated as MEDIUM potential impact classification (PIC) could be expected to require a design flood discharge capacity in the range 600 to 700m^3/s. 700m^3/s has been used in previous studies.
- An impoundment rated as HIGH PIC could be expected to require a design flood discharge capacity in the range 1000 to 1100m^3/s representing probable maximum flood (PMF) condition.

- The existing bell mouth spillway is subject to choking behaviour as the discharge exceeds some 400m3/s. It is expected that the maximum discharge capacity will be in the range 425m^3/s to 450m^3/s subject to the actual surcharge head applied. This limiting capacity represents a 1 in 500 or possibly 1 in 1000 annual recurrence interval (ARI) flood event.

We are of the view that while the existing 9.5Mm^3 impoundment may be confirmed as a MEDIUM PIC rating, it is quite likely that the larger impoundments associated with redevelopment will attract a HIGH PIC rating. This matter will require attention in any feasibility phase, involving reassessment of the potential dam breach mechanism and quantification of the downstream effects of such a breach in both sunny day and background river flood scenarios.

Other flood hydrology considerations applicable to this update include the low level dewatering of the existing dam face, along with the downstream discharge of construction flood flows and the associated construction dewatering demands of the dam site(s). Construction flood handling requirements will need specific assessment as construction programming is developed, but as a general guide the flood discharge capacity would need to be in the same order as the current bell mouth spillway for any sustained construction period where the reservoir may be nominally at FSL.

4.2.1 Construction Diversion and Dewatering

Figure 3 bell mouth spillway showing raised sill

Construction phase flood handling for all redevelopment options will require the existing bell mouth spillway and tunnel to be utilised. There are differences between options associated with the need for extensions to the tunnel, and there is potential for a mid-height transition to a new spillway facility. This aspect is discussed further in the discussion on concept designs below.

Dewatering requirements differ substantially between the raising and standalone redevelopment options. In fact this factor is perhaps the primary differentiator between these two approaches to redevelopment. Raising the existing embankment will require access over the existing upstream face. There may be potential to undertake some underwater construction activity, but it has been assumed that the scope of such work would need to be kept to a practical minimum in terms of the cost implications.

The diversion tunnel invert levels are:

- inlet =R L 533.9m,
- outlet = RL 532.2m

The original low level left abutment diversion tunnel was plugged with a concrete bulkhead when the reservoir was commissioned. A 33" (0.838m) diameter cast iron pipe was incorporated into the bulkhead for supply offtake purposes. This relatively small pipe is still in service, but the capacity is very limited in terms of ability to contribute to sustained dewatering of the existing dam face to facilitate redevelopment of those options that involve raising the existing dam. This concern is a serious constraint to be addressed, and the maximum potential discharge rating of this feature with free discharge into the tunnel is illustrated in the figure below.

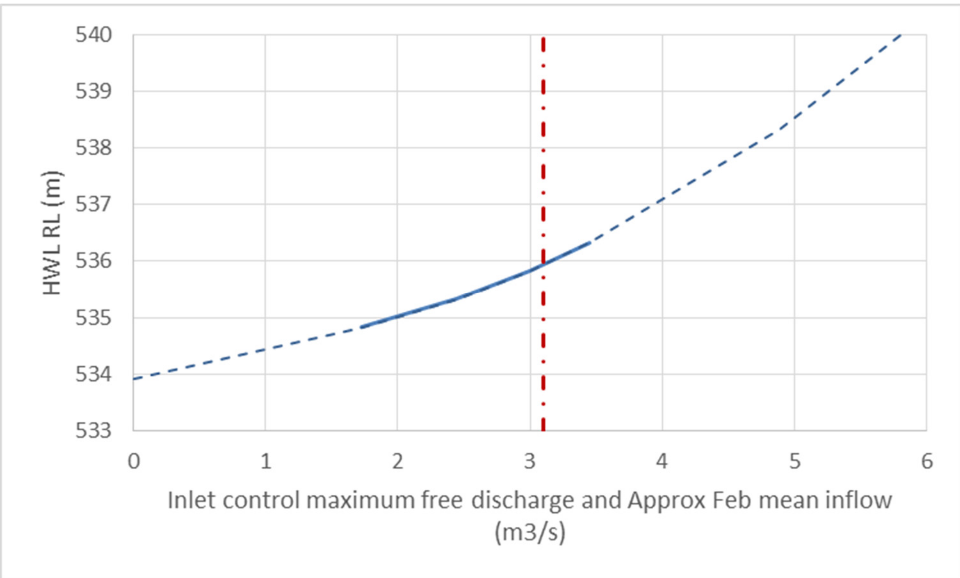

Figure 4 DN838 CI offtake maximum potential rating

The 33" pipe invert in the tunnel bulkhead is RL 533.9m, and it includes a large radius bell mouth entry.

We have included an approximate reservoir inflow value of 3.1 m^3/s to illustrate the lowest mean monthly figure that applies in February. Care needs to be taken when considering mean flows, as the challenge is really a dynamic one, insofar as the construction requirement is to handle variable inflow conditions. With very little storage available under lake draw down conditions, there would be little ability to buffer the effect of any rainfall events, even with the provision of a substantial upstream coffer dam. This aspect will require careful assessment for all the embankment raising options.

Note: The operating consent conditions limit the permitted irrigation draw off when the reservoir is below RL 554.6m to only 1.0m^3/s.

Existing supply offtake from the reservoir discharges at either the tunnel portal or from the nearby small hydropower tailrace. These discharge locations will conflict with the footprint of several of the embankment dam redevelopment options considered in this update, and further diversion and dewatering works will be necessary to facilitate development of these options. Some form of extension of the existing spillway tunnel is envisaged as being necessary to address this constraint; similar perhaps to the RCC development concepts (Golders 2015) or possibly involving an equivalent tunnelling solution.

4.3 Site Seismicity

The seismicity of the dam site has been specifically quantified as part of the feasibility study recently undertaken for the roller compacted concrete (RCC) dam development options (Golders 2015). This study indicated relative high maximum seismic loading values should apply for design purposes; significantly higher than had previously been assumed for the earlier prefeasibility study (Opus 2012).

Seismic ground actions at a given site are examined in both probabilistic and deterministic terms. That is the site ground actions arising from all possible contributing earthquakes in terms of probability or likelihood, and the site effect of the maximum controlling earthquake. Dams of the nature being considered here are generally designed for the effects of the maximum controlling earthquake (MCE) after attenuation with distance from the controlling fault is allowed for. However, the maximum ground action does not need to exceed the values obtained from probabilistic assessment at an annual recurrence interval of 1 in 10,000, (i.e. 0.01% probability of occurrence per annum). For dams, it is often the moderate infrequent earthquakes on nearby faults that determine the maximum design ground actions, rather than the larger more distant events on say the Alpine Fault.

Further considerations concerning ground actions include the nature of the foundation (i.e. rock or soil etc.,), and the energy distribution across differing vibration periods, as dams often have natural vibration modes that closely match the peak energy levels imparted by earthquake ground vibrations.

Dams are not expected to be unaffected nor undamaged when experiencing the maximum design earthquake (MDE), and for embankment dams some permanent deformation is to be expected. However, such deformation may not be so great as to lead to a risk of uncontrolled release from the impoundment, and the facility must remain sufficiently resilient after the primary event to resist aftershocks and to confidently provide for an effective response strategy to be implemented. That is, following an extreme seismic event the dam may be significantly damaged in terms of remaining permanently in service, but the impoundment must not be subject to the risk of uncontrolled release.

Dams must also be resilient when subjected to a smaller more likely seismic loading case at the 1 in 150 annual recurrence interval level (0.67% probability per annum). This is referred to as the operating basis earthquake (OBE) that is reasonably expected to occur within the service life of the dam, and under which no significant damage is permitted.

As seismic resilience is a key consideration for dams of this scale and type, the latest seismicity findings have been briefly examined relative to other rock foundation dam sites where earthquakes of similar magnitude and distance from the site apply. I have plotted the MDE spectral response curve from the Lee River project (a 53m high CFRD), and the peak ground acceleration (PGA) values from the Ross Creek Dam refurbishment project (18m high puddle clay core embankment) alongside the spectral response applicable to the Falls Dam site.

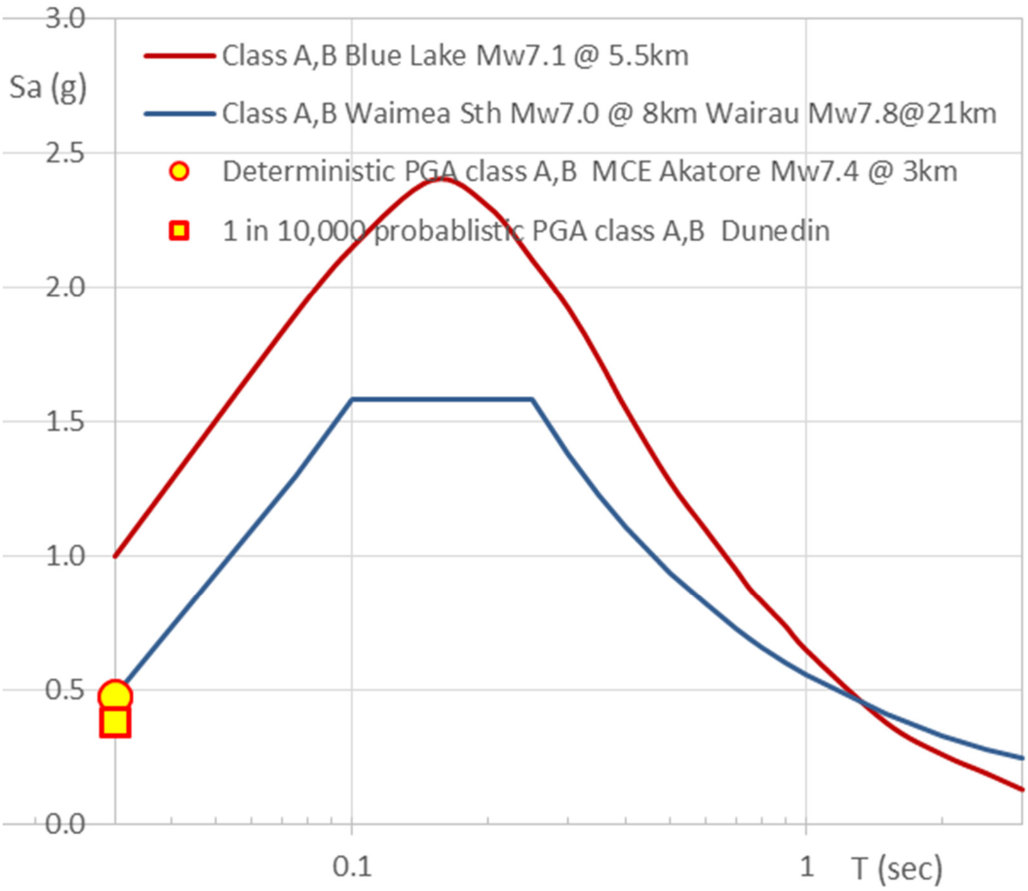

Figure 5 5% damped spectral response comparison

The standard building loading code Z factors obtained from the National Seismic Hazard Model (NSHM) for these sites that relate to the peak ground accelerations at around a 1 in 500 year annual recurrence interval (0.2% likelihood per annum) are:

- Lee River 0.30
- Falls Dam 0.24
- Ross Creek 0.13

The relatively high design values shown in the above figure for the Falls Dam site do not superficially match the relative Z hazard factors, so some further examination of the design parameters may be warranted before progressing into more detailed design of embankment dam options.

4.4 Geotechnical

The site geological setting and geotechnical characterisation is described in the recent feasibility investigation report (Golders 2014). The dam sites have extensive exposures of jointed Argillite that generally present stable and erosion resistant characteristics. This is perhaps most evident around the original falls where the river bed was shown to be highly erosion resistant. There are also very steep rock slopes present downstream of the existing dam, with no apparent evidence of significant instability in the rock mass. There are however many rock mass defects present, including steeply dipping shear zones that contain weathering products. Stereonet plots of the defect mapped rock mass defects are presented in the investigation report (Golders 2014). No subsurface investigations of the redevelopment dam sites other than the saddle dam site were undertaken during the feasibility study.

The existing dam foundation has performed well in service, with no significant performance deficiencies identified. It is understood that only limited grouting was undertaken in the vicinity of the upstream cut off, indicating that despite the degree of jointing generally present in the rock mass, relatively low permeability conditions prevail. We have appended selected construction record drawings, including some details of the rock conditions observed at the upstream cut of trench and within the diversion tunnel excavation.

Figure 6 View of quarry face above left abutment bell mouth bench showing shear zones

Figure 7 View of Left Abutment. Valve chamber access adit at lower left, Powerhouse Lower right.

Figure 8 Excavation for hydropower scheme c2003 illustrating typically limited surficial cover

Figure 9 Wide angle view of right abutment area

Figure 10 View of rock mass exposure downstream of tunnel portal

Figure 11 View of steep rock mass exposure on left bank of river near proposed spillway chute

Figure 12 Excavation for Powerhouse at original falls

4.5 Condition of Existing Assets

The existing dam assets at the site do have deferred maintenance liabilities and design deficiencies relative to current engineering standards. These matters are presented in the previous prefeasibility reporting (Opus 2012), and are not duplicated in detail herein. Key liabilities include:

- Limitations in extreme flood discharge capacity
- Deterioration of dam membrane joints and concrete (some recent temporary repair work has been undertaken)
- Deterioration of spillway and tunnel concrete lining

Recent site seismicity findings described above also indicates that :

- Resilience of steep embankment shoulder may be deficient.

We have included selected views of the key assets to illustrate their overall condition.

Figure 13 View of crest settlement c2001

Figure 14 Closer view from true right crest c2015

Figure 15 Offset at TR double post c2015

Figure 16 Offset at TR double post c2015

Figure 17 Offset at TL perimetric joint c2015

Figure 18 Offset at TL perimetric joint c2015

Figure 19 c2015 view of membrane

Figure 20 View of TR perimetric joint prior to 2015 repair

Figure 21 Seepage monitoring prior to c2015 membrane reapir (115mm on V notch)

Figure 22 View of Cu waterstop exposed at TR crest c2015

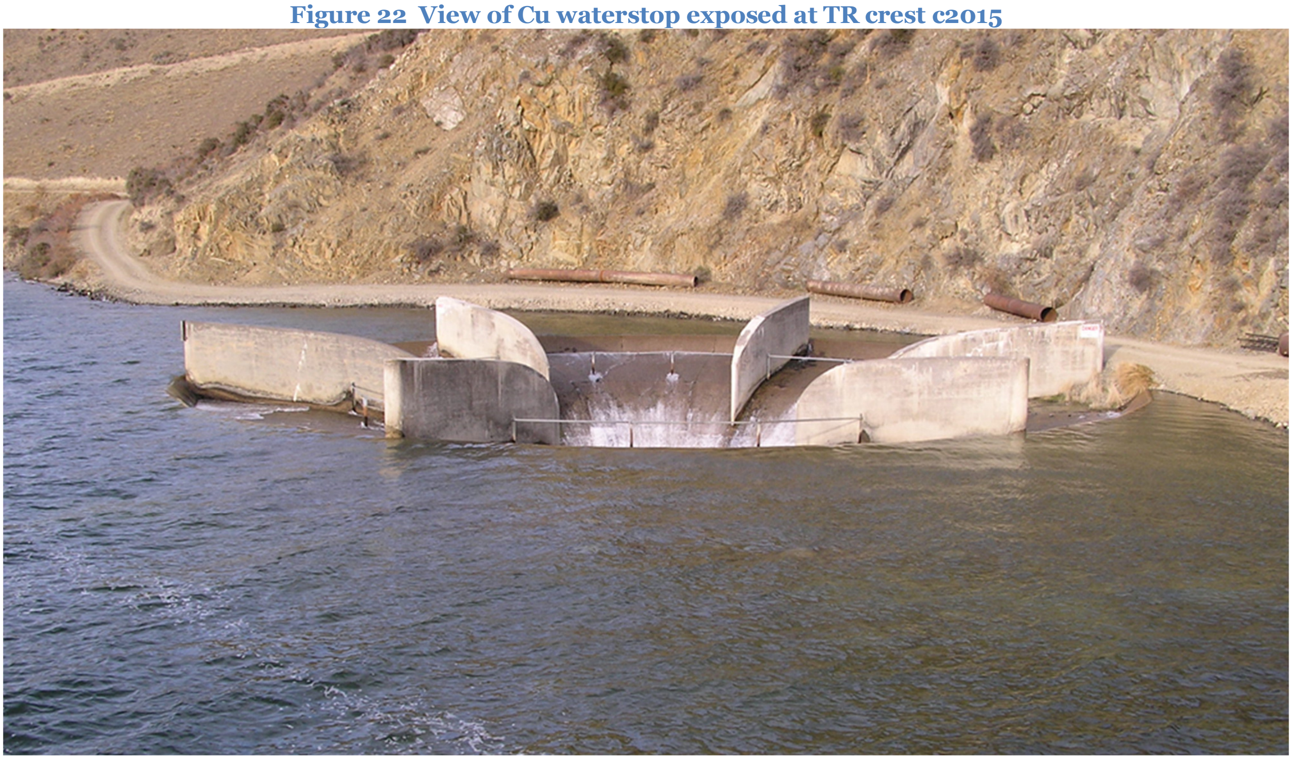

Figure 23 Partially obstructed bell mouth at track.

Figure 24 DN1200 Penstock Cut Off at Crest

The retrofitted small hydropower scheme was not designed with allowance for dam redevelopment of the scope envisaged in this study . We have included a view above of the cut off extension at the syphonic penstock; noting this area would be subject to significantly changed conditions under a dam raising option.

Furthermore, the DN1100, DN1200 and DN1400 buried steel penstocks are located within the footprint of the redevelopment dam options, along with the powerhouse in some options. These features will require treatment / removal as part of foundation preparation activity.

We have also appended selected drawing records of the existing assets to provide a record of the key existing features.

5 Scoping of Redevelopment Options

In this section we outline the design thinking underlying the key elements of the updated scoping concepts. We have appended sketches of the concepts to illustrate the proposed layouts and relationship of the various elements.

We again note that the offtake works are not specifically included in the sketch layouts.

5.1 Potential for Staging of Redevelopment

As introduced in our methodology outline, we have given thought to the potential to stage the redevelopment. The upstream membrane rockfill embankment dam form does lend itself to progressive development, but the spillway geometry can be the controlling factor. In this case the spillway provision is the dominant factor, and we have allowed for the mid height (+12.5m) option to be transitioned to the full height (+20m) option by the introduction of a raised crest weir into the spillway crest. This aspect is discussed further below.

The moderate raising (+5m) option does not lend itself so readily to this progressive spillway raising approach, as the increase in crest elevation to the full height option is too great for a simple crest weir approach. However, if the ultimate development was to be limited to the mid height reservoir elevation, then the single step increase may still be a possibility. This highlights the key strategic consideration needed before embarking upon a moderate raising project.

Offtake works for mid and high level options may still utilise the existing spillway and diversion tunnel in a similar manner to that outlined previously (Opus 2012). Alternately the standalone options could be provided with new conduits as part of such development.

5.2 Diversion and Dewatering

The raising options up to 12.5m above existing can utilise the existing bell mouth spillway and tunnel for diversion purposes. Once this mid height embankment level is reached, it is intended that the left abutment spillway chute described below will become functional, allowing floods to be discharged via this feature as the embankment is raised beyond this level. Once the ultimate embankment and membrane work is completed, a concrete weir would be constructed in the spillway crest to raise the operational water level above the mid height elevation accordingly.

Dewatering of the upstream face to facilitate practical membrane placement and any foundation treatment required to address the increased hydraulic gradient will present practical challenges. A planned reservoir lowering and moderate upstream coffer dam approach is envisaged at this time.

Downstream redevelopment options conflict with the existing tunnel, and separate diversion works will be necessary. We have envisaged a tunnel extension in the left abutment to convey flood discharge clear of the dam site, together with a downstream coffer dam. The length of such a tunnel extension would be dependent upon any decision to stage the embankment construction, with some 240m required for the mid-range development, and over 300m required for the high level option without staging.

The 500 year current flood handling capacity will be quite adequate for construction diversion purposes, so the tunnel can be expected to be of similar cross sectional dimensions to the existing.

In referring to the Lee River proposal and its design concepts, the diversion there comprises a 168m long 20m^2 waterway area structural concrete twin cell box culvert able to withstand the later rockfill imposed loading pressure. It may be possible to construct a similar feature at Falls Dam on a rock bench alongside the river channel below the existing tunnel outlet as an alternative to tunnelling. However, the topography is not automatically well suited to this approach, and more detailed work will be required to assess feasibility. The culvert length would be at least 150m or 190m for the mid and high level options respectively.

5.3 Rockfill Embankment Design Concepts

The rock material able to quarried from the site is expected to be quite suitable for the embankment construction work involved for all options being considered in this update study. Zoning within the dam and production quality control will need to more sophisticated than the approach taken in the original construction, but this factor is not expected to present significant technical constraints to the redevelopment.

Heavy compaction of the new rockfill will be required to improve the performance of the embankment, and this aspect highlights the lesser performance of the dumped rockfill material in the existing dam. The steep shoulders present in the existing embankment may also present design compliance challenges in terms of the raising options; becoming potentially more significant as the height is progressively increased.

Further to the previously published prefeasibility report findings, additional considerations for the current embankment options include:

- The high seismic resilience demands identified in the recent Golder's study
- The rock mass defects and state of weathering identified in the recent Golder's study

For preliminary scoping purposes we have decreased the steepness of the embankment batter slopes to improve resistance to deformation under severe seismic loading, and introduced reinforcement of the rockfill in strategic locations where local failure may be initiated. This approach would introduce a step in the upstream shoulder and membrane that will require careful analysis and assessment to ensure that any local strains can be accommodated. The footprint of the intermediate 12.5m raising option can fit above the original falls, raising the possibility that the existing hydropower scheme might be retained in service if appropriate measures were taken in regard to such aspects as foundation treatment around buried penstocks etc.

However, the steep bluff above the tunnel portal may present stability concerns under the imposed embankment loading, so the foundation properties will need to be fully investigated. This aspect become less significant for the full height option when the bluff is no longer an exposed face.

The behaviour of the existing dumped rockfill embankment under increased mechanical and hydraulic loading, and the potential for differing stiffness and strength of the dumped vs compacted rockfill is a matter for detailed assessment. Characterisation of the existing rockfill in quantitative terms will not be a simple matter, and consideration needs to be given to the investigation methods that may practically be used. It is thought that geophysical investigation techniques may be useful insofar as wave transmission behaviour within the rockfill will give a good indication of its stiffness. However, care will be needed to determine what the large strain behaviour might be from testing based on small strain response, as the rockfill is likely to exhibit non-linear behaviour as the stresses increase.

5.4 Spillway Design Concepts

Our thoughts on the spillway layouts for the current redevelopment options all involve left abutment rock cuttings. We envisage utilising a weir crest discharging to supercritical lined spillway channel on 2:1 maximum slope with a terminal flip bucket for the mid and high level options. The layout of raising and standalone options is similar, but positioned to suit the respective footprints of the embankments. We have provisionally sized the spillway crest length at 40m to limit flood surcharge effects for the nominal 700m^3/s flood condition, and a 7.5m high concrete ogee weir can allow for raising from the 12.5m to 20m water level increase if required. We have adopted a 16m wide chute with a plan length of the lined spillway in the order of 166m or 200m for the mid and high level options respectively. In referring to the Lee River proposal and its proposed scope of work, the spillway there comprises a 182m long 20m wide concrete lined chute on a 2:1 maximum gradient with a terminal flip bucket. The scope of this work is broadly comparable to the Falls Dam proposals, although some lining differences may arise.

The concept designs have adopted a freeboard dimension above FSL of nominally 5m. Allowing for wave run up margin of say 0.9m, the spillway surcharge is around 4.1m. This figure is assessed to be in the right order for the concepts shown provided the design discharge requirement is not more than 700m^3/s and hydraulically efficient crest weir conditions can be achieved. If these assumptions do not apply, the spillway dimensions will need to increase. Freeboard is effectively created by the coping wall detail shown in the drawings, so there is potential to adjust this within reasonable limits by structural means. However, substantial changes from the adopted freeboard would result in corresponding changes to the earthworks crest.

The spillway cutting can provide a substantial quantity of rock fill for embankment construction purposes, subject to detailed geotechnical investigation and assessment. Sequencing of construction would necessitate early excavation of the spillway to facilitate transition of flood handling capability for the high options.

Spillway concepts for the moderate (+5m) raising option are essentially unchanged from those presented in the previous prefeasibility report (Opus 2012). The bell mouth spillway is retained in a raised form as the service spillway, and an auxiliary rock cutting is provided in the left abutment to discharge

The hydraulic aspects of the spillway is concept have been examined by our specialist engineer, and the review findings are appended in full to this report. Outcomes from this overview include:

- The chute spillway concept is essentially sound, but details will need to be thoroughly assessed.
- The key dimensions in the concept sketches are lean for the 700m^3/s discharge case, and will be significantly increased for the PMF case.
- The hydraulic efficiency of the crest detail will be significantly enhanced by improving the approach depth; i.e. deeper rock cutting.

- Detailed attention will be needed to maintain supercritical flow away from the crest and avoid unstable flow in some discharge conditions.
- The peak flow velocity in the chute is close to cavitation threshold, so design will need to address this aspect.
- The terminal jet and plunge pool performance will need to be specifically assessed.

For the moderate raising option retaining the bell mouth spillway:

- Careful attention will need to be given to aeration of the flow entering the vertical shaft, and potentially accumulating in the crown of the tunnel. This behaviour could lead to unstable flow conditions.
- Vortex control (guide vanes) will be needed for the raised bell mouth.
- Venting of the "void" created beneath the falling jet will be needed to control flow stability.
- Jet impact on the existing concrete lining will need to be assessed in both hydraulic behaviour and structural performance terms.
- The potential cavitation potential related to the increased head on the vertical shaft will need to be assessed.
- The possible physical characteristics of the auxiliary spillway are also discussed in the review report; emphasis is given to the need to balance the operation of the two spillways while minimising freeboard demands.

5.5 Membrane / Water Retaining Design Concepts

Previous redevelopment studies have assumed the use of a slipformed reinforced concrete membrane for the upstream waterproofing element; similar in general appearance to the concrete panels on the existing dam. The Lee River proposal has also been based on this concept, including the use of a concrete starter dam at the dam heel in place of the conventional plinth at this foundation location. We propose a similar starter dam approach for use at the stand alone new Falls Dam sites, as it would improve working access for construction in the tight area at the toe of the existing dam. There is also the possibility of taking a similar approach to raising the existing dam by utilising the 1.5H:1V heel facing zone below the existing membrane "hinge". However, such a hybrid approach would require careful assessment to ensure stiffness compatibility and deformation detailing at under all loading conditions.

Alternative membrane material in the form of a PVC composite flexible liner is being considered for the current options as it is believed it will offer cost advantages over concrete, and potentially deliver improved seismic resilience with its ability to accommodate deformation without tearing. The service life of this product is comparable to concrete, so deterioration rates are not a disadvantage. However, mechanical damage through accident or malicious action could be a concern, as we have envisaged the membrane remaining exposed, without any cover being provided.

Details of the proposal to use PVC composite have been forwarded to the specialist international supplier for their feedback and indicative costing. Subject to review following receipt of the feedback received in due course, we have assumed that for the embankment raising options the flexible membrane would be placed over the existing concrete facing following preparation of the surface and suitable mechanical support being provided over open panel joints etc. The membrane would be laid in vertical sloping strips and mechanically secured to the substrate. Closer panels would then be welded over the vertical sloping joints. A conventional foundation cut off detail will be required to provide a waterproof connection to the foundation. This is likely to be reinforced concrete plinth similar in form to the detail previously shown for a concrete membrane (Opus 2012). Foundation preparation and grouting along the plinth line is expected to be required once detailed subsurface investigations have been undertaken to establish the rock mass permeability the spatial variability in the defect characteristics. Location specific treatment requirements can be anticipated in the geotechnical conditions expected, but overall the past performance of the lower head dam has indicated overall good performance with limited treatment. Drilling and water pressure (Packer) testing along the plinth line will clarify the situation for design. The upper edge of the flexible membrane would be terminated along the crest coping wall with a similar mechanical joint to that envisaged at the plinth. The suppliers may also suggest a more economical welded detail based on their experience elsewhere.

The ability of the existing cut off trench detail to perform under increased hydraulic gradients is not established. Some enhancement in flow path control is to be expected, especially for redevelopment options that have significantly increased reservoir elevation. We have assumed for raising concept design that that grouting and upstream plinth extension will probably be required.

5.6 Concept Scoping Drawings

Taking account of factors outlined above, scoping of the works required for the 3 raising and 2 standalone embankment dam concepts has been prepared so that preliminary options to identify key technical aspects requiring further development, and to enable preliminary construction costing figures to be prepared. Copies of the sketch drawings illustrating the key features are appended.

We have compiled preliminary take off quantities from the 3D computer models of the options represented on the drawings for the purpose of determining the overall scope of the options

5.7 Comparison of Option Scope

As introduced in our methodology outline earlier, we have taken the opportunity to refer to the Lee River project where a similar rockfill dam has been proposed and design has progressed to a detailed stage. This project has also been the subject of several independent cost estimation reviews, including valuation the risks and remaining uncertainty. The high degree of confidence now established in the construction cost of this project is potentially of value to the redevelopment of Falls Dam, provided the similarities and differences between the projects are well understood. We have included the previous CFRD prefeasibility options (Opus 2012) in this exercise to illustrate how the various options and versions of embankment dam development relate to one another.

The following graphical figures have been included to illustrate the overall scope of the Lee River project relative to the Falls Dam embankment redevelopment options. We have included the incremental steps that might occur if the construction of the current redevelopment options were to be staged.

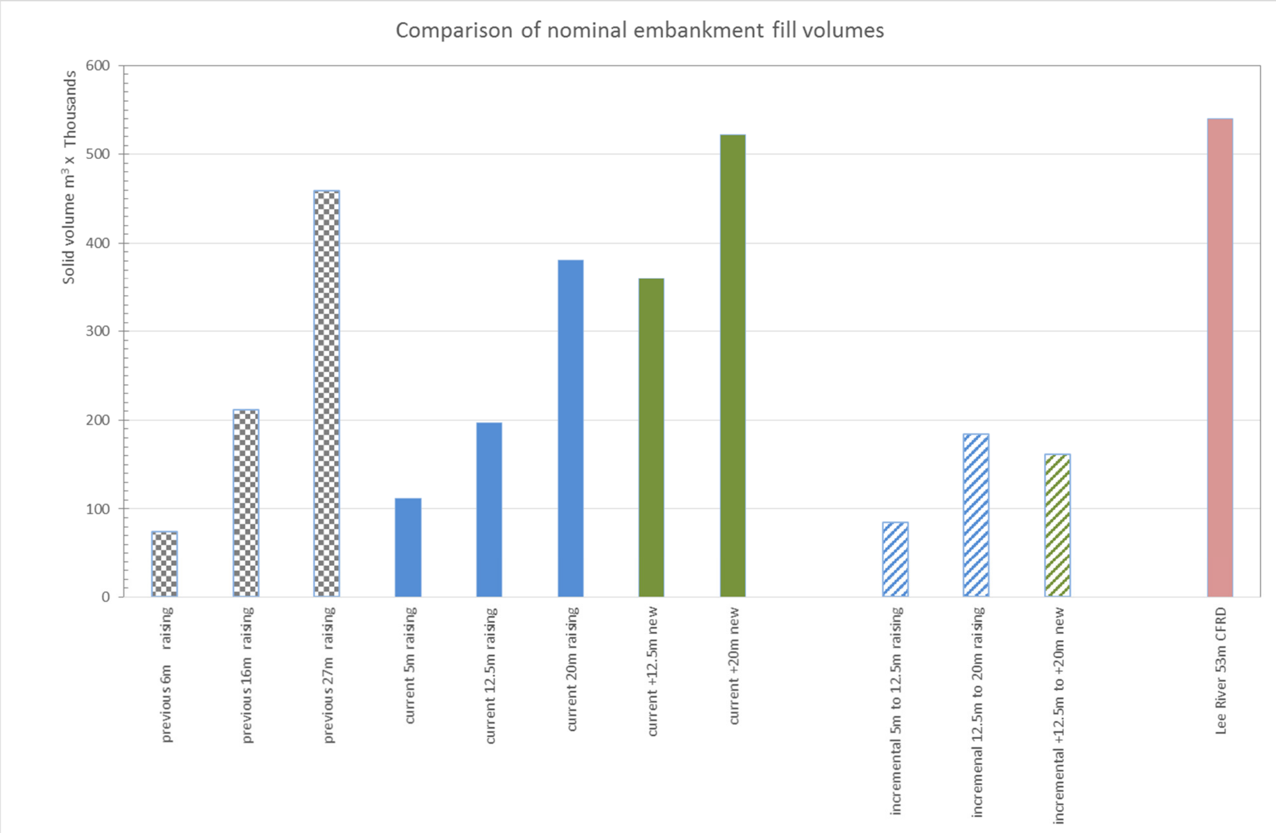

Figure 5-1 Relative embankment rockfill volumes

At 53m high, the Lee River Dam is very similar to the 20m increased water level development case at Falls Dam, and it can be seen from the above figure that the overall rockfill volume is also comparable.

Site conditions at Lee River are more challenging than those at Falls Dam, with excavation of up to 12m required on the plinth line to reach sound foundation rock. Grouting to 30m depth below this excavated level is also required to treat the foundation.

The upstream membrane area is another measure that indicates the relative scope of the project. Lee River dam design is based on a conventional slipformed conctrete facing, but the composite geosynthetic alternative is also being considered as a value engineering measure there.

The scope of membrane work within the options is presented in the figure below.

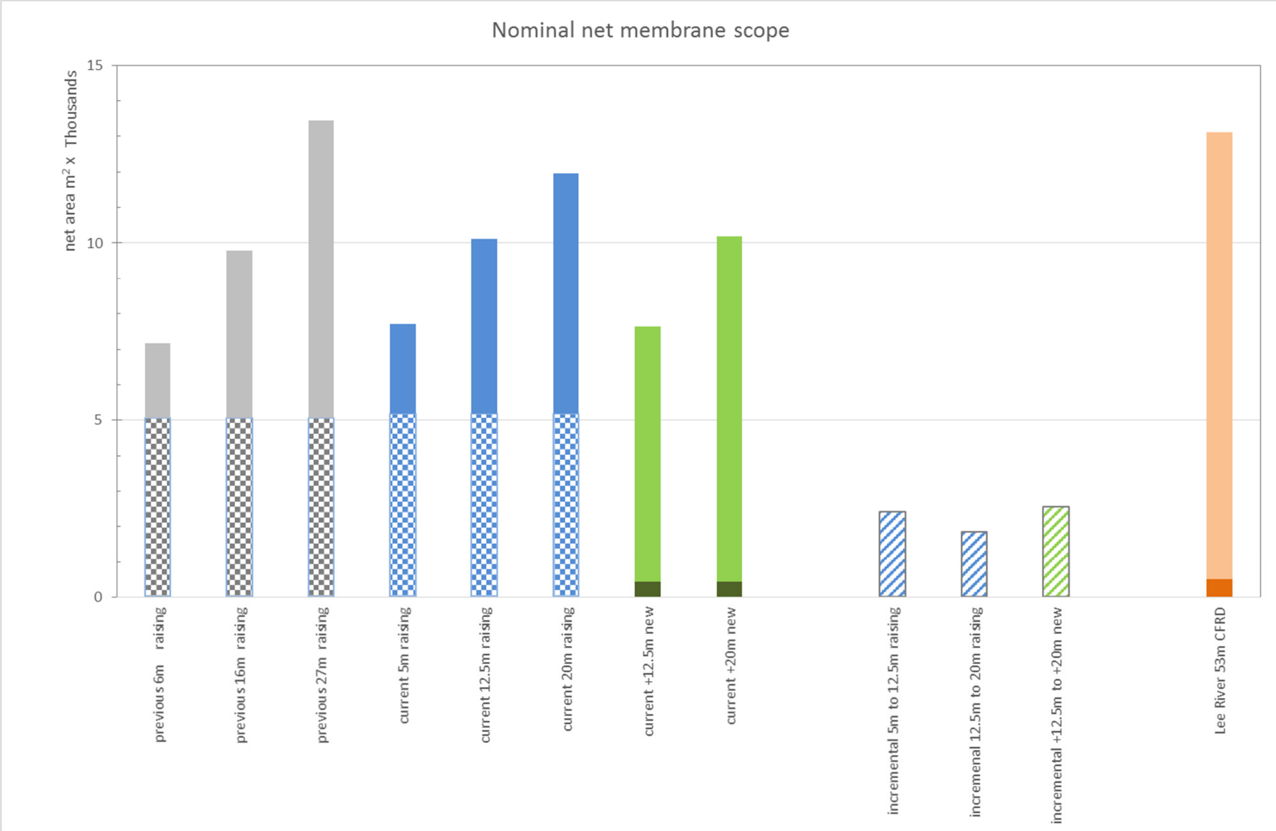

Figure 5-2 Relative membrane scope

The checkerboard hatching refers to the existing membrane area. The membrane requires repair, and it has been assumed that covering with PVC composite will be the most practical treatment subject to dewatering considerations.

The above figure illustrates that the downstream dam site is quite efficient in terms of the scope of membrane work required. The figure also reveals that the scope of membrane work for the equivalent Falls Dam redevelopment option is expected to be a little less than that required at the Lee River site.

Other points of scoping comparison have been briefly discussed within the preceeding text, including the diversion conduit, concrete lined spillway etc. Generally these elements are quite comparable in scope between the Falls Dam +20m options and the Lee River project. Key differences relate to the presence of the existing assets, both in terms of the favourable ability to be directly utilised, and the less favourable aspects of needing to achieve building code compliance for works that may be integrated into the permanent redevelopment. On this basis it is option 5, the stand alone +20m redevelopment scenario that is most directly comparable to scope of the Lee River project. The other redevelopment scoping options require a greater degree of interpretation concerning the differences in scale and approach.

5.8 Preliminary Costing Update.

As a key objective of this update study is to provide direction on expected construction costs, we have used the robust cost estimate total for the Lee River project as a basis for independently compiling a top down costing for the similarly scoped redevelopment option 5. We have made adjustments for broad elemental differences in scale, and we have considered the relative site characteristics and differing stages of design development.

Taking these factors into account, our view of the option 5 construction cost is that it should not exceed the comparable $80M risk adjusted budget developed for Lee River project, and there is potential for the actual costs to come in below this figure once the design and constructability aspects are further developed. The most influential factor in arriving at this opinion is the potential for tunnelling costs associated with the diversion extension to offset the otherwise lower construction costs applying to Falls Dam compared to Lee River. We are of the view that if the tunnelling solution shown proves to be too costly, there is the potential to revert to the fall back structural conduit solution similar in scope to that envisaged at Lee river, and not too dissimilar in concept to the lighter structural conduit approach adopted in the Golder RCC feasibility design.

The $80M figure is GST exclusive and includes allowance for .... In escalation terms the figure relates to first quarter 2015 costs.

Comparable risk adjusted totals for the other scoping options 1 to 4 are ....

By way of further comparison, the three earlier prefeasibility options prepared in 2012 have been updated to the same 2015 values. The risk adjusted totals are ....

The liabilities associated with both deferred maintenance of the existing assets and the mitigation of identified deficiencies in the existing assets have also be simply updated from earlier reporting. The risk adjusted figure in terms comparable to the redevelopment costs summarised above is........

6 Preferred Option(s) for Feasibility Study

6.1 Selection of Redevelopment Option(s)

In terms of looking forward to a feasibility study phase, three key questions need to be addressed as part of the option selection process.

- Are the indicated preliminary costs / benefits / risks sufficient to warrant further investment in progressing the embankment dam redevelopment project at this site through to feasibility level?

- Do the challenges and risks associated with redeveloping the existing assets (i.e raising options) balance the potential cost savings compared to the a new standalone option?

- Does staging the redevelopment present potential commercial advantages over a single construction package?

I need to discuss the answers to these questions as they are not simply matters for engineering consideration.

6.2 Key Items for Further Investigation

The matter of the maximum flood inflow hydrograph for design requires finalisation. Various views have been expressed on suitable design parameters, but in our view the upper bound or PMF value has not been authoritatively established. This is most relevant to the higher redevelopment options, but finalisation of the design discharge requirement for the current dam is also important in terms of quantifying any safety deficiency. This also applies to the 5m raising option in a similar context. New spillway chute intended for the 12.5m and 20m water level increase options can be designed to accommodate a revised peak discharge, but the design criteria needs to be established.

Topographic information has some uncertainty when matching historical records with the latest Golder's model. The datum adjustments aspect has been clarified, but the derived basin volumes still have too much variation between methods. Clarification of the differences will be important in terms of detailed water balance modelling and supply reliability. More detailed topographic definition of the potential dam footprint(s) will also be desirable when developing the designs. This includes understanding of the river bed below the falls.

Recent seismicity assessment for the RCC option has been assessed as imposing very severe loading demands on the dam. The deterministic 84%ile peak ground acceleration for design purposes is around double that applied to the Lee River site, although the controlling earthquake magnitudes and distances from the site are similar. These findings would benefit from specialist review before undertaking detailed seismic resilience analysis in subsequent phases.

We are of the view that while the existing 9.5Mm3 impoundment may be confirmed as a MEDIUM PIC rating, it is quite likely that the larger impoundments associated with redevelopment will attract a HIGH PIC rating. This matter will require attention in any feasibility phase, involving reassessment of the potential dam breach mechanism and quantification of the downstream effects of such a breach in both sunny day and background river flood scenarios. We note that breach mechanisms for embankment dams differ from those of concrete dams, and the ratings assigned to an equivalent storage option for one dam type cannot be automatically be applied to an alternative type.

Although feasibility reporting for the RCC alternative has been recently completed, there is little specific subsurface knowledge on the dam site, particularly in terms of rock defects and permeability characterisation for medium and high dam options. Stability of the existing steep rock bluff above the tunnel portal will require specific attention if the +12.5m raising option is to be pursued. Stability of the left bank area within the spillway chute footprint will also require investigation to confirm cut batter stability and excavation methodology. The higher embankment options will also require rock quarrying to complement the rockfill obtainable from the spillway cut. Suitable quarry locations will need to be determined such that constructability asp[ects are enhanced.

The scope of grouting or other foundation treatment required will need investigation. This may present challenges if raising options are to be pursued, as the areas of interest will generally be situated within the reservoir.

If the raising options are not ruled out, characterisation of the existing dumped and hand placed rockfill will be needed. Geophysical techniques may be applicable in terms of shear wave velocity etc., to assess the state of compactness and the resultant stiffness characteristics to input to analytical models to show compliance with required standards of performance.

Dewatering aspects require further detailed investigation and assessment, especially for the embankment raising options that are not isolated from the reservoir. The degree of siltation present near the dam, along with detailed analysis of summer flood inflow risk during any period of lowered reservoir level and upstream coffer dam protection. The degree to which underwater work can be practically undertaken should be investigated, particularly in relation to work on the lowest portion of the existing membrane. Further to the practical construction implications of dewatering for raising the existing embankment, the irrigation supply implications and costs will need to be assessed relative to any saving that may be available by incorporating the existing assets into the permanent redevelopment.

The form and layout of offtake works will be dependent upon factors such as:

- the distribution works design
- the possibility of staging the redevelopment, and
- the inclusion of any hydropower scheme

There will also be water quality management considerations, especially for the higher options where reservoir stratification may become important. Any use of the existing spillway shaft and tunnel will need to take account of the lining condition, although we have presumed that no concepts involving pressurising the existing tunnel would be pursued.

Further consideration should be given to the potential strategic and economic benefits of staged redevelopment of the storage before embarking on the next phase. From an engineering perspective the opportunity to stage the mid to high dam redevelopment appears quite attractive, but the real driver for such an approach will be dependent upon the wider scheme development strategy. We highlight the potential for an unfavourable staging outcome if the modest raising +5m option is progressed while longer term mid to high level raising is still being considered. Lower elevation left abutment auxiliary spillway cutting could significantly impact on later development of a new higher elevation replacement spillway.

References

To be added

Appendix A

Hydraulic Review of Spillway Concepts

Falls Dam Pre-Feasibility Study Hydraulic Review of Spillway Concepts for Dam Raising Options

Falls Dam Pre-Feasibility Study

Hydraulic Review of Spillway Concepts for Dam Raising Options

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Contents

1 Introduction

This review briefly examines the range of hydraulic issues that will need to be considered in more detail for each spillway concept for the three alternative dam raising options for the existing concrete-faced rockfill (CFR) dam.

Initially previous flood estimates for Falls Dam are discussed and commented on. Broad assumptions are made about the design magnitudes appropriate to each dam raising option.

Then the existing bellmouth spillway is described and the hydraulic issues associated with it are discussed. This has relevance to the spillway concept for Option 1 for raising the existing dam.

Thereafter the spillway concepts for each of the alternative options for raising the dam (Options 2 and 3) are considered. Key risk factors are identified and preliminary spillway dimensions quantified.

Recent dam feasibility reports have used a different level datum to the old imperial construction datum. To avoid confusion about levels, this report refers to the relative change in level for each dam option. For example the change in Full Supply Level (FSL) for each dam raising option relative to the FSL for the existing dam is as follows:

- Existing dam 0 m increase in FSL
- Option 1 5 m increase in FSL
- Option 2 12.5 m increase in FSL
- Option 3 20 m increase in FSL

2 Flood Hydrology Aspects

2.1 Previous Flood Estimates for Dam

Several previous studies have investigated flood magnitudes for the dam.

Jowett and Horrell (1984)[1] correlated the 6 year long flow record for the Manuherikia at Downstream Forks gauging station (upstream of the dam site) with the 26 year long flow record for the Manuherikia at Ophir gauging station (downstream of the dam site). Van Rensburg (2007)[2] extrapolated the flood estimates for the dam obtained using this approach (assuming a Gumbel distribution) from an annual exceedance probability (AEP) of 1 in 500 down to an annual exceedance probability of 1 in 5000. The original flood estimates for the dam and the extrapolated flood estimates are summarised in Table 2-1.

Van Rensburg (2007)[2] also carried out a flood frequency analysis on the 20 year long (1975–1994) Manuherikia at Downstream Forks gauging station flow record and scaled the resulting flood

[1] Jowett, I and Horrell, G (1984). "Manuherikia Dam: Spillway Capacity and design Flood Study". Report prepared by Investigation Section, Power Directorate, Ministry of Works and Development, Wellington, September 1984

[2] Van Rensburg, J (2007). "Falls Dam Probable Maximum Flood". Draft report prepared by Opus International Consultants. August 2007.

estimates up based on catchment area $A^{0.8}$ as per McKerchar and Pearson's (1989) Regional Flood Frequency method.[3] The resulting flood estimates are summarised in Table 2-1.

Van Rensburg (2009) also estimated the magnitude of the Probable Maximum Flood (PMF) for the dam.

Brown (2013)[4] carried out a flood frequency analysis on the 41 year long (1971–2012) Manuherikia at Ophir gauging station flow record and assumed conservatively, based on a limited number of small-sized floods, that peak flood magnitudes at the dam were approximately 40% of the peak magnitude at Ophir. He summarised the flood estimates as a range and gave a predictive equation for flood magnitude as a function of annual exceedance probability.

Brown (2013) also cautioned against extrapolating flood estimates beyond a 1 in 200 AEP because of the limited length of the annual flood maxima series on which the estimates were based. While this is strictly correct in a statistical sense, extrapolation is required to obtain spillway design flood estimates, albeit with an appropriate degree of conservatism applied.

Table 2-1 Summary of flood inflow estimates for Falls Dam

Annual Exceedance Probability (1 in T)	**Flood Estimate (m^3/s)**		
	Jowett and Horrell (1984)	**Van Rensburg (2007)**	**Brown (2013)**
100	312	232	239 (190 – 290)
200	362	257	268 (210 – 320)
500	430	292	306 (230 – 370)
1,000	490*	319	334**
5,000	630*	390	401** (300 – 490)
10,000	~700	-	430**
PMF	-	1,300	-

* extrapolated by Van Rensburg (2007) based on a Gumbel distribution
** predicted using Brown's best fit linear regression equation

The range of flood estimates for each annual exceedance probability value in Table 2-1 is quite wide and needs to be reconciled moving forward. Current best practice for flood frequency analysis usually considers a range of flood frequency distributions and uses the one that provides the best fit to the annual maximum flood maxima data. None of the three approaches described above appeared to consider more than one distribution.

Of the approaches, the one used by Jowell and Horrell (1984) is probably the most robust although it only used data from a 6 year long record for the Manuherikia at Downstream Forks gauging station. More weight is therefore placed on their flood estimates than those of the other two more recent studies.

[3] McKerchar, A I and Pearson C P (1989). "Flood Frequency in New Zealand". Hydrology Centre, Division of Water Sciences, Dept. of Scientific and Industrial Research, Christchurch.
[4] Brown, P (2013). "Manuherikia Valley Hydrology: 2013 Update". Report prepared by Aqualinc for Manuherikia Catchment Water Strategy Group, Report C14000/1, September 2013

Van Rensburg's (2007) PMF inflow estimate of 1,300 m^3/s also appears on the high side. Further work needs to be done to provide a more robust estimate.

Flood routing by Van Rensburg (2007) also demonstrated that there is very minimal attenuation of peak flood inflows by the existing reservoir and a reservoir with the full supply level (FSL) raised by 6 m. It is reasonable therefore to assume that the peak reservoir outflow approximately equals the peak reservoir inflow under extreme flood conditions.

2.2 Design Flood Assumptions

Dam Option 1 involves a 5 m increase in the Full Supply Level (FSL) of the reservoir through dam embankment raising and morning glory spillway raising. For this moderate increase in dam height, it is assumed that the dam could still be classified as a MEDIUM Potential Impact Category (PIC) structure. This would require the dam to have a total spillway capacity equivalent to 1 in 5,000 or 1 in 10,000 AEP flood (NZSOLD, 2015)[5], i.e. ~600-700 m^3/s.

Dam Options 2 – 5 involve more significant increases in reservoir FSL through embankment raising and alternative spillway concepts. For these more significant increases in dam height, the dam would probably need to be classified as a HIGH PIC structure. In this case, the dam would be required to have a total spillway capacity equivalent to a PMF. For a 372 km^2 catchment at the dam site, the PMF is more likely to be in the order of 1,000-1,100 m^3/s based on comparison with previous PMF estimates for other drier-type Central Otago Catchments.

3 Existing Dam Spillway

3.1 Description of Spillway Concept

The design and construction of the existing Falls Dam (including its bellmouth spillway) was described in a couple of papers published in the Proceedings of the NZ Society of Civil Engineers in 1937[6] and 1938[7] respectively. The design of the spillway was unique in that it was tested with the aid of a physical hydraulic model. The model was used to refine and fine-tune the spillway design.

The spillway incorporates:

- a large diameter bellmouth entry with a rounded weir lip;
- six round-nosed vanes arrayed symmetrically around the perimeter of the bellmouth entry to promote radial flow into the spillway throat and dropshaft and to prevent vortex occurrence;
- a spillway throat section with a gradually varying diameter linking the bellmouth entry to the vertical drop shaft;
- a short constant diameter vertical spillway dropshaft section;
- a ninety degree circular bend at the foot of the spillway dropshaft;
- a transition section from the circular cross-section of the dropshaft bend to the horseshoe shape of the outlet tunnel; and
- a long outlet tunnel with a relatively steep gradient and a free discharge at the exit.

[5] NZSOLD (2015). NZ Dam Safety Guidelines, New Zealand Society on Large Dams, 2015
[6] Gilkison, J T (1937). "The Manuherikia Rock Fill Dam ". Proc. NZ Soc. Civ. Engrs., Vol. XXIII, pp 279-312.
[7] Park, A G (1938)> "Model Tests for a Bellmouth Shaft Spillway". Proc. NZ Soc. Civ. Engrs., Vol. XXIV, pp 139-145.

The outlet tunnel for the bellmouth spillway follows a straight alignment and connects into the original diversion tunnel constructed to facilitate dam construction.

3.2 Hydraulic Behaviour

The hydraulic behaviour of a bellmouth type spillway changes under differing conditions of head. Three types of hydraulic control can occur:

- weir control
- throat control
- outlet conduit control

These hydraulic control types are illustrated in Figure 2-1 below.

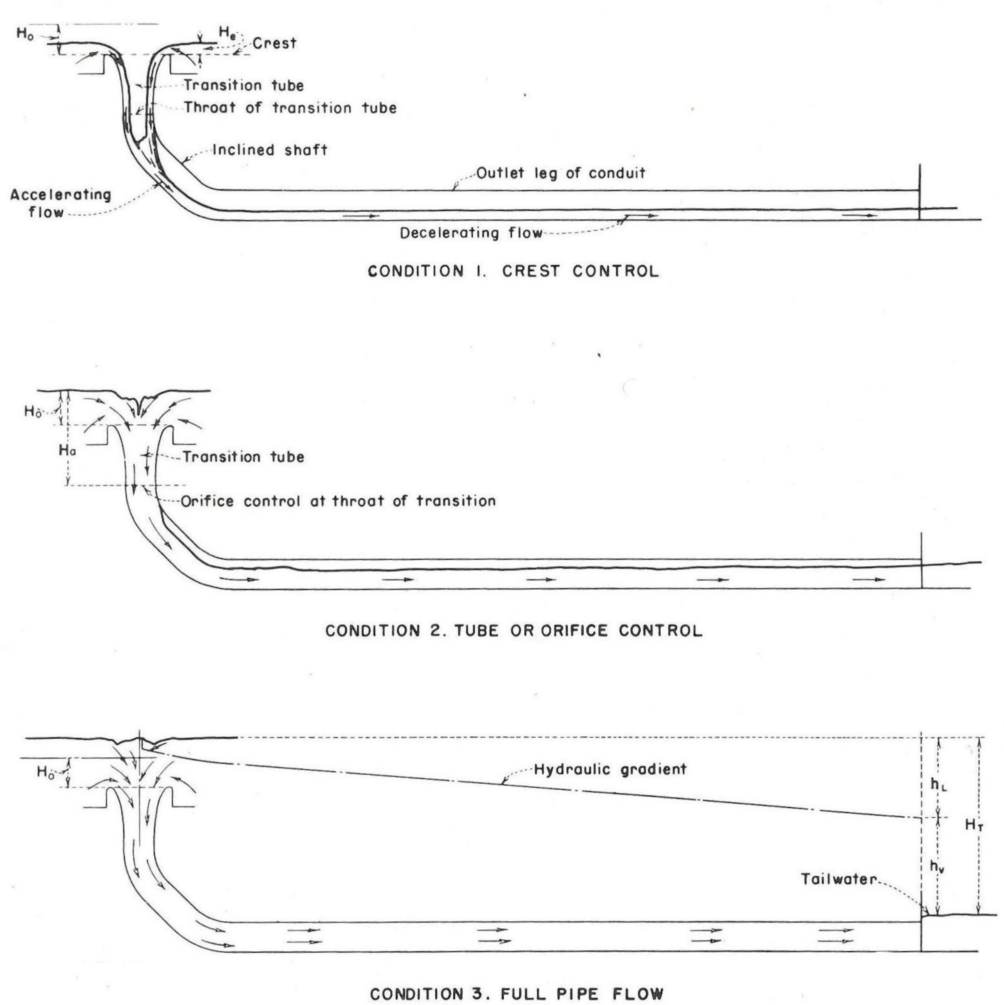


Figure 3-1 Flow behaviours of a bellmouth spillway (after *Design of Small Dams* (USBR, 1977)[8])

[8] USBR (1977). *Design of Small Dams*. Water Resources Technical Publication, Bureau of Reclamation, US Department of the Interior, pub. US Government Printing Office, Washington, 816p.

Weir control (condition 1 - crest control in Figure 3-1) occurs under small heads over the spillway crest.

As the head increases, control can shift to the dropshaft throat (condition 2 - tube or orifice control in Figure 2-1) so long as the outlet conduit continues to only run partially and is able to vent air above the high-velocity conduit flow. However, throat control would only occur if the dropshaft diameter was very small relative to the diameter of the bellmouth weir.

At large heads on the spillway crest, the capacity of the outlet conduit under open channel flow conditions is exceeded and the pipe flows full (condition 3 - Figure 3-1).

In the context of the existing Falls Dam bellmouth spillway, the diameter of the vertical dropshaft is relatively large so that throat control (condition 2) would never occur. In a very extreme flood, the spillway flow would transition from a weir control regime (condition 1) to an outlet conduit control regime (condition 3) as the upstream reservoir level rose and then from outlet conduit control back to weir control as the reservoir level fell.

3.3 Discharge Capacity

Figure 3-2 shows a previously calculated rating for the existing bellmouth spillway compared to the rating obtained from the physical hydraulic model study described by Park (1938). Spillway flow starts to transition from weir control (Condition 1 in Figure 3-1) to outlet conduit control (Condition 3 in Figure 3-1) at a flow approaching 400 m^3/s. Outlet conduit control is fully effective at a spillway discharge of about 425 m^3/s at a reservoir of 2.5 m above weir crest level (note the levels in Figure 3-2 are based on those for Watercare Services' Cosseys Dam for which the spillway was modelled on the Falls Dam one (Webby *et al*, 1995)).[9]

With outlet conduit control, the spillway discharge only increases very slightly as the head on the reservoir level increases. In other words, the spillway discharge for outlet conduit control effectively puts a cap on spill capacity which in this case is equivalent to about a 1 in 500 AEP flood discharge based on Jowett and Horrell's (1984) flood estimates.

[9] Webby, M G, Ong, S W and Devgun, M S (1995). "Spillway and PMF Review for Cosseys Dam". Report prepared by Works Consultancy Services for Watercare Services, Ref 5-9620A. MO, July 1996

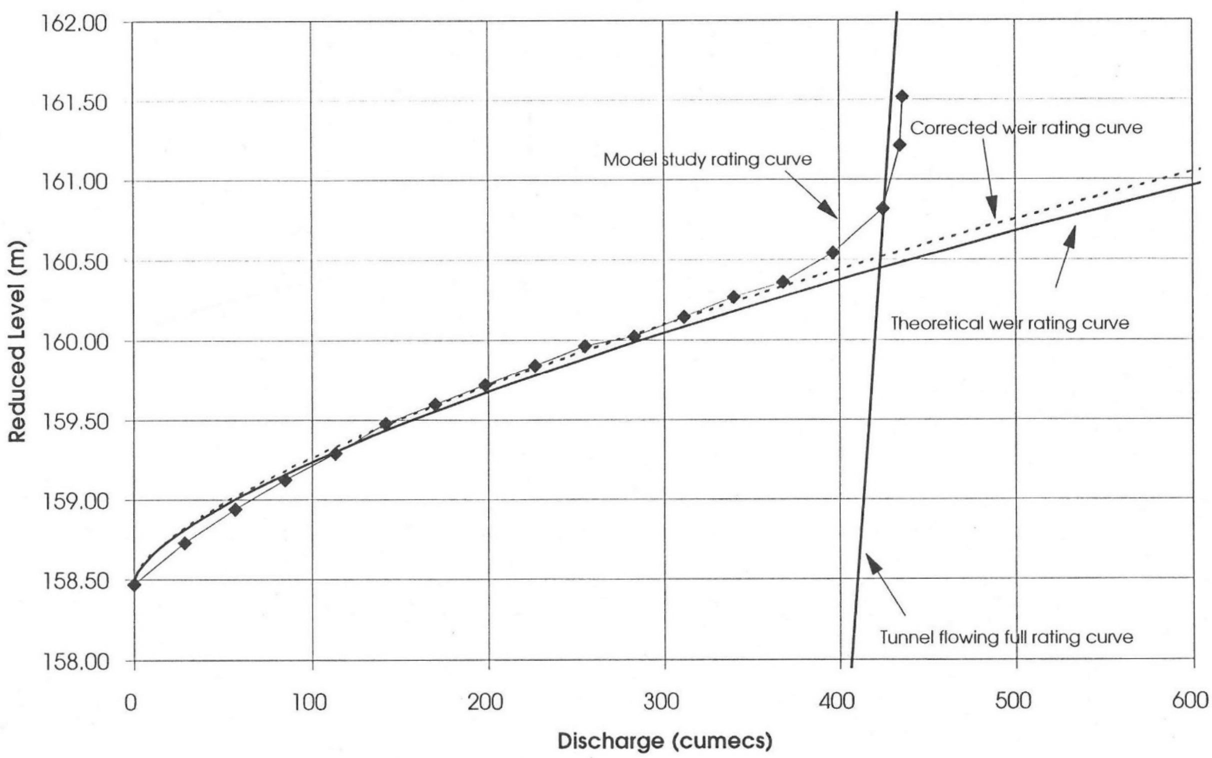


Figure 3-2 Discharge rating for existing bellmouth spillway (levels in terms of those for Cosseys Dam spillway (Webby *et al*, 1995))

3.4 Hydraulic Performance Issues

There are a couple of major hydraulic performance concerns with bellmouth spillways.

The first concern relates to the potential for air-entraining vortex formation at the bellmouth entry which results in undesirable rotating and surging flow in the spillway dropshaft and outlet conduit and reduces the discharge capacity of the spillway under weir control conditions. The potential for this type of behaviour is mitigated by providing conditions and features that promote the radial entry of flow towards the spillway bellmouth.

In the context of the spillway on the existing Falls Dam, the area surrounding the rock bench on which the bellmouth entry is sited has been excavated to minimise the restriction on flow entry to the bellmouth. More importantly though, the bellmouth entry incorporates six large vanes aligned to promote radially directed flow into the spillway dropshaft.

When a bellmouth spillway operates in the weir control mode with well-behaved radial flow entry conditions, the weir overflow will fall into the vertical dropshaft around the perimeter of the throat section with a central air core. This results in the occurrence of significant air entrainment by the falling spill flow. A second major concern then with bellmouth spillways is the potential for surging and explosive venting of entrained air that has become trapped in a large bubble in the outlet conduit, particularly during the falling phase of a flood when flow velocities through the outlet conduit gradually decrease. Explosive venting of a trapped air bubble has the potential to damage a bellmouth spillway.

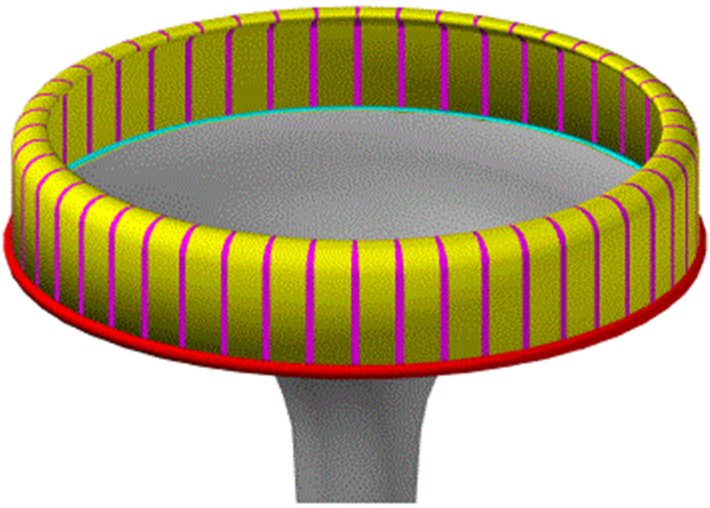
4 Dam Raising Option 1 – 5 m Increase in FSL

4.1 Description of Dam Raising Option

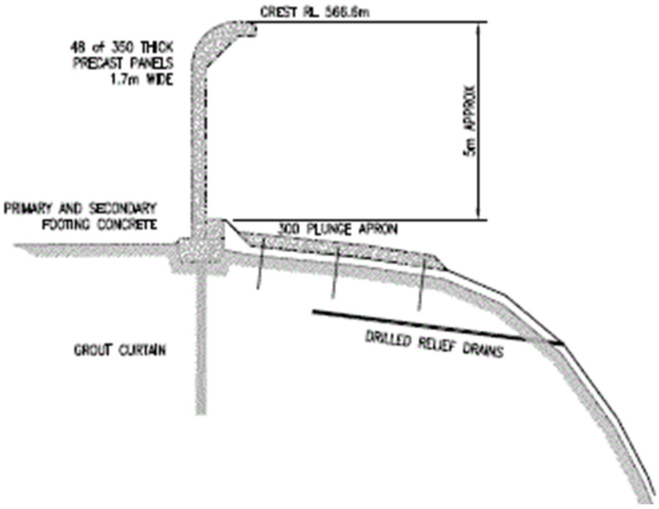
Option 1 involves raising the reservoir FSL relative to that of the existing dam by 5 m. The increase in crest level for the raised dam would need to be at least 5 m. The actual increase in dam crest level would be determined by the minimum freeboard requirement above the maximum reservoir level caused by the inflow design flood for this dam option (assumed to be between a 1 in 5,000 and 1 in 10,000 AEP flood).

4.2 Description of Spillway Concept

The spillway concept for this dam raising option comprises raising the crest level of the existing bellmouth spillway using fabricated steel ogee-weir segments. The individual steel weir segments would be roughly shaped like an upturned “J” with the shorter curved top leg of each unit forming the ogee-type weir crest. The weir segments would be bolted or welded together and would sit on the crest of the existing bellmouth weir crest. This is illustrated by Figures 4-1(a) and (b) based on the concept for a previous dam raising option.



(a) Isometric view of modified bellmouth spillway



(b) cross-section through modified bellmouth spillway

Figure 4-1 Spillway concept for Option 1 – height of pre-cast concrete elements limited to 5 m

4.3 Spillway Discharge Capacity

A discharge rating for the modified bellmouth spillway was previously presented in Van Rensburg (2007).[10] The discharge rating is very similar in form to that for the existing bellmouth spillway and is effectively translated vertically by the increase in weir crest level, despite the difference in structural form of the weir crest.

As discussed in Section 3.3, the effective maximum discharge capacity of the existing bellmouth spillway is in the order of 430 m^3/s. This will also be true of the modified spillway concept for Option 1. If the desired total spill capacity equivalent to a 1 in 5,000 or 1 in 10,000 AEP flood of ~600-700 m^3/s is to be achieved, then additional capacity of ~170-270 m^3/s would need to be provided through an auxiliary spillway.

It is envisaged that an auxiliary spillway could be constructed as an open rock excavation cut on the left abutment of the existing dam. The excavation cut would provide rock fill material for raising the dam. It is assumed that the excavation cut would be unlined with only limited securing work (e.g. rock bolting and dental concrete). The preliminary sizing of the auxiliary spillway is discussed further in Section 4.6 in relation to the erodibility of the excavation cut to high velocity spill flows[11].

4.4 Hydraulic Behaviour

A raised spillway based on the concept outlined above would in general behave hydraulically in a similar manner to the original bellmouth spillway. This assumes that splitter vanes would be incorporated within the fabricated steel raised weir structure to ensure radial approach flow

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[10] Refer Figure 6.1 of Van Rensburg (2007).
[11] The erodibility of rock surfaces under high velocity flow conditions is a function of rock type, hardness and jointing etc. It is related to the energy dissipation rate for water flows and compared against empirical data available in the literature for examples of eroded and non-eroded rock surfaces.

conditions into the spillway dropshaft (note that these vanes are omitted for the purposes of clarity in Figure 4-1 (a)).

However there would be two important differences between the hydraulic behaviour of the modified spillway and the original bellmouth spillway:

- Water flowing over the weir crest would fall in the form of a continuous curtain-type jet and impact on the gradually contracting throat section of the spillway dropshaft. This curtain-type jet would create an enclosed air void between it and the original bellmouth structure. This enclosed void would need to be vented to the atmosphere with a large pipe to prevent the falling water curtain from "flapping" and therefore being very noisy. Venting would mitigate the risk of structural vibrations being induced in the fabricated steel weir extension.
- The falling weir jet resulting from crest overflow would impact on the surface of the contracting throat section of the existing spillway dropshaft. The flow would be deflected by this impact and could potentially give rise to less stable flow conditions down the dropshaft than with the existing spillway.

The hydraulic behaviour of the modified spillway under outlet conduit control conditions would be unchanged from that of the existing spillway.

4.5 Key Hydraulic Performance Issues and Risk Factors

Key hydraulic performance and risk factors for the modified bellmouth spillway concept include:

- the need for large splitter vanes to be incorporated to ensure radial flow entry to the spillway and to minimise the potential for air entraining vortex formation;
- venting of void between the underside of the weir overflow jet and the inside of the circumferential weir to prevent unstable flapping of the falling weir jet and any noise and potential structural vibrations induced by this flapping motion;
- impact of the falling weir jet on the existing spillway throat section, deflection of the jet and the potential for unstable / surging flow conditions down the spillway dropshaft;
- air entrainment by the weir overflow down the dropshaft;
- the potential for air ingestion into the downstream tunnel and the potential for blowback (burping) incidents under falling discharge conditions;
- cavitation potential at the bottom of spillway dropshaft; and
- the potential loss of discharge capacity due to partial blockage by woody debris under extreme flood conditions.

Given the range of potential performance issues and risk factors with this modified spillway concept, it would be appropriate to conduct a physical hydraulic model study test of the concept if this dam raining option was selected.

As discussed above, the total required spill capacity for Option 1 is in the order of 600-700 m^3/s. The maximum discharge capacity of the modified bellmouth spillway is about 430 m^3/s under outlet conduit control conditions. The auxiliary spillway would therefore need to have a discharge capacity of 170-270 m^3/s to make up the shortfall in required total spill capacity.

Key performance issues and risk factors for an open rock cut auxiliary spillway would be:

- the discharge capacity of the unlined auxiliary spillway;

- hydraulic interaction of the approach flow to the modified bellmouth spillway and the auxiliary spillway chute and whether the discharge capacity would be compromised;
- the erodibility of the exposed rock surface under high velocity flow conditions;
- energy dissipation at the toe of the spillway;
- potential erosion of the toe of the spillway rock cut at the spillway / river interface;
- orientation of the spillway rock cut relative to the river alignment at the spillway / river interface; and
- erosion of the riverbed and opposite bank at the spillway chute outlet.

4.6 Preliminary Sizing of the Auxiliary Spillway

The following assumptions have been made with respect to the configuration of the raised dam crest, the modified bellmouth spillway and the auxiliary unlined rock-cut spillway:

- the minimum level for the top of the rockfill core of the dam is set at the predicted PMF level for the dam;
- the crest detail for the raised CFR dam consists of a capping on top of the rockfill core incorporating a concrete wave overtopping wall;
- the wave wall is high enough to provide the necessary freeboard for wind generated waves coincident with a PMF event;
- the modified bellmouth spillway is raised 5 m in line with the increase in reservoir FSL for this option;
- the auxiliary spillway crest starts to operate for floods larger than the 1 in 100 AEP flood (the 1 in 100 AEP flood was estimated to be 310 m^3/s by Jowett and Horrell (1984) – from Figure 3-2 this equates to a level of about 1.6 m above the crest level of the raised bellmouth spillway and 1.6 m above the FSL for this option).

Figure 4-2 shows discharge rating curves for a range of crest widths for the auxiliary spillway chute based on a discharge coefficient of 1.54 $m^{1/2}/s$. As noted in Section 4.3 above, the auxiliary spillway chute is required to provide an additional spill capacity of 170-270 m^3/s.

Table 4-1 summarises the minimum freeboard to the top of the rockfill core of the dam required for each spillway chute crest width considered. This assumes that the wave wall on top of the dam crest is high enough to contain any wind-generated waves coincident with a PMF event.

Table 4-1 Estimated minimum freeboard to the top of rockfill core of dam for different auxiliary spillway chute widths (Option 1)

Auxiliary Spillway Chute Width (m)	Required Auxiliary Spillway Capacity (m^3/s)	Head on Auxiliary Spillway Crest (m)	Height of Auxiliary Spillway Crest above FSL (m)	Minimum Freeboard (m)
30	170-270	2.4-3.3	1.6	4.0-4.9
40	170-270	2.0-2.7	1.6	3.6-4.3
50	170-270	1.7-2.3	1.6	3.3-3.9

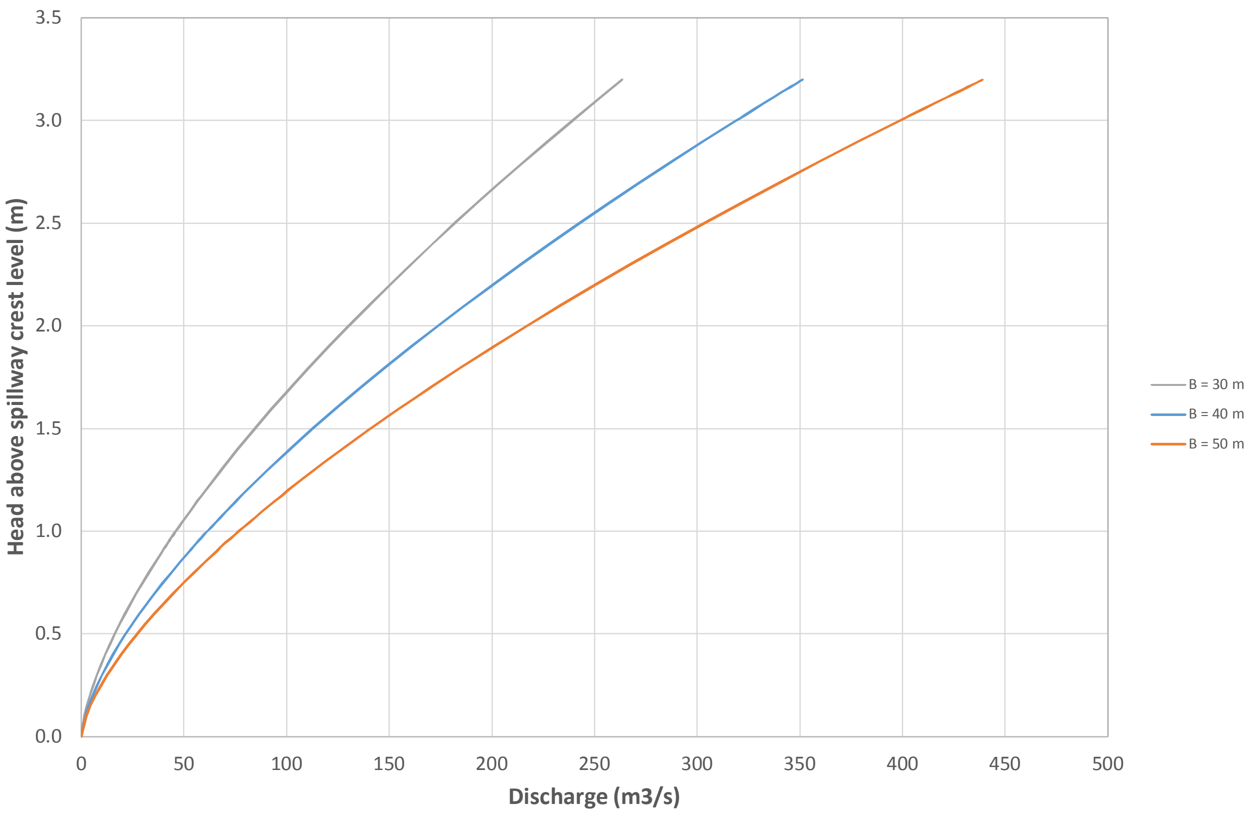

Figure 4-2 Discharge rating curves for unlined rock-cut auxiliary spillway chute for Option 1

Table 4-2 shows that the required freeboard to the top of the rockfill core of the dam for Option 1 is a trade-off with the crest width of the spillway chute. The minimum freeboard is given as a range for each crest width value because of the uncertainty in the magnitude of the PMF for the dam. This underlines the importance of establishing a reliable estimate of the PMF for the dam.

The minimum freeboard could be reduced further by:

- increasing further the crest width of the auxiliary spillway chute; or
- lowering the threshold level at which the auxiliary spillway starts to operate.

To reduce the minimum freeboard on the dam to less than about 2.5 m, the crest level for the auxiliary spillway chute would need to be set at about the same level as the crest of the raised bellmouth spillway.

The spillway chute would need to have a minimum downstream slope of about 5% to ensure super-critical flow under all discharge conditions. Assuming the unlined rock-cut chute would only have minimal dental concrete to patch significant rock defects, the Manning's n surface roughness value for the chute would be about 0.040. If the chute width was assumed to be 40 m, the maximum equilibrium flow depths and velocities for a discharge of 170-270 m^3/s would be in the ranges of 0.9-1.2 m and 4.7-5.6 m/s respectively with a Froude number of about 1-6-1.7.

The quality of the greywacke and argillite rock on the alignment of the proposed auxiliary spillway would appear to be adequate to sustain these maximum flow velocities. This would need to be confirmed at the detailed design stage if this dam options was selected. The discharge point of the auxiliary spillway over the steep side of the valley into the downstream river would need to be reinforced to prevent erosion of the rock at that location.

5 Dam Raising Option 2 – 12.5 m Increase in FSL

5.1 Description of Dam Raising Option

Options 2 and 3 represent a staged development over time. Option 2 involves raising the reservoir FSL relative to that of the existing dam by 12.5 m with a new primary spillway formed by an uncontrolled chute spillway on the left abutment of the dam. Option 3 involves a further incremental increase in the reservoir FSL which would be achieved by constructing an ogee-crested overflow weir structure in the spillway chute constructed for Option 2.

The increase in crest level for the raised dam in Option 2 would need to be at least 12.5 m. The actual increase in dam crest level would be determined by the minimum freeboard requirement above the maximum reservoir level caused by the inflow design flood for this dam option (assumed to be the PMF).

5.2 Description of Spillway Concept

The spillway concept for this dam raising option comprises a new concrete-lined uncontrolled chute spillway similar to the spillway concept for the proposed Lee Valley Dam near Richmond, Nelson. It would be excavated in rock through the hillside forming the left abutment of the existing dam. The excavated rock for the spillway chute cut would be utilised for raising the existing dam for Option 2.

Figure 5-1 shows a longitudinal profile sketch for the chute spillway concept (note the roughly triangular-shaped extension at the spillway chute crest applies only to Option 3). The longitudinal profile of the chute spillway is indicated by the parallel blue (rock excavation cut profile) and red (spillway invert) lines in the sketch.

The spillway chute would incorporate the following features:

- a wide curved entrance starting approximately coincident with the back side of the existing bellmouth spillway;
- a gradually converging throat section with a very gradually rising invert level to the spillway crest;
- a narrow crest length (in the direction of flow) at the high point of the spillway with a fixed crest width and vertical side walls;
- a long straight, constant width, constant slope, concrete-lined chute with vertical side-walls on the downhill side; and
- a ski jump type flip bucket at the end of the long spillway chute, possibly with the same width as the chute.

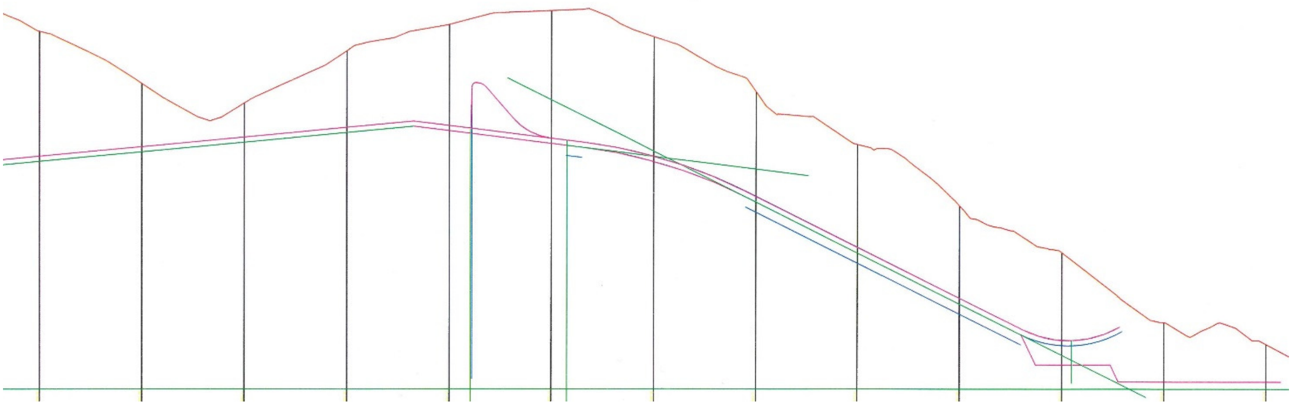

Figure 5-1 Longitudinal profile sketch of spillway chute concept for Option 2 (note the roughly triangular-shaped extension at the spillway chute crest applies only to Option 3)

The chute spillway in this concept would be able to be constructed while the original bellmouth spillway remained operational and the existing dam was being raised.

5.3 Spillway Discharge Capacity

Figure 5-2 shows predicted discharge ratings for crest widths in the range of 40-80 m based on a discharge coefficient value of 1.59 $m^{1/2}/s$.

The spillway for Option 2 would probably need to be able to pass a maximum discharge in the order of 1,000-1,100 m^3/s. Depending on the spillway crest width, the required reservoir head to pass this maximum discharge reduces as the crest width increases.

Similar assumptions have been made about the crest detail for the raised dam in Option 2:

- the minimum level for the top of the rockfill core of the dam is set at the predicted PMF level for the dam;
- the crest detail for the raised CFR dam consists of a capping on top of the rockfill core incorporating a concrete wave overtopping wall;
- the wave wall is high enough to provide the necessary freeboard for wind generated waves coincident with a PMF event;

Table 5-1 summarises the minimum freeboard to the top of the rockfill core of the dam required for spillway chute crest widths in the range of 50-80 m. This assumes that the wave wall on top of the dam crest is high enough to contain any wind-generated waves coincident with a PMF event.

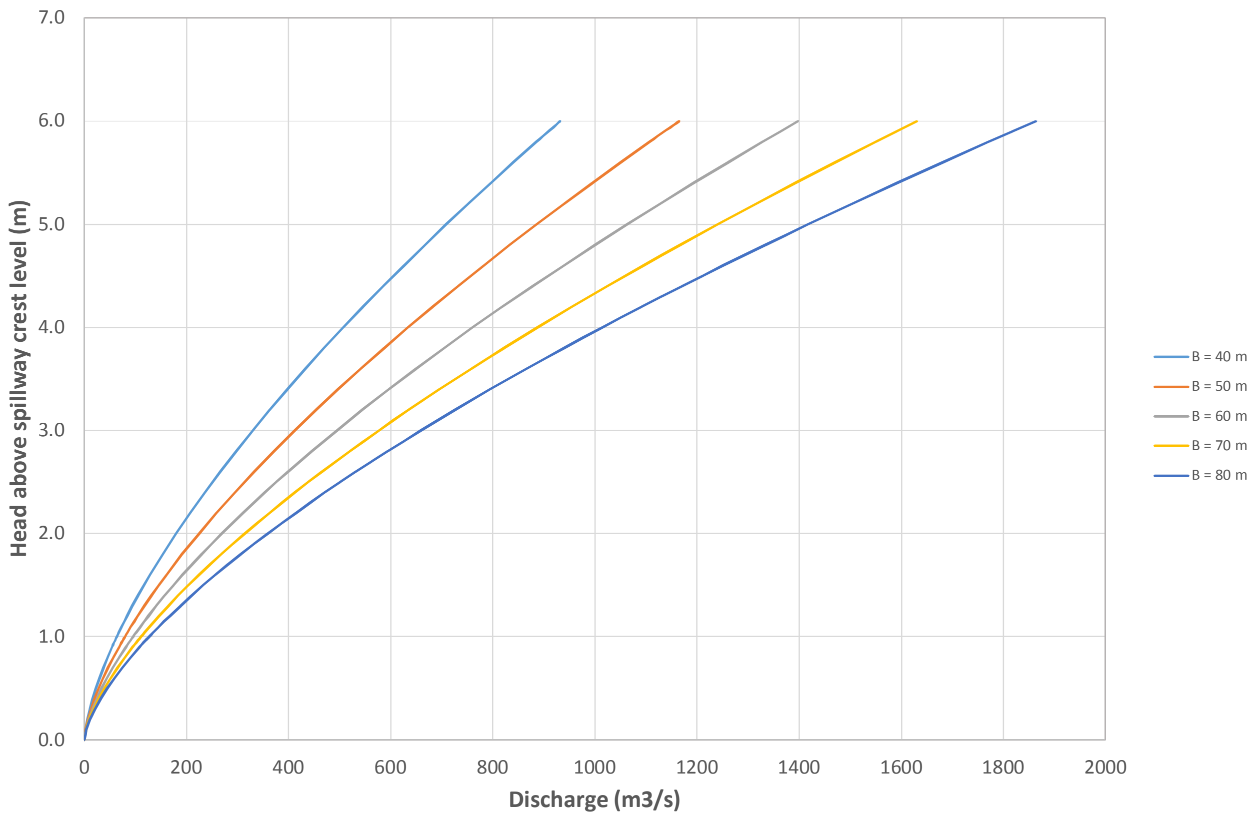

Figure 5-2 Discharge rating curves for uncontrolled spillways with different crest widths

Table 5-1 Estimated minimum freeboard to the top of rockfill core of dam for different chute spillway chute (Option 2)

Chute Spillway Crest Width (m)	Required Spillway Capacity (m^3/s)	Head on Spillway Crest (m)	Minimum Freeboard (m)
50	1,000-1,100	5.4-5.8	5.4-5.8
60	1,000-1,100	4.8-5.2	4.8-5.2
70	1,000-1,100	4.4-4.6	4.4-4.6
80	1,000-1,100	4.0-4.2	4.0-4.2

Optimising the spillway chute geometry for Option 2 would also be linked to optimising the volume of rock fill material required for raising the dam. Table 5-1 indicates that the chute spillway would need to have a minimum width of about 60 m to limit the minimum freeboard to the top of the rockfill core of the raised dam to about 5 m, or a minimum width of about 80 m to limit the minimum freeboard to about 4 m.

5.4 Hydraulic Behaviour

The wide curved entrance and the converging throat section of the spillway would steer the spillway flow through a change in flow direction and gradually accelerate it towards the spillway crest. The spillway crest forms a hydraulic control and marks the transition from sub-critical approach flow to super-critical (and high velocity) chute flow.

Downstream of the spillway crest, the spillway flow would gradually accelerate down the long spillway chute slope. At some point down the slope, the flow would become aerated (denoted by a marked change in surface colour to the white of aerated water). This aeration increases the depth of flow so that the side-walls of the chute need to be high enough to contain the chute flow at the design discharge. Irrespective of this, the spillway chute flow would generate a substantial amount of spray.

At the bottom of the long spillway chute, the flip bucket would deflect an aerated water jet upwards at a low angle. Additional aeration of the deflected jet would occur through the air which would assist in dissipating some of the residual energy of the chute flow. The deflected jet would follow a low trajectory and impact on the elevated river flow downstream of the dam. Over time the flip bucket jet would form a downstream plunge pool in the river bed.

Care would need to be taken during design to ensure that the falling jet did not impact on the opposite river bank and that the formation of the plunge pool over time did not have any other adverse effects. This would require careful consideration and optimisation of the flip bucket location, orientation and geometry.

5.5 Spillway Chute Geometry

It has been assumed that the spillway chute downstream of the spillway crest would be fully concrete-lined. Allowing for construction joints between the slabs making up the chute invert, the Manning's n surface roughness of the spillway chute would be about 0.015.

To achieve an economical design for the chute spillway, the side-walls downstream of the spillway crest would need to converge gradually to a narrower chute width. Table 5-2 summarises estimated flow depths and velocities for a PMF spillway discharge of 1,000-1,100 m^3/s for a range of chute widths after uniform flow conditions had become established.

Table 5-2 Estimated maximum flow depths and velocities assuming uniform flow conditions for PMF spillway discharge of 1,000-1,100 m^3/s for range of spillway chute widths (Option 2)

Chute Width (m)	Spillway Discharge (m^3/s)	Maximum Flow Depth (m)	Maximum Flow Velocity (m/s)
40	1,000-1,100	0.7-0.9	37-41
30	1,000-1,100	0.8-1.0	40-44
20	1,000-1,100	1.0-1.3	48-53

The maximum uniform flow depths and velocities in Table 5-2 allow for the effects of self-aeration of the spillway flow. At a spillway chute slope of 2:1 (H; V0 or 26.6°, the equilibrium average air (volumetric) concentration over the flow depth in the uniform flow region is estimated to be about 35%. The relative flow depth at 99% air concentration is very roughly 35% of the pure water depth in a very wide channel.

Maximum flow depths would be higher and maximum flow velocities lower than those in Table 5-2 as the flow gradually accelerates down the spillway chute between the spillway crest and where uniform flow conditions are established. The transitions at the start and end of the converging side-walls downstream of the spillway crest would cause the formation of shock waves. These shock waves would be transported further downstream, be reflected off each opposite side-wall and then be re-reflected again setting up a cross-wave pattern. The shock waves would locally increase the flow depth.

Table 5-2 shows that, the narrower the chute width is, the higher the flow velocities are in the uniform flow region. The magnitude of the flow velocities is such that cavitation of the invert of the spillway chute is highly likely to be an issue under PMF conditions requiring the incorporation of flow aeration ramps as a mitigation measure.

The magnitude of the flow velocities down the chute will significantly influence the trajectory of the jet projected into the air by the ski jump at the exit. The higher the terminal chute flow velocity, the further downstream the jet will impact on the downstream river channel. Froude numbers at the ski jump exit would be in the order of 14-15.

5.6 Key Performance Issues and Risk Factors

Key performance issues and risk factors for the chute spillway concept for Option 2 include:

- the required spillway discharge capacity;
- the spillway crest level and width needed to pass the selected design discharge;
- the horizontal alignment of the spillway;
- the longitudinal profile of the chute spillway;
- changes in alignment of the spillway chute walls and shock wave formation arising from sharp changes in alignment;
- self-aeration of the spillway chute flow and the increased depth of aerated flow;
- the height of the vertical side-walls to accommodate the height of shock waves and the aerated flow;
- cavitation occurrence down the spillway chute due to excessive velocities and cavitation indices exceeding critical values;
- the need for artificial aeration of the spillway chute flow using passive flow aerator devices to counter unacceptably high cavitation indices;
- the location, orientation and geometry of the flip bucket at the foot of spillway chute;
- the location of impact of the deflected jet from the spillway chute flip bucket; and
- the suitability of the riverbed at the point of deflected jet impact to sustain the formation of a plunge pool at that location in the longer term.

6 Dam Raising Option 3 – 20 m Increase in FSL

6.1 Description of Dam Raising Option

As discussed in Section 5.1 above, Option 3 represents a staged development following on from Option 2 at some stage in the longer term. It involves raising the reservoir FSL by 20 m above that of the existing dam, an incremental increase of 7.5 m on the reservoir FSL for Option 2. It would be achieved by constructing an ogee-crested overflow weir structure in the spillway chute constructed for Option 2.

The increase in crest level for the raised dam in Option 3 would need to be at least 20 m above that of the existing dam.

6.2 Description of Spillway Concept

The spillway concept for this dam raising option comprises the addition of an uncontrolled ogee-crest spillweir structure within the concrete-lined spillway chute proposed for the dam raising Option 2. The ogee-crested spillweir structure is indicated by the roughly triangular-shaped extension of the spillway crest seen in the sketch in Figure 5-1.

6.3 Spillway Discharge Capacity

Figure 6-2 shows predicted discharge ratings for crest widths in the range of 30-70 m based on a discharge coefficient value of about 2.1 $m^{1/2}/s$ for an ogee-crested spillweir structure at a design head equal to that for the PMF discharge.

As with the chute spillway for Option 2, the ogee-crested spillweir structure would probably need to be able to pass a maximum discharge in the order of 1,000-1,100 m^3/s. Depending on the spillweir structure crest width, the required reservoir head to pass this maximum discharge reduces as the crest width increases.

Similar assumptions have been made about the crest detail for the raised dam in Option 3:

- the minimum level for the top of the rockfill core of the dam is set at the predicted PMF level for the dam;
- the crest detail for the raised CFR dam consists of a capping on top of the rockfill core incorporating a concrete wave overtopping wall;
- the wave wall is high enough to provide the necessary freeboard for wind generated waves coincident with a PMF event;

Table 6-1 summarises the minimum freeboard to the top of the rockfill core of the dam required for spillweir structure crest widths in the range of 40-70 m. This assumes that the wave wall on top of the dam crest is high enough to contain any wind-generated waves coincident with a PMF event.

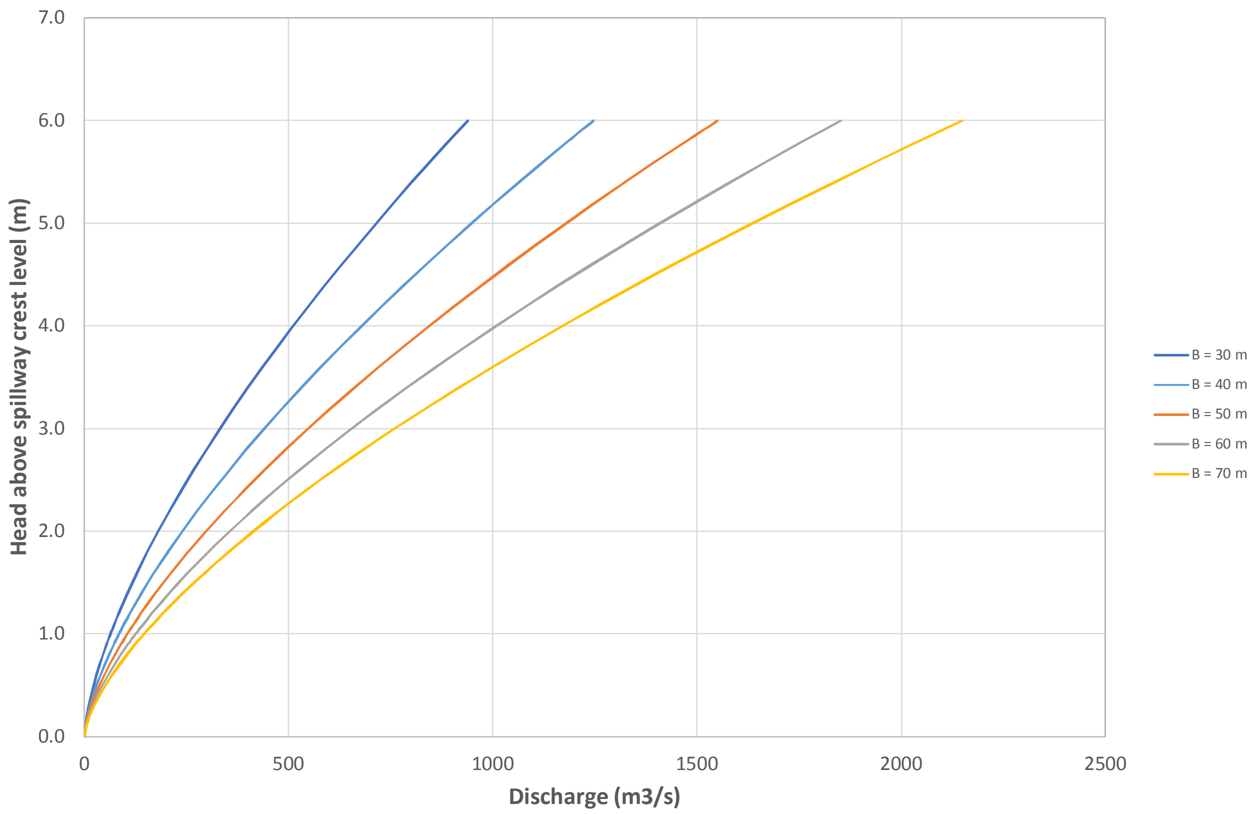

Figure 6-1 Discharge rating curves for ogee-crested spillweir structures with different crest widths

Table 6-1 Estimated minimum freeboard to the top of rockfill core of dam for different ogee-crested spillweir structures (Option 3)

Chute Spillway Crest Width (m)	Required Spillway Capacity (m^3/s)	Head on Spillway Crest (m)	Minimum Freeboard (m)
40	1,000-1,100	5.2-5.7	5.2-5.7
50	1,000-1,100	4.5-4.8	4.5-4.8
60	1,000-1,100	4.0-4.3	4.0-4.3
70	1,000-1,100	3.6-3.9	3.6-3.9

Table 6-1 indicates that the chute spillway would need to have a minimum width of about 50 m to limit the minimum freeboard to the top of the rockfill core of the raised dam to about 5 m, or a minimum width of about 70 m to limit the minimum freeboard to about 4 m.

6.4 Hydraulic Behaviour

The hydraulic behaviour of the modified chute spillway incorporating the ogee-crested spillweir structure would be very similar to that of the uncontrolled chute spillway for Option 2. However there would be some differences in behaviour as follows:

- the approach flow would be of much greater depth and have much slower velocities (hence the approach flow would be much less sensitive to the spillway entrance and throat geometry);
- the transition in spillway geometry between the toe of the ogee-crested spillweir structure and the invert of the downstream spillway chute could induce some degree of flow disturbance compared to the spillway chute flow in Option 2; and
- the residual energy requiring dissipation at the foot of the long spillway chute would be greater so that the spill flow jet deflected by the flip bucket at the end of the chute would impact further downstream than with Option 2.

6.5 Key Performance Issues and Risk Factors

The key performance issues and risk factors with the spillway concept for Option 3 would the same as for the chute spillway concept for Option 2.

However there would be some additional performance issues and risk factors that would require consideration as follows:

- the location of the ogee crested spillweir structure relative to the converging side-walls of throat section of the spillway;
- the optimisation of the crest level and width of the ogee-crested spillweir structure relative to the required spillway discharge capacity and the selected dam crest level and reservoir FSL;
- the convergence of spillweir flows down the side-walls of the ogee-crested structure and the transition to the flatter sloped spillway chute; and
- the change in location of the impact point for the deflected jet from the flip bucket at the end of the spillway chute.

7 Conclusions

Existing flood estimates for the dam site need to be reconciled before a design discharge can be established for any preferred dam upgrade option.

None of the performance issues and risk factors identified for the spillway concepts for each of the dam raising options would appear to be insurmountable or rule any of the options out from further consideration.

However the hydraulic risk factors associated with modification of the existing bellmouth spillway for Option 1 would appear to pose the greatest challenge. The modified spillway for Option 1 would only provide part of the required total spill capacity and would need to be augmented by an auxiliary unlined rock cut spillway chute through the left abutment hillside.

The crest width of the spillway chute has a significant influence on the minimum freeboard to the top of the rockfill core of the raised dam for Option 1 (assuming the dam crest level is set at the predicted peak PMF level for the reservoir and the wave wall on the dam crest provides the necessary freeboard for coincident wind-generated waves). This underlines the critical importance of being able to establish a reliable estimate of the PMF for the dam.

To reduce the minimum freeboard on the dam crest level to less than about 2.5 m for Option 1, the crest level for the auxiliary spillway chute would need to be set at about the same level as the crest of the raised bellmouth spillway.

The dam crest level has been assumed to be set at the predicted peak PMF level for the reservoir for both Options 2 and 3 while the wave wall on the dam crest has been assumed to provide the necessary freeboard for coincident wind-generated waves. Based on these assumptions, the minimum freeboard on the dam crest level for Option 2 would be significantly larger than the equivalent minimum freeboard value for Option 1. The minimum freeboard on the dam crest level for Option 3 would be slightly lower than that for Option 2 but still larger than that for Option 1

The performance issues and risk factors identified for the spillway concepts for Options 2 and 3 would appear to be less challenging than those for Option 1.

The spillway concept for Option 2 could be designed with that for Option 3 in mind as part of a subsequent second stage dam and reservoir upgrade in the future.

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