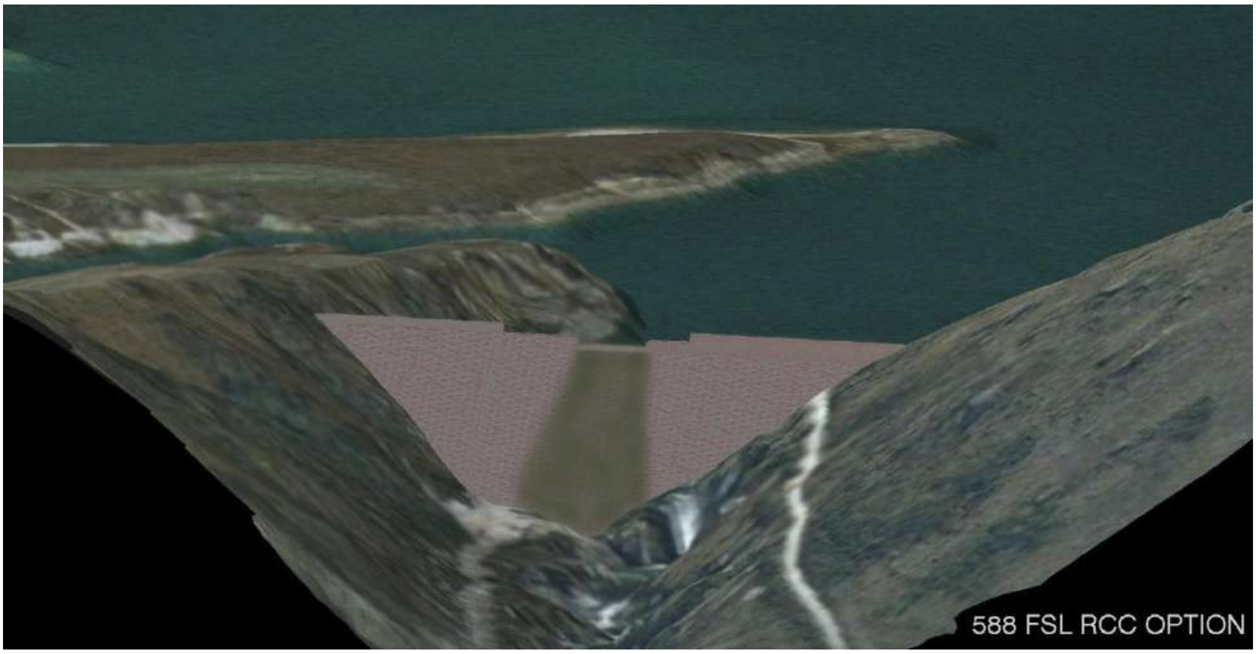
Manuherikia River Catchment
Water Strategy Study Stage 3

FALLS DAM REDEVELOPMENT
ENGINEERING PREFEASIBILITY STUDY

Report commissioned by Aqualinc Ltd

Manuherikia Catchment Water Strategy Study Stage 3 Falls Dam Redevelopment Engineering Prefeasibility Study



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Executive Summary

This report addresses the engineering aspects of a range of storage enhancement developments of the existing Falls Dam reservoir first filled in 1935. The Upper Manuherikia River water resource has potential for increased irrigation demand servicing than that provided from the existing storage of some $10Mm^3$, with $20Mm^3$, $50Mm^3$ and $100Mm^3$ options being covered. These storage volumes correspond to full reservoir supply level (FSL) increases above that impounded by the existing 33.5m high concrete faced rockfill embankment dam of some 6m (RL567.5m FSL), 15m (RL577m FSL), and 26m (RL588m FSL) respectively.

The existing dam site above the original falls is seen to present a most suitable situation for development. Engineering options examined involve either;

- raising the existing concrete faced rockfill (CFRD) embankment to accommodate the new reservoir level, or
- constructing a new roller compacted concrete (RCC) dam immediately downstream for the higher options, and utilising the existing facilities for construction diversion purposes.

Limitations in the extreme flood discharge capacity of the existing morning glory spillway is an important consideration for design, as is the in-service deterioration of the existing concrete facing and spillway tunnel lining. Retention of the morning glory spillway is possible for the lowest dam raising option considered, but not practical for the higher options due to hydraulic and structural factors. An ungated auxiliary spillway cutting on the true left abutment would provide enhanced flood discharge capacity for the lowest embankment raising option, and this would be enlarged to a full service spillway for the higher embankment options. The new concrete dam options would have an integral overspill ungated spillway, as is typical of RCC designs. A flood discharge capacity of 700 m^3/s has been adopted for scoping purposes in this preliminary study. This is a substantial increase on the current spillway choking limit of some $430m^3/s$. However, this design parameter is still subject to further consideration, as even this increase leaves flood handling well below the currently determined probable maximum flood (PMF) demand at this site.

The highest (RL588m FSL) options will require the construction of a saddle dam to prevent discharge down Shamrock Gully to the west of the existing site. This is envisaged as a zoned earth embankment structure. The present scope of this work is poorly defined, and specific investigation and design development is required to progress this aspect.

Development of irrigation storage would have a substantial impact on the $4m^3/s$ 1.2MW capacity Falls Dam hydropower scheme commissioned by Pioneer Generation Ltd in 2003. This factor is not covered in detail nor costed in this report, although it is assumed that hydropower assets will continue to be part of any enhanced storage scheme. Conceptual layouts are postulated, along with supply conduit arrangements to deliver the $4m^3/s$, $6.5m^3/s$, and $11m^3/s$ offtake flows corresponding to the three levels of storage increase respectively.

The selected CFRD and RCC dam options have been scoped to derive an indication of likely construction quantities. Construction rates have been derived from previously priced CFRD and RCC projects adjusted for cost escalation to 2nd quarter 2012 values. Land and property aspects

and resource consenting costs are not included in the construction estimates, and all figures exclude GST.

Even in the absence of investment in major storage development, there are significant potential liabilities associated with owning the existing ageing dam assets. Quantifying such potential liabilities has presented challenges as they are dependent on factors which are still open to a number of influences. However, these potential liabilities are an important factor in making a decision to invest in further development of the storage, so assumptions have been made to arrive at a valuation range. The issue is complex insofar as some approaches to addressing the potential liabilities lead logically to some dam raising. This situation arises as a result of generating “waste” rockfill through constructing an auxiliary spillway cutting to enhance extreme flood handling capacity. This aspect requires further detailed consideration in any subsequent feasibility study.

The summary tabulation below presents the preliminary estimated construction costs of the study options, along with the associated rates per additional cubic metre of storage.

Full Supply Level		**Raised CFRDam**		**New RCC Dam**	
		$M	**$/m^3**	**$M**	**$/m^3**
Development Options	RL588m	54.1	0.57	68.0	0.71
	RL577m	35.5	0.91	42.9	1.10
	RL567.5m	16.2	1.56	-	-
Current Potential Liabilities	RL561.5m To RL566.5m	7.2 To 11.3	- To 0.87	-	-

This summary reveals that the RCC options show a 20% to 25% premium over the equivalent CFRD options, which is consistent with the proportion of existing assets being retained in service. Although this difference is significant, full replacement of the aging assets is potentially attractive in the long term, and the RCC options should be carried forward into any feasibility study for further consideration of these preliminary costings.

The storage efficiency of the reservoir at higher elevations is also revealed in the summary tabulation, with the per cubic metre additional storage cost shown to be quite sensitive to the scale of the engineering development.

The manner in which any decision to progress this project takes the potential liabilities into account (or the “do nothing” scenario in storage development terms), is complex, and not fully represented by the simplified summary above. However, it is clear that consideration of this factor may be a significant influence on any investment decision.

Several design and construction factors condense out from the preliminary discussion presented in the report, leading to aspects requiring specific consideration in any subsequent feasibility study. These include

General

- Saddle dam topography and foundation conditions, especially the depth of stripping required.
- Saddle dam construction material sources and properties.
- Confirmation of impoundment PIC rating(s) and associated design criteria.
- Determination of specific design loading conditions at the site including extreme flood and seismic events consistent with confirmed PIC rating(s).

Raised CFRD Options

- Develop rock cutting spillway hydraulic and geotechnical design to suit rock mass quality, especially hydraulic aspects where the morning glory spillway is to be retained in a raised form in the path of the auxiliary spillway flow, and spillway lining requirements.
- Develop the offtake works general arrangement with a view to determining the specific investigations needed for input to the feasibility study.
- Develop construction site layouts such that practical access roading and working areas can be established for this construction method noting the restricted nature of the site.
- Extent of processing of rock to meet fill requirements in the context of a bulk spillway cut source.
- Determination of the significance of stiffness differentials, (i.e. original dumped vs new compacted embankment fills) and the effects of increased stresses on existing embankment fill.
- Significance of the deformed shape of the existing membrane due to accumulated settlement over the life of the embankment.
- Repair technique for the degraded membrane joints and exposed concrete, (including the existing spillway lining for the lowest option), especially given access constraints arising from the in-service reservoir and potential for below water repair requirements.
- Influence of increased reservoir elevation on the integrity of existing works such as the tunnel bulkhead and tunnel lining subjected to water pressure loading.
- The scope of foundation treatment needed for raised plinth works given the potential for reduced rock mass quality at higher elevations.

RCC Options

- Develop spillway hydraulic and geotechnical design to suit foundation rock mass quality, especially the transition into the receiving river channel noting the likely low tailwater level conditions.
- Develop the offtake works general arrangement with a view to determining the specific information needed to input to the feasibility study.
- Develop construction site layouts such that practical access roading and working areas can be established, especially the preferred quarry site(s) and RCC aggregate processing and mixing area(s), noting the restricted nature of the site.
- Investigation of RCC materials able to be economically produced on site.
- Investigation of potential cementitious materials to complement Portland cement.
- Nature and scope of decommissioning work required for the existing dam and tunnel assets, including the lowering of the existing embankment and the long term integrity of the tunnel lining and bulkhead under increased reservoir elevation.

Contents

Figures

1 Introduction

1.1 Manuherikia Valley Study Background

The irrigation development potential of the Manuherikia Valley in Central Otago is currently being investigated as part of Stage 3 of a comprehensive full catchment water strategy study. Aqualinc is leading this study, and Opus has been engaged to provide specialist engineering advice on matters pertaining to potential impoundment enhancements for the Valley. Opus has an extensive background in advising on the existing Falls dam impoundment originally created in the 1930's, and this role has extended to the current Falls Dam Company, and to Pioneer Generation as owners of the mini hydropower scheme retrofitted to the dam in 2003.

The Manuherikia River provides the principle water resource for the valley, and the upper valley storage potential is not fully utilised by the existing facilities at Falls Dam. The potential to enhance utilisation of the water resource through increased seasonal water storage in the upper catchment is the subject of this report. Other potential complementary or alternative storage options that may be present elsewhere in the catchment are not examined in this document, but may be the subject of other reports.

1.2 Previous Redevelopment Studies

Original development of the Falls dam impoundment during the 1930's great depression era included consideration of a much larger dam and reservoir than was actually constructed at that time [Gilkison 1937]. The possibility of increasing the 33.5m high embankment dam with its full supply level at approx RL561m, by some 25m to a total height of some 58m by extending the embankment downstream was scoped. This scale of development with an eight fold increase in storage capacity was scoped to fully service potential land development in the valley. While this conceptual work was not developed to a stage of engineering design suitable for commitment to construction, the dam embankment work that was completed was positioned to allow such further development. The records reveal the recognition at that time of the degree of potential for further development of both the water resource and the impoundment basin.

In 1974 the Water & Soil Division of the Ministry of Works investigated major development of the water resource [MoW 1974], including a new Falls dam downstream of the original to a new maximum reservoir operating level in the range RL 591 to 597m, requiring an embankment height exceeding 70m. The impracticality of significantly raising the morning glory spillway was recognised, and a new flood spillway at the Shamrock Gully Saddle Dam was considered, as an alternative to a new spillway at the dam site. Engineering scoping and construction costing supporting this predominately hydrological study was rudimentary, and even considering the costs of the day the estimates did not recognise the real nature of the construction work that would be required. The concept layout is shown on the figure below.

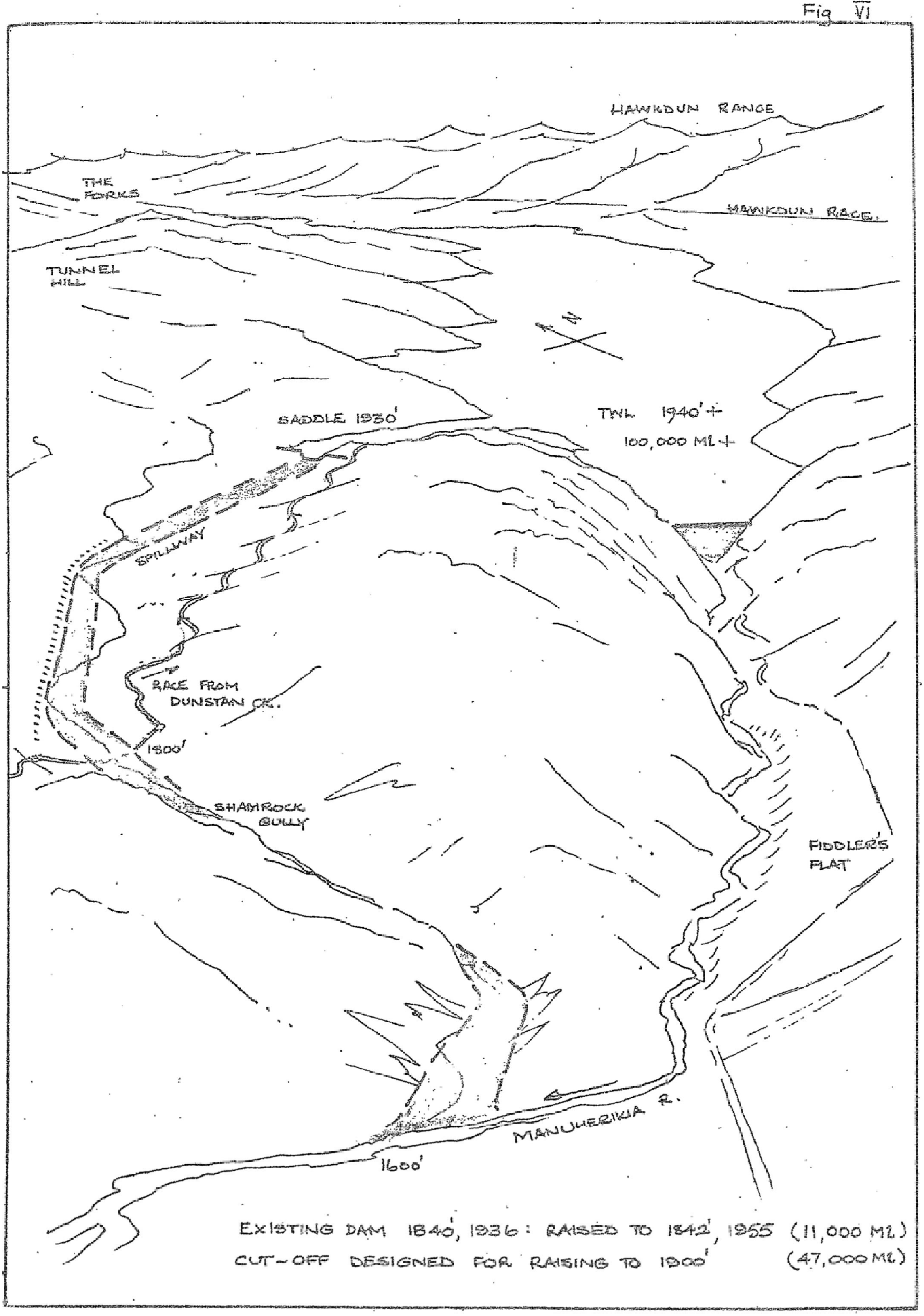
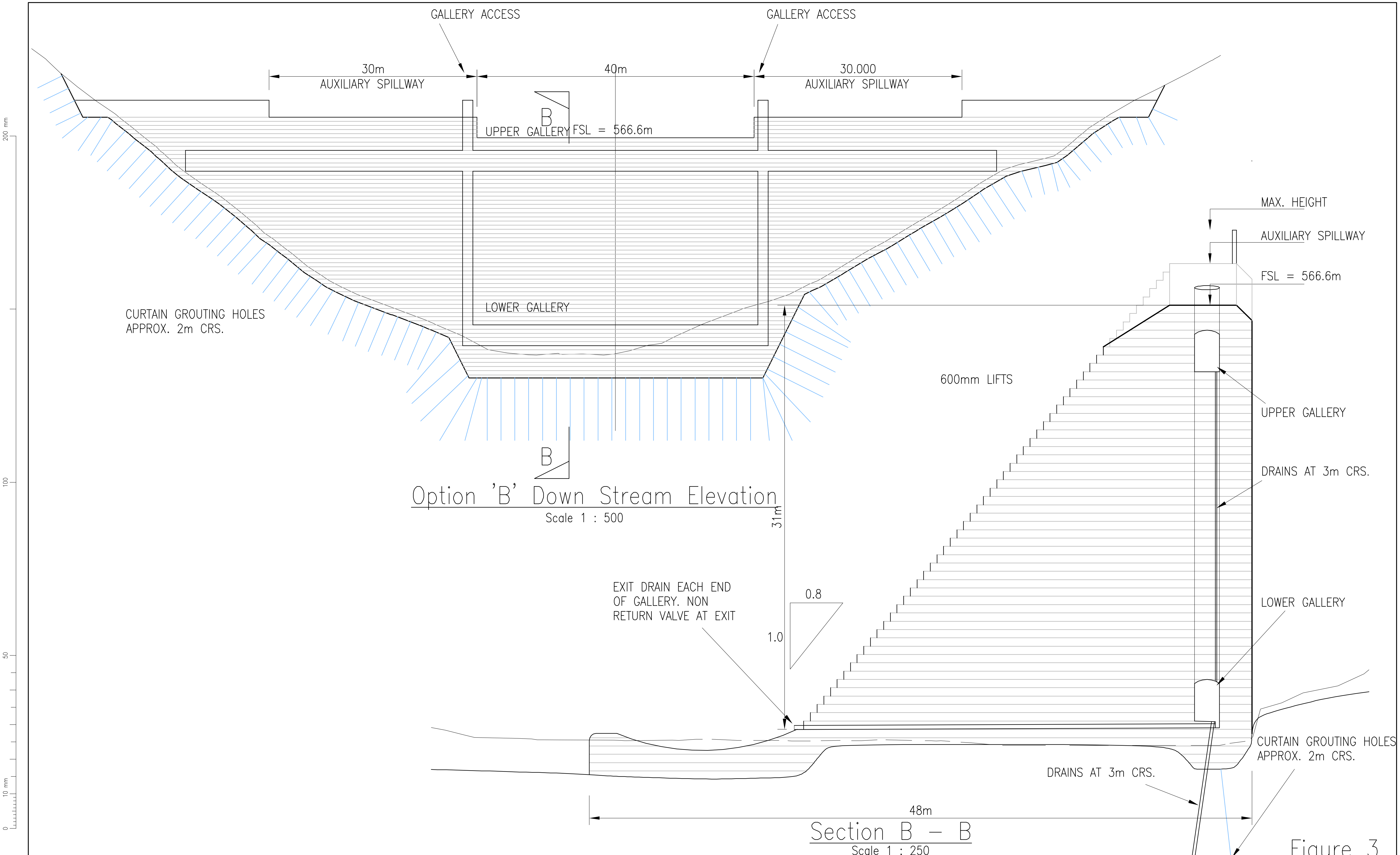


Figure 1 c1974 Irrigation Storage Redevelopment Concept

In more recent times since the Falls Dam Company has taken over responsibility for managing the Falls Dam assets, a series of engineering studies [OPUS 2003-06, 2010, 2011] have been undertaken examining various lesser scale dam raising proposals focussed upon improving the security of supply to existing water users. These studies did entail careful re-examination of the available water resource, through the creation of a long term synthesised inflow record [RAINEFFECTS 2002] and long term water balance modelling. These studies examined concrete faced rockfill dam (CFRD) embankment and spillway raising up to +8m. However, the engineering challenges were considered to become very significant above +5m due to the performance characteristics of the morning glory spillway, and replacement roller compacted concrete (RCC) overspill dam options constructed immediately downstream were examined up to some +12m above existing maximum operating level. This work on RCC dam options included replacement of the existing dam assets for the purposes of replacement insurance valuation. Improving safe flood discharge capacity under extreme events was also a key factor in these studies, and this aspect is elaborated on later in this report. Copies of the layout concepts considered at this time are shown on the figures below. The +5m CFRD dam raising option study findings have been updated for inclusion in this report for comparison with the specific options defined for this prefeasibility study.

Figure 2 Previous (+5m) RCC Replacement Dam Layout Concept



Option 'B' Down Stream Elevation

Scale 1 : 500

Section B – B

Scale 1 : 250

Figure 3

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			DESIGN	I G Walsh		06–06
			DRAWN	D Wilson		06–06
			APPROVED			
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FALLS DAM COMPANY

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TITLE Falls Dam Irrigation Co.
Falls dam
R. C. C. option For Insurance Costing

Option 'B' 5m Lift
Sections

STATUS	Preliminary	FILE	6cwi01.30		
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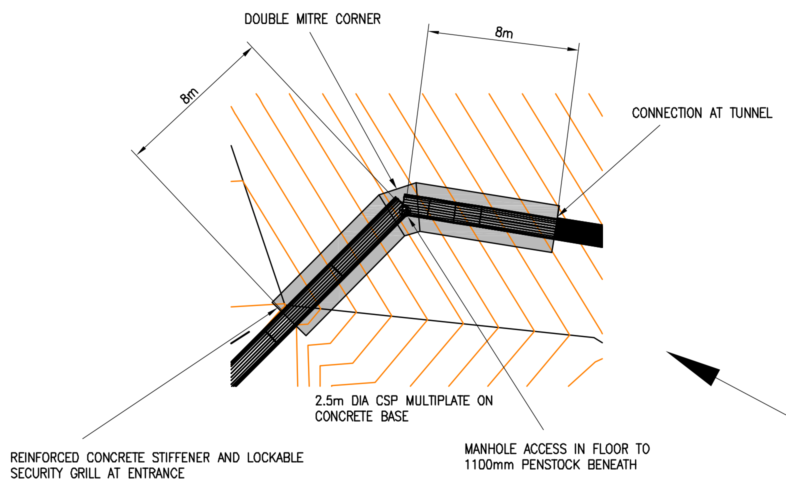

ACCESS TUNNEL EXTENSION – PLAN
1:20

NOTE : NEW TR POWERHOUSE ACCESS ROAD NOT SHOWN. GRADIENT 1 in 5 FROM RIGHT ABUTMENT CREST, 20m MINIMUM CURVE RADIUS, RAISED FORMATION TO RL 543m AT INTERMEDIATE HAIRPIN BEND.

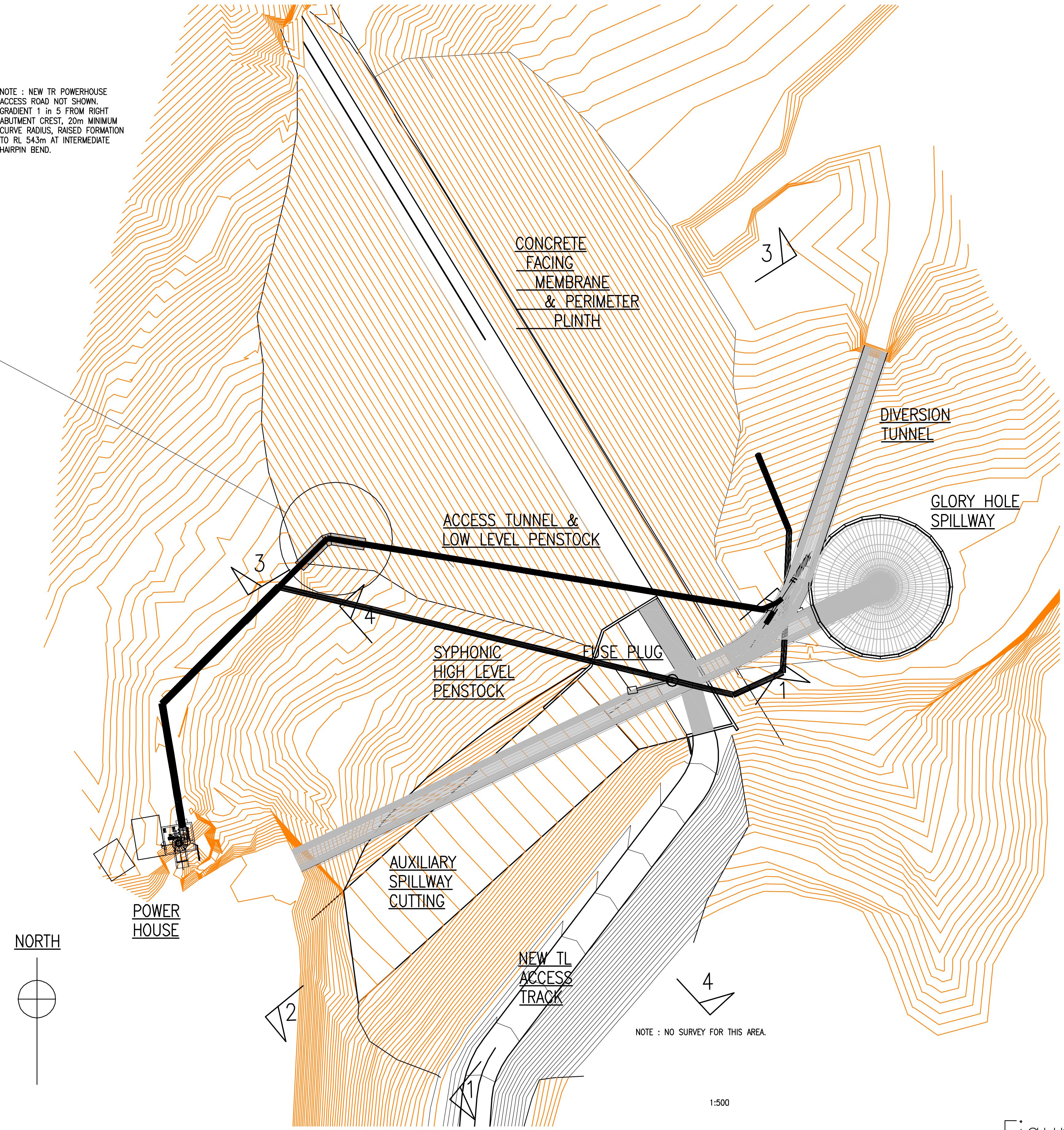

Figure 4

	BY	CHECKED	DATE
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DRAWN	I Walsh/D Wilson		8/04
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TITLE: FALLS DAM COMPANY
FALLS DAM
5m DAM RAISING CONCEPT
LAYOUT PLAN

STATUS	DRAFT	FILE	CWI01.30		
SCALE	PLOT DATE	FEATURE IDENTIFIER	CODE	SHEET	REVISION
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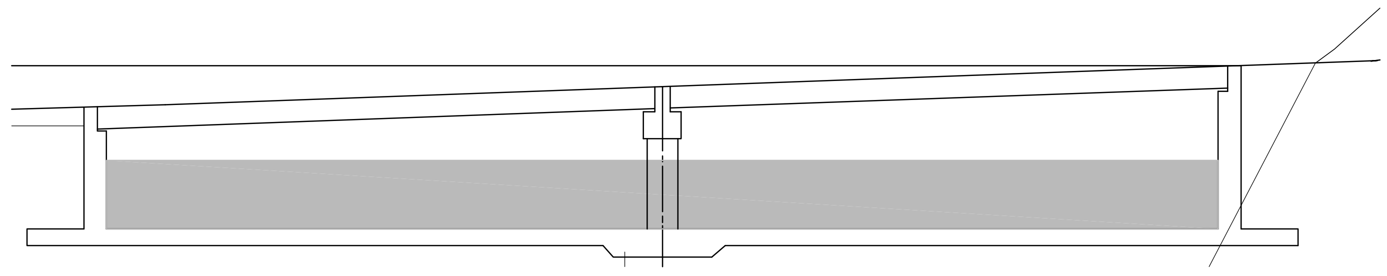
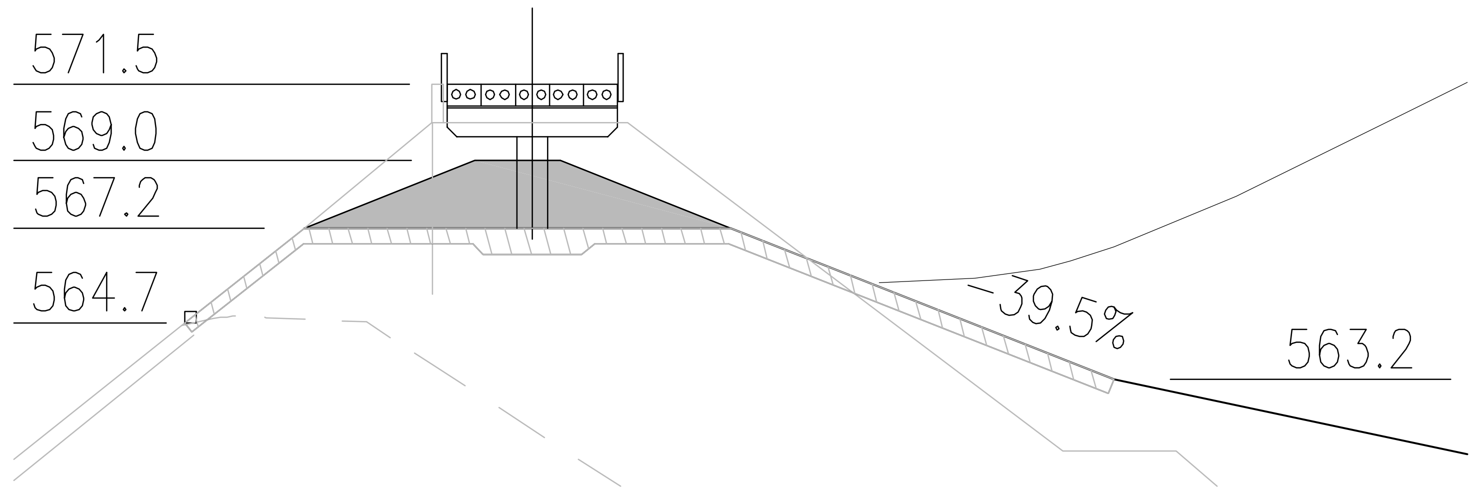
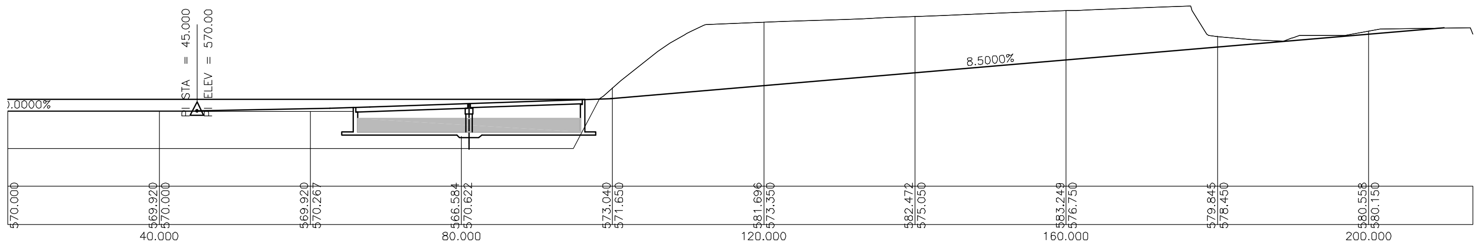


ACCESS BRIDGE – LONG SECTION

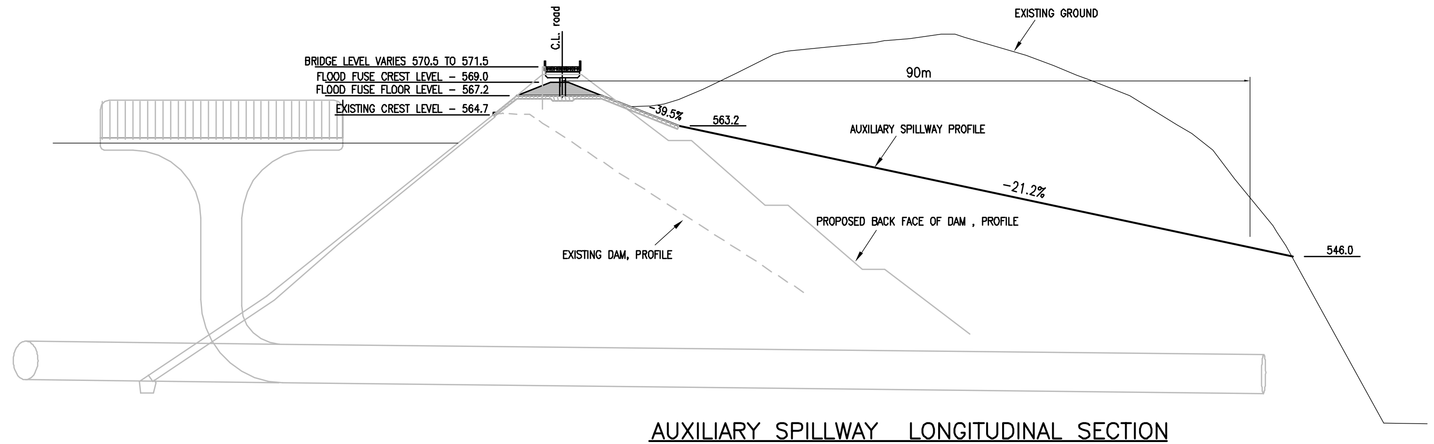
1:100





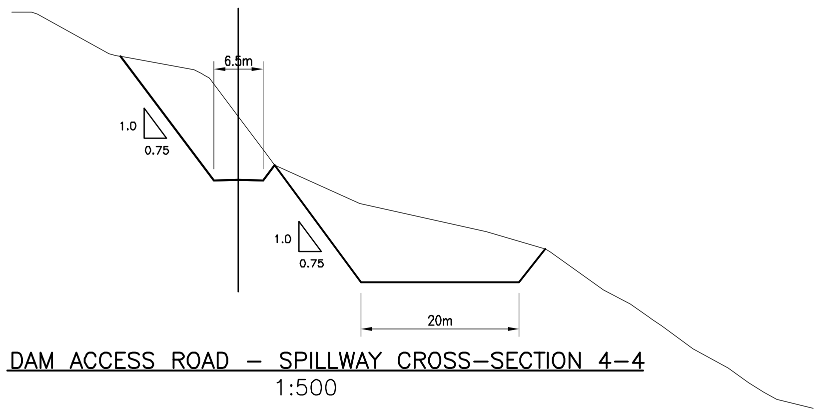
DAM ACCESS ROAD LONGITUDINAL SECTION 1–1

1:500



AUXILIARY SPILLWAY LONGITUDINAL SECTION

1:500



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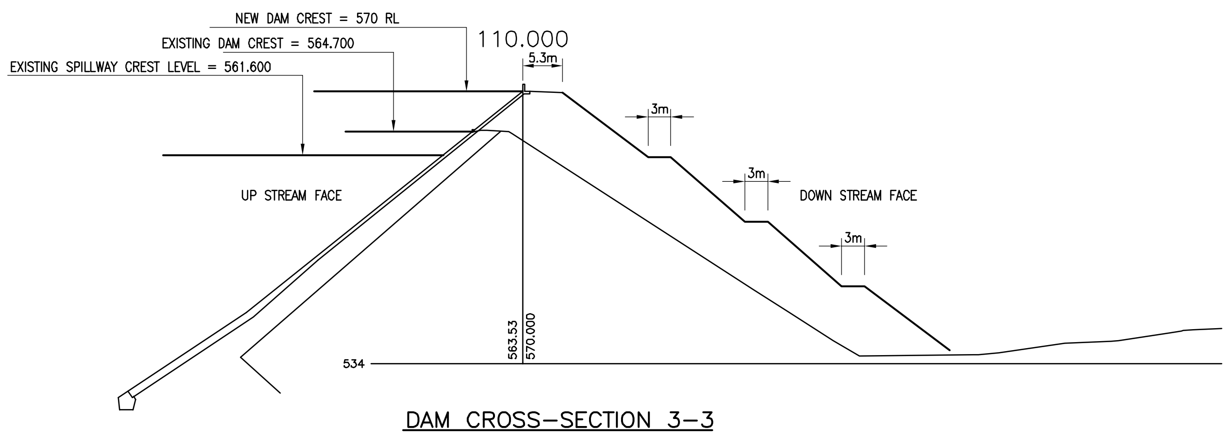

DAM CROSS-SECTION 3–3

1:500

Figure 5

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DRAWN	I Walsh/D Wilson		8/04
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TITLE FALLS DAM COMPANY
FALLS DAM
5m DAM RAISING CONCEPT

CREST, SPILWAY and ACCESS ROAD DETAILS

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SCALE	PLOT DATE	FEATURE IDENTIFIER	CODE	SHEET	REVISION
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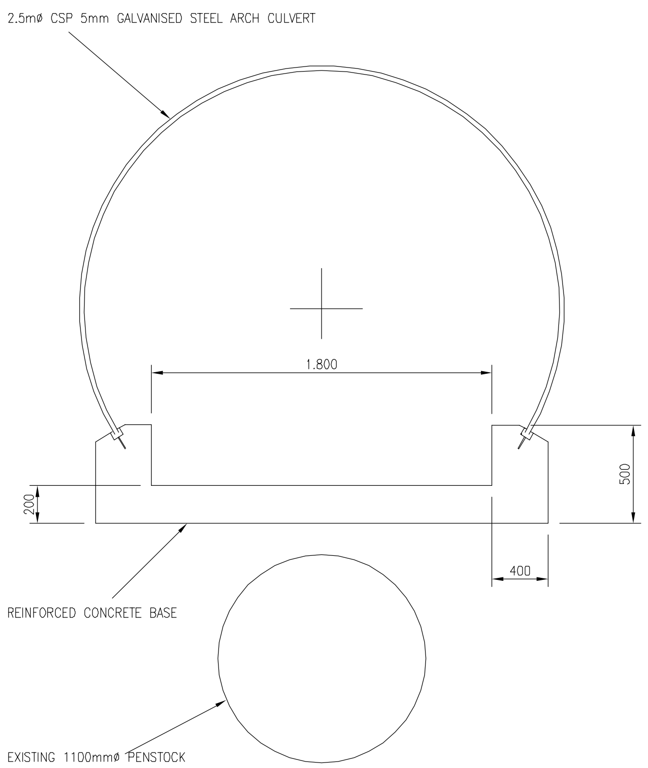

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ACCESS TUNNEL EXTENSION – SECTION
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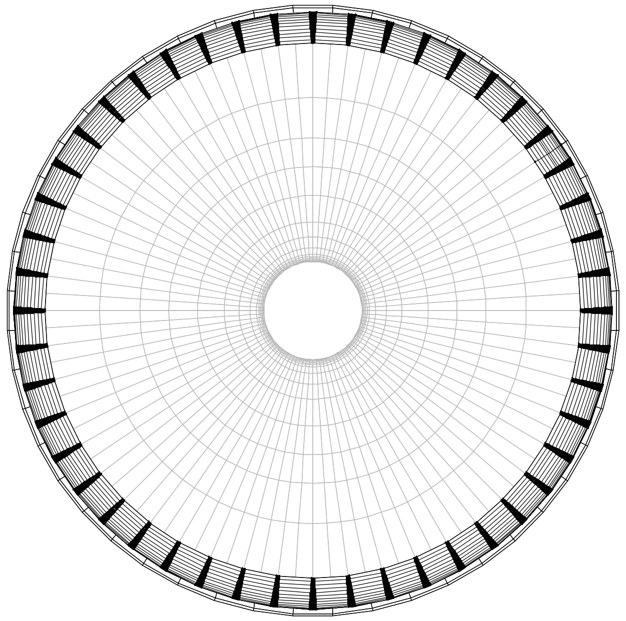
NOTE: GLORY HOLE GUIDE WALLS (6 OF) AND AERATION PIPEWORK NOT SHOWN



GLORY HOLE CREST EXTENSION – PLAN
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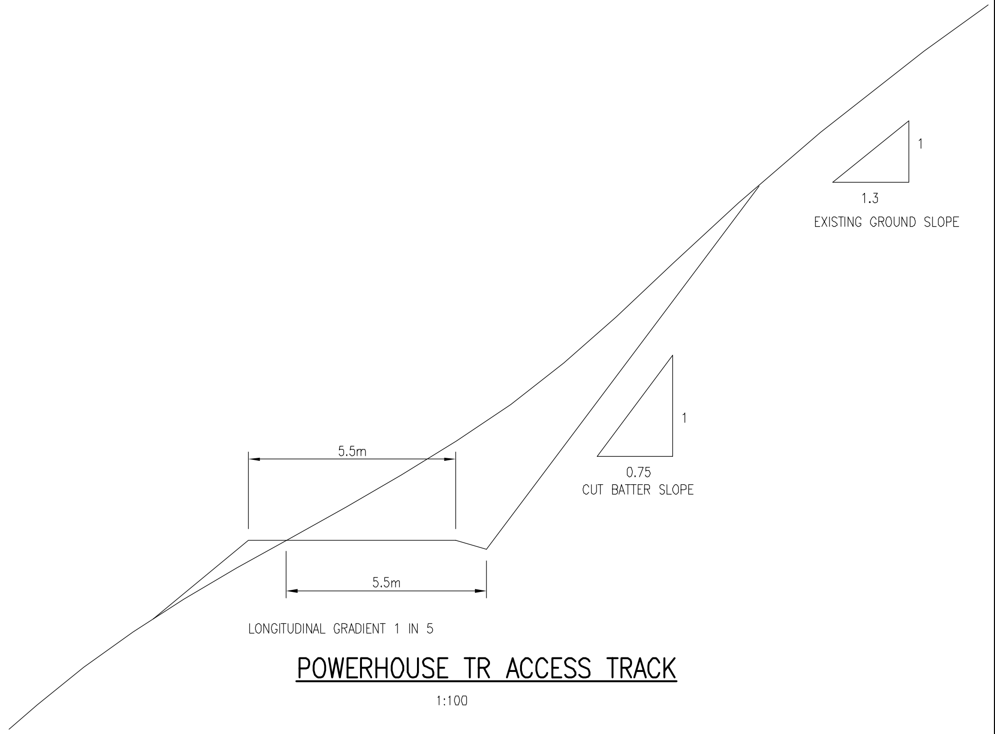

POWERHOUSE TR ACCESS TRACK
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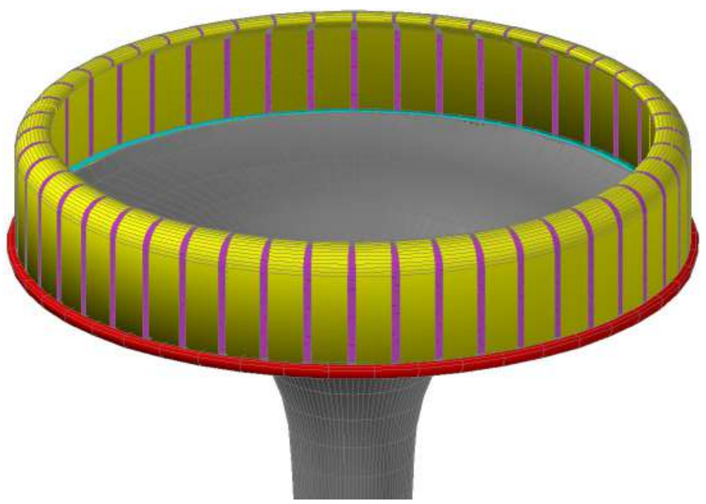
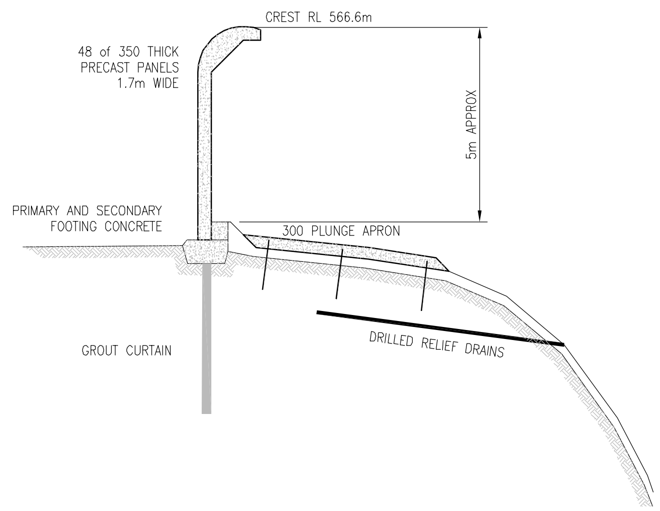


GLORY HOLE CREST EXTENSION
NTS



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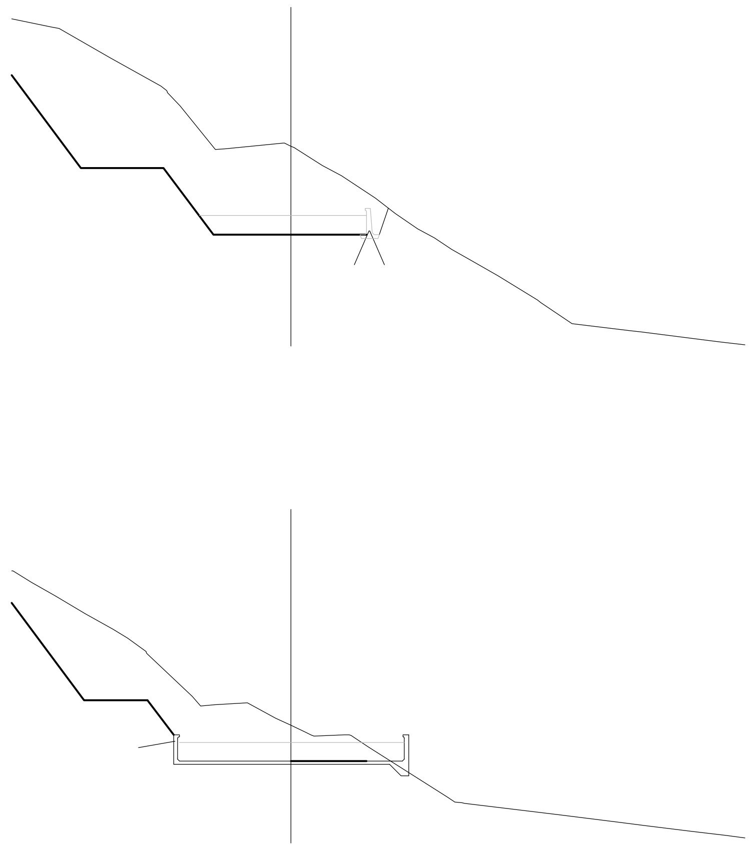

AUXILIARY SPILLWAY – SECTIONS
1:50

Figure 6

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DRAWN	I Walsh/D Wilson		8/04
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TITLE FALLS DAM COMPANY
FALLS DAM
5m DAM RAISING CONCEPT
DETAILS

STATUS	DRAFT	FILE	CWI01.30		
SCALE	PLOT DATE	FEATURE IDENTIFIER	CODE	SHEET	REVISION
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GRAPHIC SCALES

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1.3 Current Engineering Prefeasibility Study Scope and Purpose

This engineering study fits into the wider study programme by examining the engineering concepts and challenges associated with developing enhanced storage at the Falls dam site. No new field investigations have been undertaken in this study, but existing sources of knowledge on the site and the current assets have been utilised when preparing potential development concepts and preliminary construction cost estimating.

The primary purpose of these activities is to assist with the identification of the most suitable (if any) scope and form of development at this site, including the potential for any progressive investment to match land use changes in the valley. This consideration includes assessment of the likely development costs, albeit at a preliminary stage of definition.

The secondary purpose of this study is to give focus to any subsequent engineering studies leading towards detailed design and construction. The identification of issues requiring close attention is a key intended outcome, especially any aspects that have the potential to prevent the successful execution of the development.

1.3.1 Irrigation Storage Considerations

Three storage development scenarios are examined in this report to populate several points on the continuum of possibilities. Namely nominal gross storage volumes of;

- $20Mm^3$,
- $50Mm^3$ and,
- $100Mm^3$.

The findings from a previously examined lesser case involving closer to $15Mm^3$ gross storage has been carried forward into this study to complement the findings.

1.3.2 Flood Discharge Capacity Considerations

Background information on the flood hydrology of this catchment is discussed in a subsequent section, but the briefing for all scenarios considered in this study include for a design peak flood discharge capacity of 700 m^3/s for consistency of approach at this preliminary stage [pers. comm. Peter Brown]. This capacity is understood to correspond to some 50% of the inflow probable maximum flood peak (PMF).

1.3.3 Offtake Capacity and Hydropower Considerations

Peak offtake flow capacity adopted for the study is subject to the respective service areas and storage provisions of each development scenario. The three storage scenarios described in 1.3.1 above require peak discharge capacities of

- $4.0 m^3/s$,
- $6.5\ m^3/s$ and
- $11.0 m^3/s$

respectively [pers. comm. Peter Brown]. This compares to the current system peak capacity of $4.0m^3/s$, with associated attenuation of flow delivery at lower reservoir elevations due to both hydraulic effects and to water use conditions, (refer Figure 17).

It is assumed for the purposes of this study that a suitable hydropower plant will still be incorporated into the offtake works for the scheme, similar in concept to the current facility which operates up to $4.0m^3/s$ at a gross (static) head up to 36m.

1.3.4 Dam Safety Considerations

As impoundment at this site involves a “high dam” in terms of the definitions within the Building Act 2004, and Building Regulations (Dam Safety) 2008, the owners of the facility must have a dam safety assurance programme (DSAP) in place to address the management of potential hazard arising from uncontrolled release from the impoundment. The potential hazard is represented by the potential impact classification (PIC) for the impoundment, where the effects of a dam break event are considered in terms of risk to life, property, and the environment. This classification has nothing to do with the likelihood of experiencing such a failure, as it is based purely upon the assessed consequences.

The assigned PIC rating for a site of either “low”, “medium” or “high” dictates the degree of conservatism required to be applied to the design. The current impoundment has a provisional “medium” rating.

The assessment of ongoing safety is determined relative to current good practice engineering standards, not to the original design standards of the day. Therefore it is quite common for design deficiencies to be identified for such matters as seismic resilience or flood discharge capacity as new knowledge becomes available. Good practice methods to achieve compliance with the current building code are generally taken to be those presented in the NZSOLD Dam Safety Guidelines, last published in c2000.

All engineering structures exhibit some form of deterioration in service, and dams are no exception despite their generally long expected service life. This deterioration needs to be well understood and managed, particularly where it may impact upon levels of safety.

Asset reliability and business insurance considerations also may impact upon the degree of risk that is acceptable.

Given the potential to experience an event exceeding the design condition (albeit at low likelihood), the owner also needs to be prepared to respond to such an event to mitigate the adverse effects. This requires an emergency action plan (EAP) to be in place and able to be implemented in an emergency.

In summary the DSAP must address all of the above influences on dam safety in a comprehensive manner, including an independent audit process to give assurance the programme continues to be effective. This entails regular surveillance monitoring of dam condition and performance, timely identification and remediation of deficiencies, an effective preparedness response and recovery plan, regular reviews of performance and comprehensive independent safety reviews at intervals of 5 to 10 years.

All development scenarios considered in this study will need to meet this test of compliance. The building consent process will be the vehicle whereby this test is applied, once the engineering development process progresses through to detailed design stage.

1.3.5 Exclusions

Notwithstanding the above discussion on dam safety compliance requirements, scoping of RMA and BA consents is not included in the scope of this engineering study.

Similarly, land and property aspects associated with any redevelopment project are not included within the scope of this engineering study.

2 Existing Assets

2.1 Ownership

The Falls dam storage assets are owned by the Omakau Area Irrigation Company Limited and operated by Falls Dam Company Limited. The hydropower assets are owned and operated by Pioneer Generation Limited.

2.2 Hydrological Setting

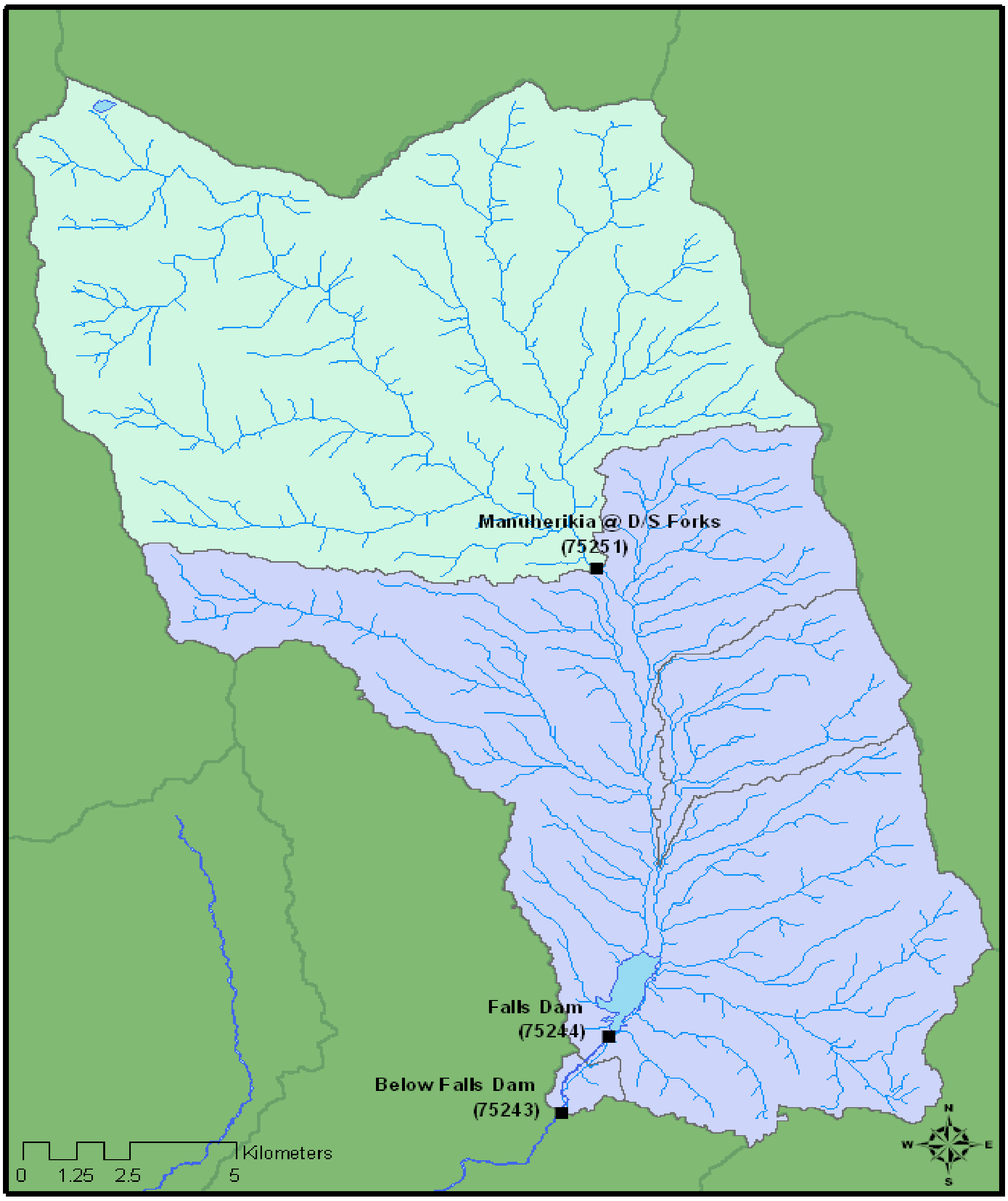


Figure 7 Catchment Plan

The Manuherikia catchment area at the site of Falls Dam is some 372km2 [Opus 2007b].

Water resources available for irrigation development are addressed in detail elsewhere [Aqualinc 2012], but in overview, it has been recognised for many years that the current storage capacity at Falls Dam is substantially less than optimum, even for the present demands.

In terms of design flood flows at the dam site, two studies have been reported [MWD 1984] [OPUS 2007b]. These two studies adopted quite different methodologies, and their findings were also quite different; the latter study producing peak flow rates somewhat lower than the earlier study. Based upon the existing dam and spillway configuration, the c2007 study presented the following tabulation of results.

Annual Exceedence Probability	Source	Peak Inflow	Existing Dam	
			Peak Outflow	Maximum Level
AEP		(m^3/s)	(m^3/s)	(masl)
1 in 500	2007 study	292	291	562.85
	1983 study	430	424	563.18
1 in 1000	2007 study	319	318	562.91
	1983 study	490	432	564.15
1 in 5000	2007 study	390	389	563.07
	1983 study	630	616	565.43

As the existing morning glory spillway is expected to experience choking above 430 m^3/s, the differences between these studies is very significant. Current thinking is that choking concerns will not in fact arise at the 1 in 500 flood flow as previously thought, but may well only occur in flood conditions above the 1 in 5000 event.

Along with these probabilistic flood events, the probable maximum flood, or the PMF, must also be considered. The peak inflow hydrograph for the critical 72 hour duration event, [OPUS 2007b] is assessed to have a peak flow of 1303m^3/s and a volume of 134.6 Mm^3, which is 84% of the assessed 72 hour probable maximum precipitation or PMP rainfall.

The actual flood discharge demand will be dependent upon the effects of routing the flood through the specific reservoir and spillway. While the existing storage is not large enough to significantly attenuate the peak flow rates, substantially increased storage volumes and significant freeboard preceeding the flood event may well become important considerations.

As presented in Section 1 above, for the purposes of this engineering prefeasibility study, the briefing has defined a design spillway discharge capacity of 700m^3/s to be used for all development options. This figure exceeds the 1 in 10,000 flood flow and approximates to 50% of the PMF flood inflow.

2.3 Topographical Setting

The existing dam site is located above the natural falls at the entry to the incised river gorge. Original construction survey records are still available, and more recently (c2005) the Falls Dam Company have commissioned the production of a digital terrain model (DTM) from existing aerial photographs held by NZ Aerial Mapping. Contour information has been generated from earlier stereo pairs SN 8183 flown 22/02/1983 at 1:25000 scale in black and white, and the later photographs SN 12780 flown 27/02/2003 at 1:50000 scale in colour have been ortho-corrected to fit the model. This digital model appears to provide good resolution for preliminary design purposes, but it does not have adequate control established for construction purposes. There is some uncertainty in the datum conversion in the order of +/- 0.15m, but this degree of error is not of concern for current purposes.

The original construction survey has been relied upon for ground definition below the normal reservoir water surface, and no specific sedimentation surface data has been obtained, nor river bed erosion soundings in the gorge section downstream.

The digital terrain model does not extend to full coverage of the potential highest elevations for new dam storage impoundment up to the Shamrock Gully saddle, so historical survey records and 1:50,000 topo maps have been relied upon for this extended coverage.

2.4 Geological and Seismological Setting

The dam site is located within Greywacke terrain, just north of the contact with the more typical Schist terrain of Central Otago. The rock exposed at the dam site is very hard, fresh to moderately weathered Argillite, typically exhibiting steeply dipping shear zones at 3m to 5m centres with associated advanced weathering extending to a depth of 200mm from the rock mass defects. Despite the influence of the tectonic processes that have occurred at the site, the rock mass exhibits very good engineering properties, as is evidenced by the original falls in the river bed, along with the very steep sided river gorge downstream. The local rock quarried to form the embankment has not shown any susceptibility to freeze-thaw breakdown in service.

Logs from the original 5.2m diameter diversion tunnel construction are still available, and more recent experience from the hydropower scheme construction is consistent with this good engineering performance.

Stability of cut slopes is dictated by the orientation, spacing, persistence and weathering on the rock mass defects. An indication of typical steep cut slope behaviour can be observed at the left abutment spillway bench cut, where the 0.65H:1V face has performed well.

The seismicity of the site has not been specifically studied, but it is to be expected that the Dunstan Fault system may be an important consideration in the determination of the maximum design earthquake (MDE) for the site. Although this system is not flagged as “active” in terms of NZS1170.5 methodology requiring consideration of near fault effects for buildings, it may well be important in the dam context.

Regional seismicity is quite low, as indicated by the Z factor of 0.22 in NZS1170.5, corresponding approximately to a service level operating base earthquake (OBE) ground acceleration of 0.136g, and a maximum design earthquake (MDE) ultimate state ground acceleration in the order of 0.40g to 0.62g, as illustrated on the figure below adapted from NZS1170.5. While NZS1170.5 is not directly applicable to the design of dams, it is relevant to the general understanding of regional seismicity, and for an indication of the seismic ground actions that might be experienced at the site.

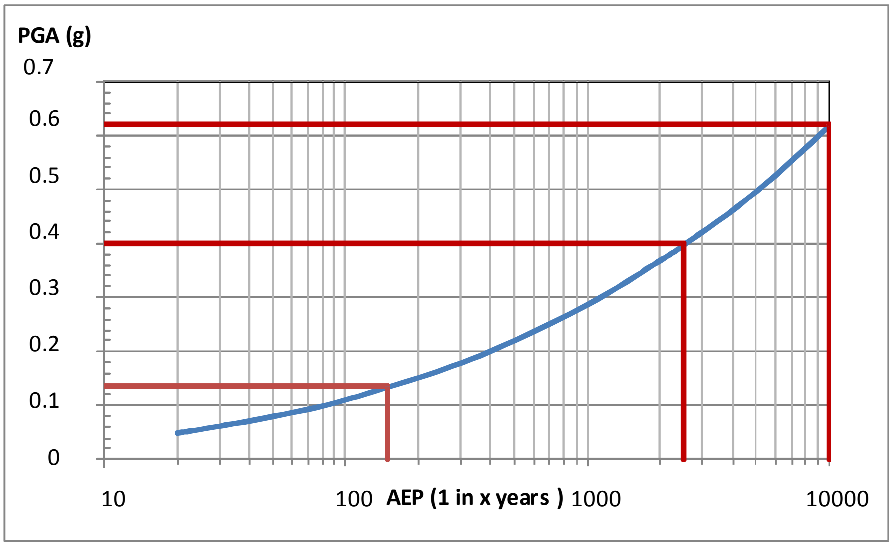


Figure 8 Site Probabilistic Seismicity

Given the hard rock nature of the site, there is no concern regarding liquefaction risks or the like, but stability around the reservoir margin would need to be investigated, along with the nature of the foundations at any saddle dam. Actual seismic analysis of dams will be subject to dynamic loading and response considerations, where both amplification of the peak ground acceleration and attenuation of forces from post elastic response may apply.

2.5 Development History

The original c1930's construction is well described elsewhere [Gilkison 1937], [Offer 1997] and [Ellis 2009].

The original works have served their purpose well, with little change being required over the years. The entry lip at the morning glory spillway was raised by 2 ft (0.61m) c1955 [MoW 1974] to improve storage capacity.

In 2003 a 1.2MW mini hydropower scheme was retrofitted to the dam, affecting the configuration of the offtake works that were originally housed underground at the end of the access adit connecting the portal at the dam toe to the diversion tunnel chamber.

Some features of the key elements are described below.

2.6 Diversion and Morning Glory Spillway

A 5.2m diameter concrete lined diversion tunnel was constructed through the left abutment rock to facilitate dewatering of the dam footprint and allow construction.

Figure 9 Left abutment diversion tunnel

This diversion is still in service, albeit with a concrete bulkhead isolating the reservoir from the valve chamber housing the original offtake works. A cast iron pipeline passes through the bulkhead, some 33inch (838mm) diameter.

Figure 10 Morning Glory Spillway Construction

This photograph of the spillway when constructed shows the 30m diameter of the concrete lining and the guide vanes that control vortex formation and facilitate aeration of the flow. The full supply level (FSL) was raised 2' (610mm) c1955 from 1840' to 1842' by constructing a low weir at the circumference of the concrete lining. The 17' (5.2m) diameter shaft connects to the concrete lined diversion tunnel below.

In terms of flood discharge capacity, the existing morning glory spillway presents some challenges due to the manner in which it chokes beyond maximum design flow. In the event of a flood exceeding 430m^3/s, the water continues to head up without significant increase in discharge.

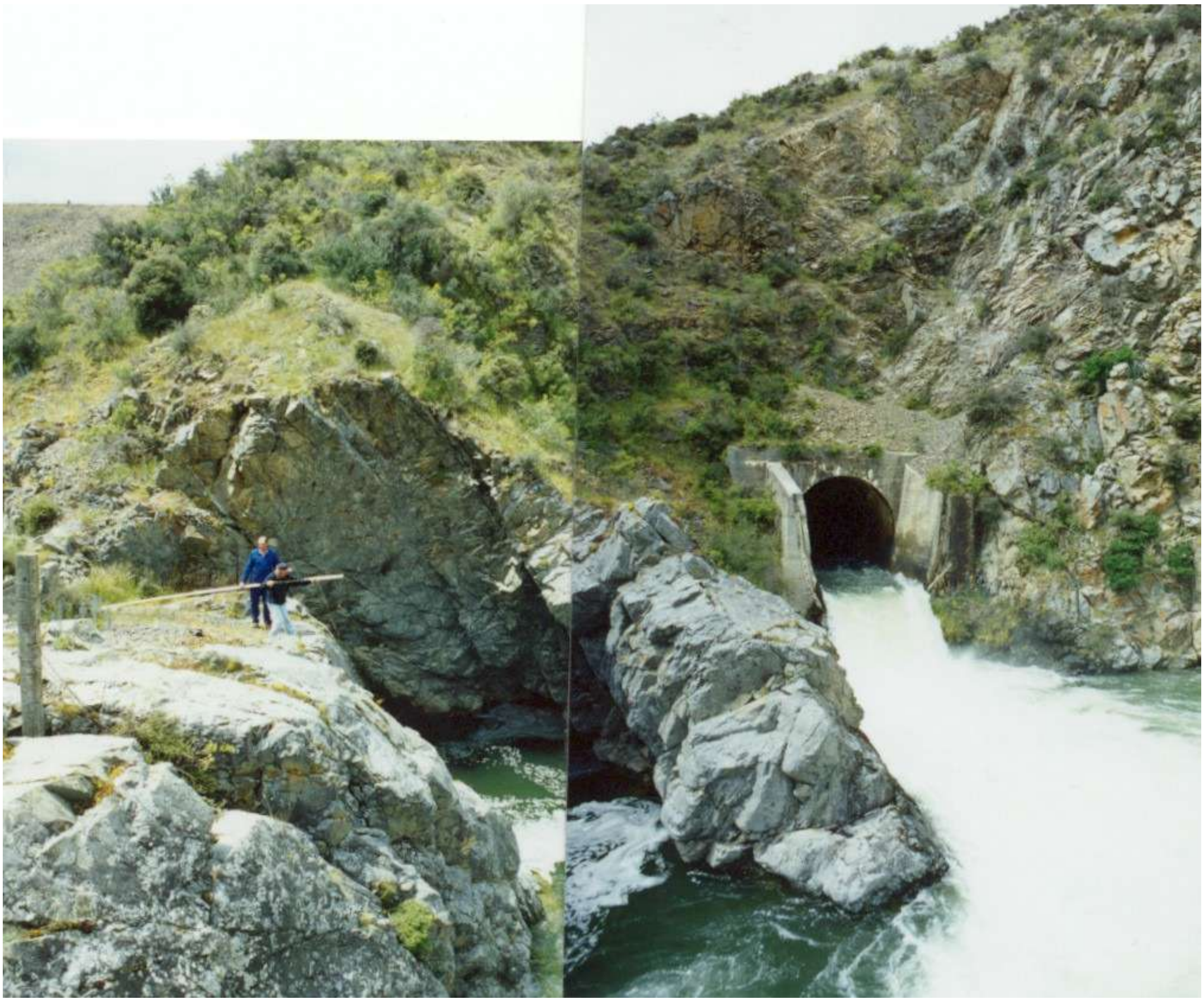


Figure 11 Tunnel outlet c2002 prior to hydropower scheme development

The tunnel discharges above tail water level (TWL) just below the site of the original falls. The photograph above shows the hard rock nature of the original river bed, with the eroded cavity behind the face associated with release on the steeply inclined defects in the rock mass which are typical of this site. This area was utilised to construct the power house and gain effective generation head from the lowest available TWL.

2.7 Concrete Faced Rockfill Dam

The site did not present suitable core material for a conventional zoned earthfill embankment dam, but suitable rock for rock fill was readily available immediately at the site.

Figure 12 Rock quarry above true right abutment

The photograph above shows the true right abutment quarry; a left abutment rock source was also used to form the spillway platform. The rockfill was dumped rather than compacted as would be the case today, and as a result there will be a greater void content in the fill. Careful attention including much hand labour was given to the upstream fill zone to provide support to the intended facing, and the lowest level included mortared rockfill which would be very stiff in comparison to the dumped rockfill.

Water tightness was provided by a concrete facing placed on the steep 1.25:1 upstream slope in 44’ (13.4m) panels with copper water stops between. The facing is sealed to the rock foundations with a flexible copper water stopped perimetric joint connection to a concrete infilled cut off trench and plinth upstand.

Figure 13 Cut off trench and plinth well advanced

Figure 14 Concrete facing nearing completion

2.8 Outlet Works and Hydropower Scheme

The original outlet works comprised 838dia cast iron pipework passing through the diversion bulkhead into the valve chamber, and terminating in a Glenfield Kennedy cone dispersion sleeve valve discharging into the tunnel. This arrangement dissipated the surplus water head. The terminal discharge valve had performance problems over the years due to the effects of cavitation on the central cone, and loading on the screw drive actuation shaft.

The original offtake works were substantially modified c2003 with the construction of the mini hydropower scheme.

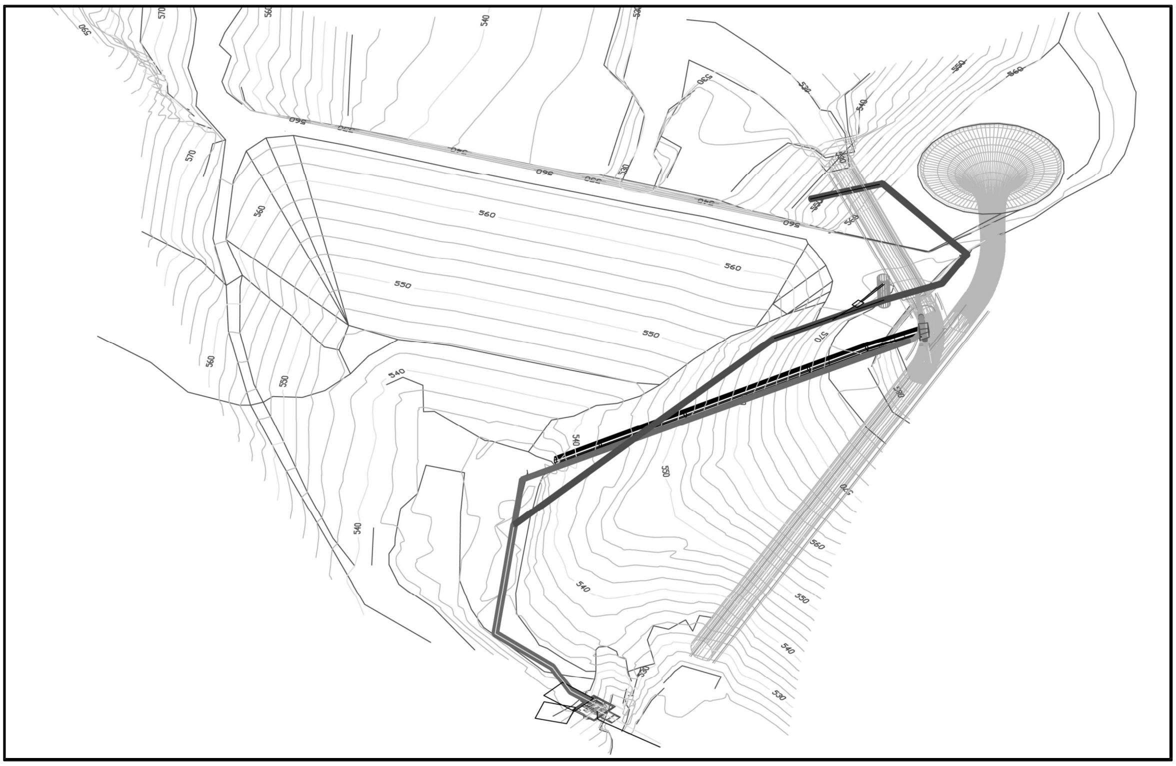


Figure 15 Mini hydropower scheme - 3D supply works layout

The original offtake pipework in the valve chamber was coupled into a new low level 1100NB steel penstock pipe laid in a trench excavated in the floor of the access adit. Because of the head losses associated with this restrictive supply system, and to conserve the available head for power generation purposes, a new high level 1200NB steel penstock was laid in a trench over the crest of the dam. These two supply pipelines converge into a single 1400NB steel penstock pipe connected to the powerhouse containing a single vertical axis Kaplan turbine.

Figure 16 Adit Trench

A key feature of this arrangement is the adoption of the syphonic operation of the high level penstock [Walsh 2010].

This pipeline operates under vacuum conditions as low as 80kPa below atmospheric pressure to draw water from the reservoir when the level falls below the pipe trenched around the dam crest in the true left abutment. A vacuum pump maintains prime on this system. The discharge capacity of this system is designed to suit the flow rate curve presented below. The original low level pipework through the tunnel bulkhead provides adequate capacity at the lower HWL and limited supply flow, with the newer syphonic penstock complementing the original capacity at higher HWL's.

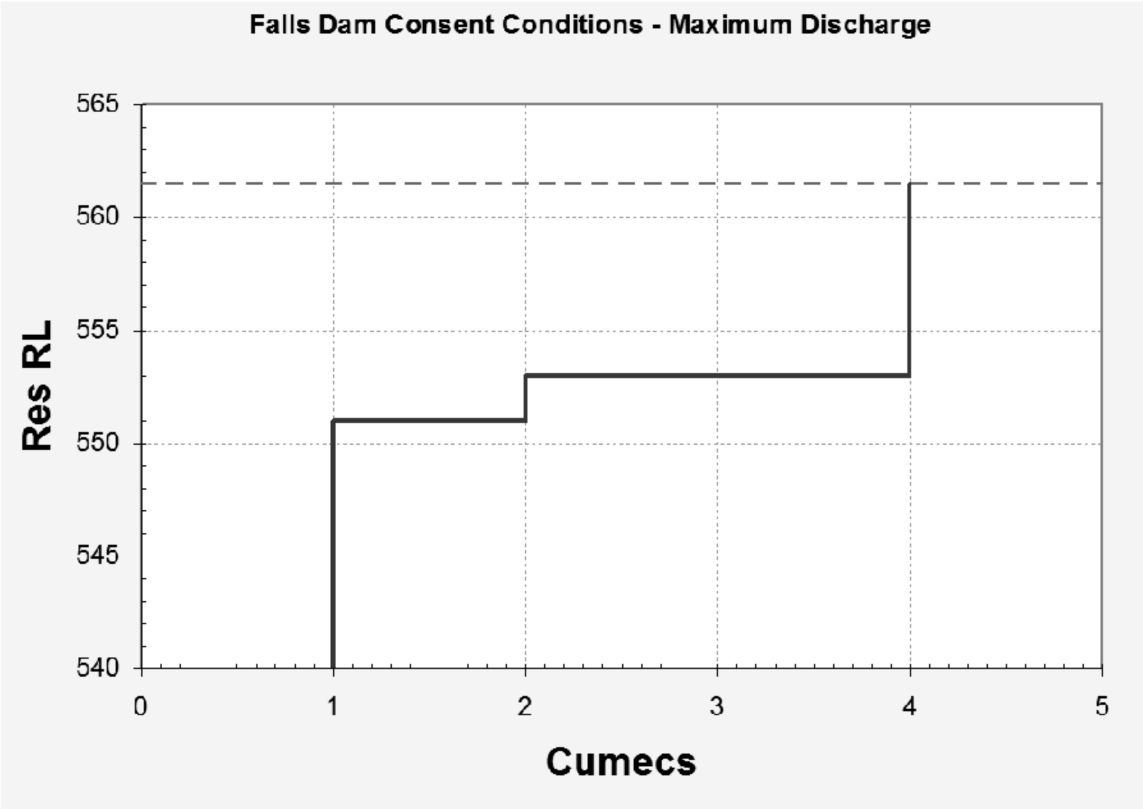


Figure 17 Offtake discharge with respect to HWL

The high level penstock bypasses the concrete dam facing membrane and plinth via a trench excavated in the true left abutment. Dam integrity is maintained by concrete pipe encasement within a cut off wall placed in the excavation prior to backfilling as shown below.

Figure 18 Extension to plinth cut off trench at left abutment crest

Figure 19 Syphonic penstock penetration through extended cut off

The powerhouse is a tanked structure arranged to utilise the low TWL below the falls.

Figure 20 Tanked powerhouse construction at the "falls"

The small Kaplan turbine has peak efficiency around 3.5m^3/s, and is able to accommodate net generation heads up to around 37m as indicated in the figure below. The influence of variable moderate head increase on generation output from this machine has been previously assessed [OPUS 2007a], including consideration of clipping the peak head at a range of levels.

Characteristics of the existing hydropower scheme are shown on the following figures.

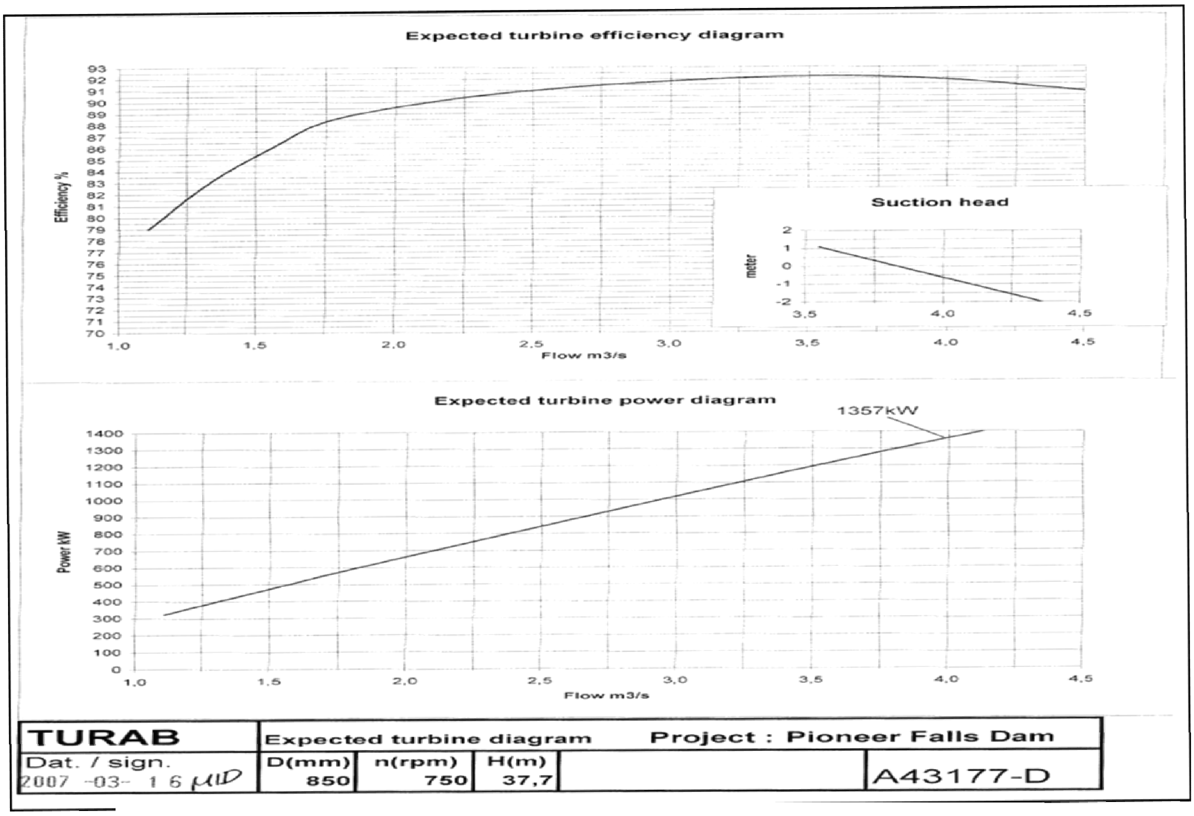
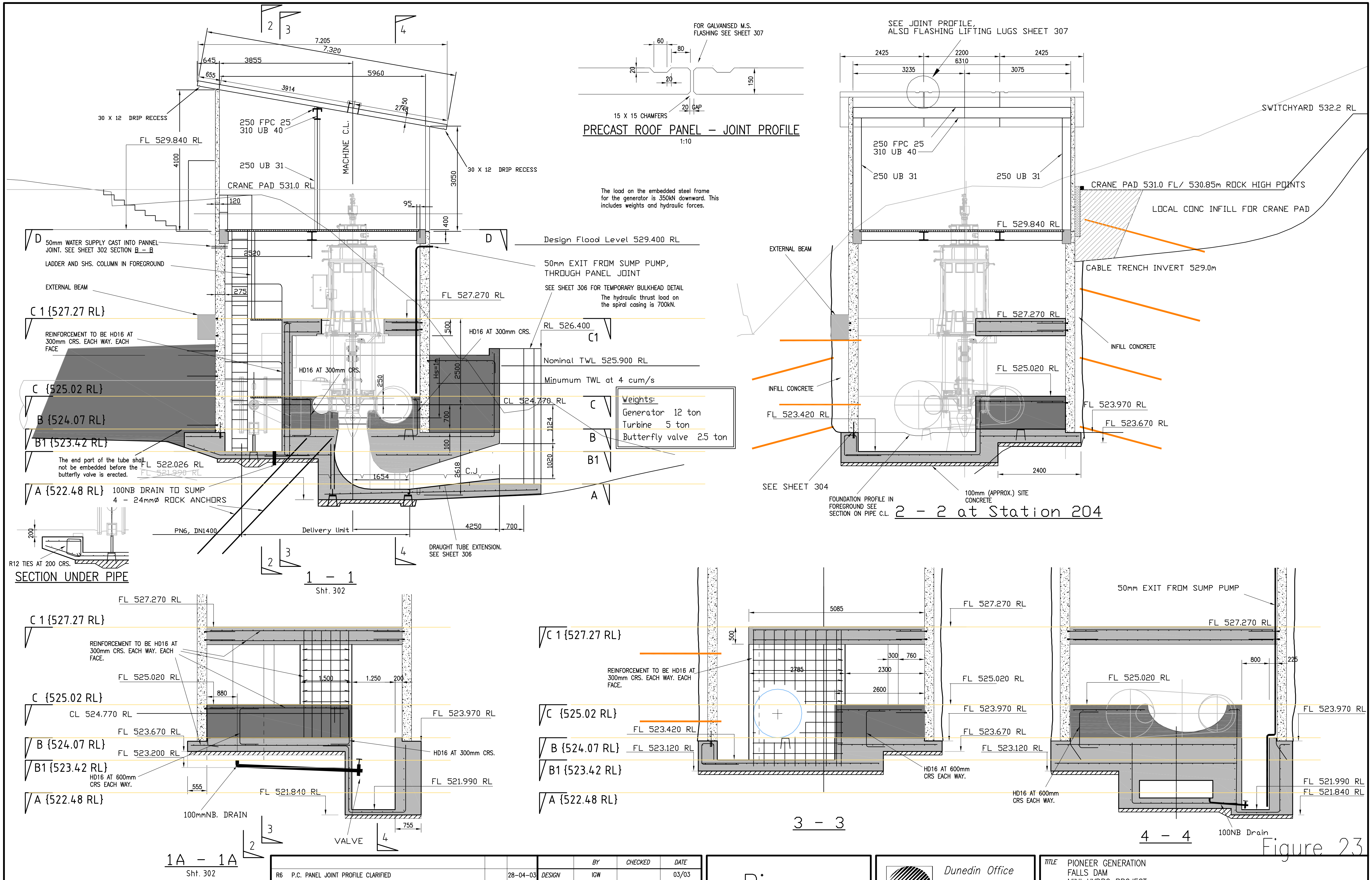


Figure 21 Kaplan Turbine Efficiency Curve (@37.7m net head)

Figure 22 Aerial view of hydropower scheme construction



NOTE:
ALL CAST IN PIPE TO BE 'CLASS C' PLASTIC

	AMENDMENT	APP'D	DATE
R6	P.C. PANEL JOINT PROFILE CLARIFIED		28-04-03
R5	UPDATED TO CONSTRUCTION STATUS		15/4/03
R4	CONC CLADDING, CONC BACKFILL	IGW	02/03
R3	ADD CROSS SECTION, RAISE CONTROL FLOOR	IGW	01/03
R2	STEEL FRAME SUPERSTRUCTURE	IGW	01/03
R1	BASED ON TURAB A31421 OF 7 Nov. 02		16-12-02

	BY	CHECKED	DATE
DESIGN	IGW		03/03
DRAWN	IGW/DW		03/03
APPROVED			

Pioneer Generation

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TITLE PIONEER GENERATION
FALLS DAM
MINI HYDRO PROJECT
POWER HOUSE
SECTION

STATUS	CONSTRUCTION	FILE	6CWR00.48		
SCALE 1:50	PLOT DATE 15/05/03 @ 13:53	FEATURE IDENTIFIER 7/461/37	CODE 7704	SHEET 301	REVISION R6

GRAPHIC SCALES

g:\projects\6cwr01.48 falls mini hydro\dwgs\construction\dips\7_461_37_7704_301_303 and 304.dwg ORIGINAL SHEET SIZE A1 [840x594]

2.9 RMA Consent Considerations

The existing Falls Dam is authorised by a mining privilege, with further consents more recently granted for the addition of power generation commissioned in 2003.

The impoundment works envisaged within the various development scenarios covered by this study would require new RMA consents and specific Building Consents. Addressing the specific aspects of such consents is outside the scope of this engineering prefeasibility study commission, but suffice it to note that compliance with all the usual dam safety obligations has been automatically assumed. Developments of this nature and scope could be expected to be subject to independent expert peer review and thorough hazard management assessment processes, in addition to the environmental protection requirements and normal engineering and construction good practice.

The NZSOLD Dam Safety Guidelines [NZSOLD 2000] and similar reference material from recognised international bodies provides a design and construction good practice framework that is generally accepted within the industry and by regulatory bodies.

2.10 Current Asset Condition and Dam Safety Deficiency Management Considerations

2.10.1 Design Deficiencies

The original dam design did not include consideration of seismic loading commensurate with current knowledge on the seismicity in this area. While the concrete faced rockfill embankment dam form is intrinsically stable under imposed water loading, even with seismic action being present, there are some potential failure mechanisms that may occur as a result of severe earthquake loading.

The steep fill batters of dumped rock do not have the same degree of shear strength as would be obtained from well compacted fill with a high degree of particle interlock, so the possibility of instability in the face around the crest cannot be discounted. The upstream face will not experience this mechanism where significant water pressure is present to stabilise the face, but the section above water level and the downstream batter may experience displacement. Such displacement is likely to be shallow given the non-cohesive fill material properties, but the crest is narrow in modern design terms with little margin for reduction before secondary effects become significant.

The dumped nature of the rockfill also makes it more susceptible to densification under seismic loading, as the particles deep within the fill experience more stress than they have previously been subjected to, and they move into a new arrangement. This results in settlement of the embankment that would be avoided or at least minimised in modern well compacted construction. Minor settlement of this nature has been experienced over the life of the dam as revealed in the deformation survey findings and as shown in the figure below.

Figure 24 Dam Crest Deformation

The observed deformed shape is typical of performance of dams of this type which traditionally exhibit long term crest settlement in the order of 0.5% to 1% of dam height (160mm to 330mm in this case), on a vector perpendicular to the membrane facing.

Spillway capacity has been shown in the latest flood study [OPUS 2007b] to be adequate to discharge major floods up to at least the 1 in 5000 AEP event with acceptable freeboard. However, choking and overtopping would be expected under 50% PMF events or greater as would commonly be adopted under current design standards.

The overall impression of these design deficiencies is that while the facility does not satisfy current best practice standards, its original design Is expected to perform satisfactorily in all but the most extreme loading conditions.

2.10.2 Deferred Maintenance

The serviceability report for Falls Dam (Works Consultancy Services, 1989) listed the following as necessary repairs:

a. Facing panels and joint deterioration
b. Control valve and associated equipment (done)
c. Leakage measuring weir (done)
d. Spillway crest concrete
e. Access tracks. (done)
f. Repair of tunnel lining concrete.

Items b), c), and e) have been addressed during the construction of the mini hydropower scheme, but the remaining items are still to be resolved.

Figure 25 (a) & (b) Joint repair condition

In the 1980's a significant leak developed that was traced to a membrane joint failure. A temporary repair comprising butynol sheeting secured with steel angles, and bitumen was undertaken, with a view to permanent repairs being designed and constructed shortly thereafter. These permanent repairs were never undertaken, and the current situation is susceptible to failure at any time.

The exposed concrete surface of the membrane also exhibits local freeze thaw damage leading to loss of protective cover over the reinforcement.

Spillway lining concrete was observed to be deteriorating in the 1980's, and some local repairs to the most severely affected construction joints were undertaken at that time. However, more widespread deterioration was present, and further degradation has subsequently occurred.

Typical erosion of concrete and exposure of reinforcement is illustrated in the adjacent photograph. Erosion of the tunnel lining concrete is also in evidence and repair is needed to ensure ongoing reliable performance of the spillway.

Figure 26 Spillway joints

3 Potential Redevelopment Concepts

3.1 Dam Location(s)

In order to establish confidence in the selection of the site to be studied for investment in upper valley storage, the effect of locating a new dam some distance downstream has been examined as a possible alternative to redevelopment at the present site.

The site beyond the gorge would flood the flats and create a significant impoundment that could complement the existing Falls dam storage.

The stage-storage characteristics for such a new downstream dam site situated where the river bed level is around RL485m is presented to the right.

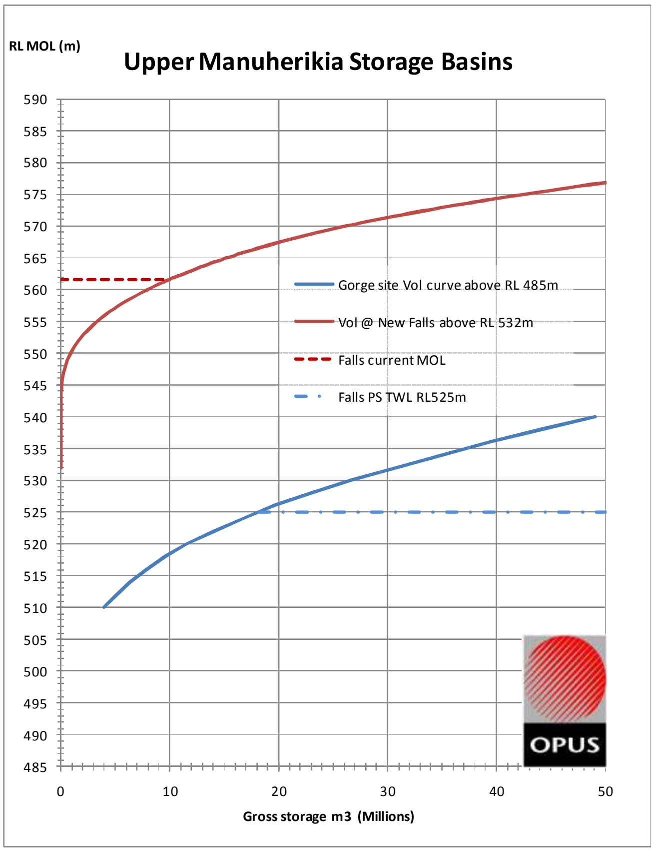

Figure 27 Alternate dam site storage

The lower curve represents the new site, with the existing impoundment curve included above. Allowing the HWL of this lower reservoir to approach RL525m would not compromise the existing hydropower scheme TWL. At this level the lower site would achieve a gross storage of some 18Mm3 which would be additional to the present storage of some 10Mm^3

The dam height to create this impoundment would be in the order of 45m, compared to the existing dam at 33.5m.

While this cascade development concept has its attractions in terms of a green fields project free of the limitations of ageing assets and the need to abandon investment in facilities that will become undersized, it does not in itself achieve the range of storage volumes being targeted in the study, nor does it appear to offer advantages over redevelopment at the current dam site.

To examine this aspect of storage efficiency and potential further, the above graph of storage capacity has been normalised to a common dam base elevation, such that the two sites can be directly compared.

The figure on the right reveals the significantly greater storage efficiency of the current basin, and the prima face' basis to focus on redevelopment along the general lines envisaged by the original developers.

A further major attraction of utilising the current site is the potential to avoid construction diversion costs. The present dam and spillway works can effectively be used for this purpose provided that any new works do not obstruct their functionality and subject to their ongoing serviceability.

The need to maintain irrigation supply during any new construction period is also a key consideration in any redevelopment concept.

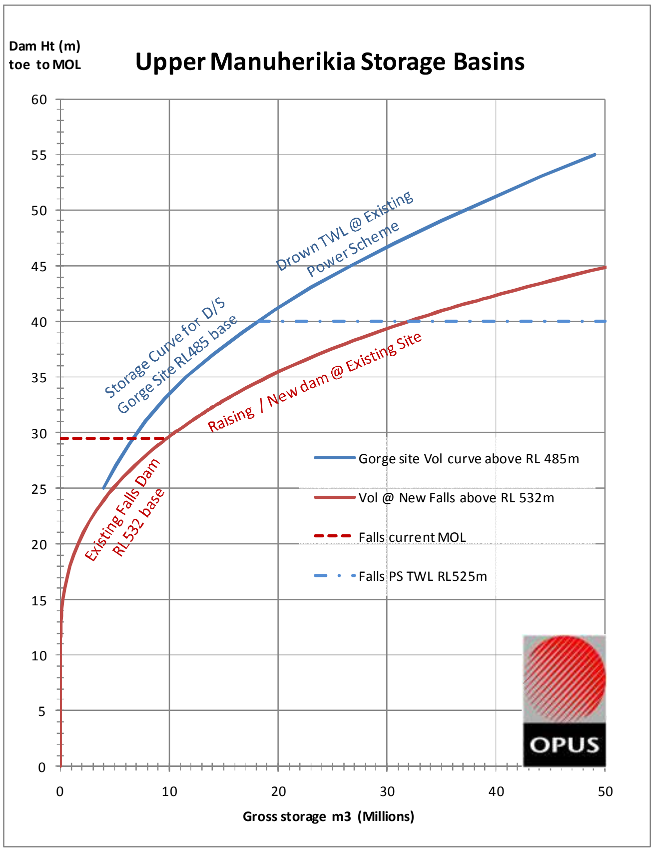

Figure 28 Normalised storage curves

3.2 Reservoir Full Supply Levels

The storage curve for the current dam site has been checked off the latest terrain model as presented in the figure overleaf extended to high elevations requiring a saddle dam at the head of Shamrock Gully. The reason for the departure (favourable) from historical topo graphical survey values is not clear at this time. For the target storage volumes previously adopted for this study, the following full supply levels (FSL) and their approximate relationship to the existing reservoir have been obtained;

- RL588m (+26m) ,
- RL577m (+15m) and,
- RL567.5m (+6m)

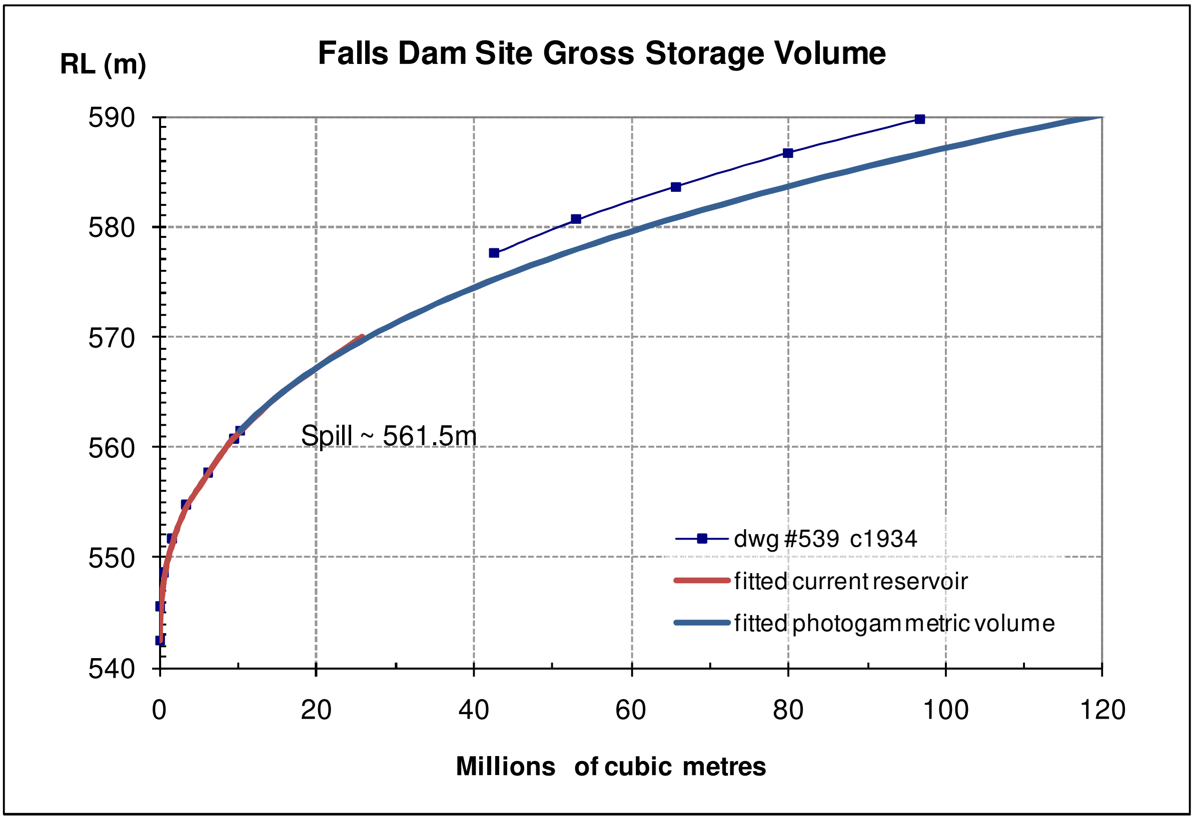

Figure 29 Potential Gross Storage Capacity

The RL 588 highest development scenario does require some saddle dam works to retain the impoundment, as the FSL is close to the ground level at the saddle indicated from the early survey (current aerial mapping photogrammetry and the DTM does not extend to this location). Retention height will need to allow for flood rise and wave action at this saddle location, and of course also at the main dam structure.

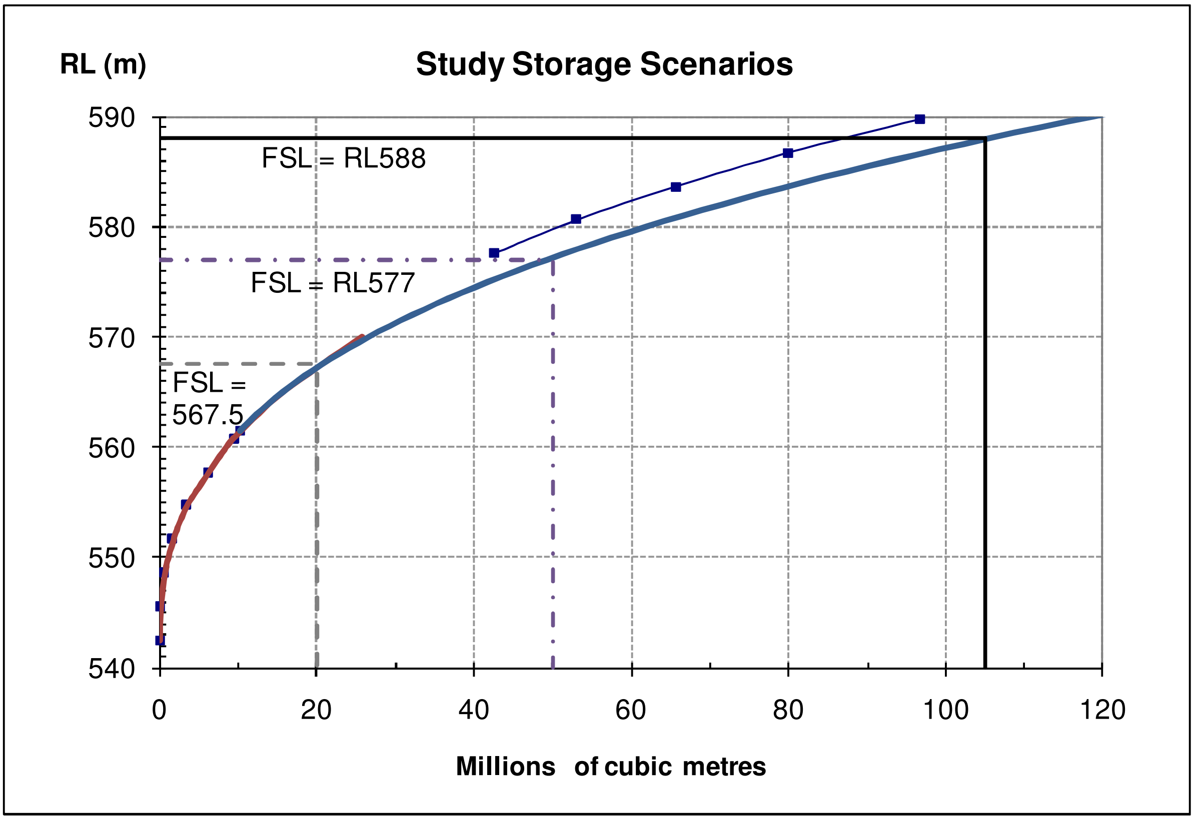

Figure 30 Storage Development Scenarios

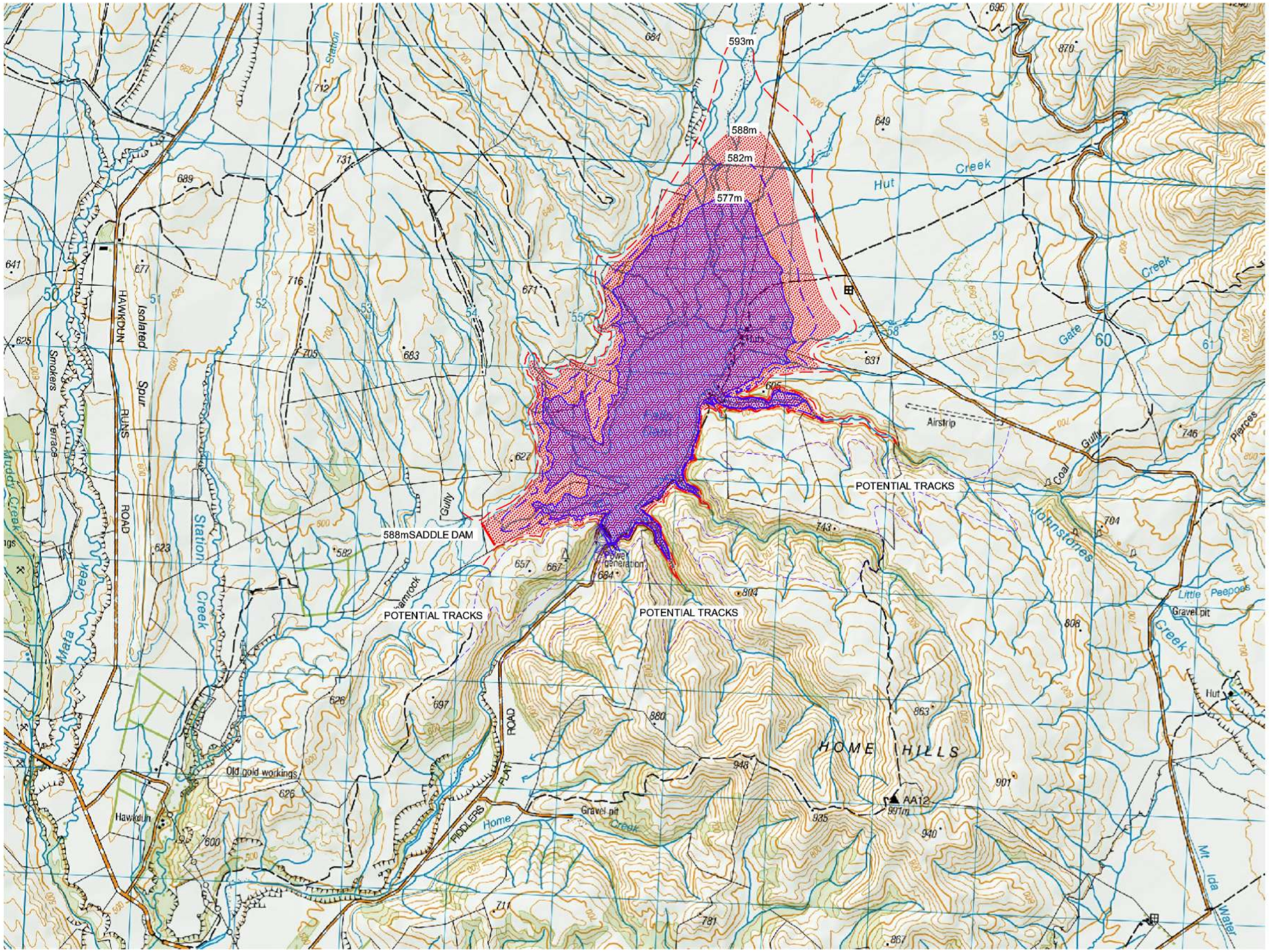

Figure 31

PLAN

1:2000 at A1 size

PRE-FEASIBILITY

Revision	Amendment	Approved	Revision Date

OPUS

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Project
Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
577m and 588m - OPTIONS - RESERVOIR AREAS

Drawn	Designed	Approved	Revision Date
D Wilson	I G Walsh		30-8-2012

Project No.	Scale	Drawing No.	Sheet No.	Revision
6-cwl04.13 / 25dd	As shown	7 / 1071 / 1 / 1704	2	

1:1 @ A1
1:2 @ A3 0 10 20 30 40 50 60 70 80 90 100 mm

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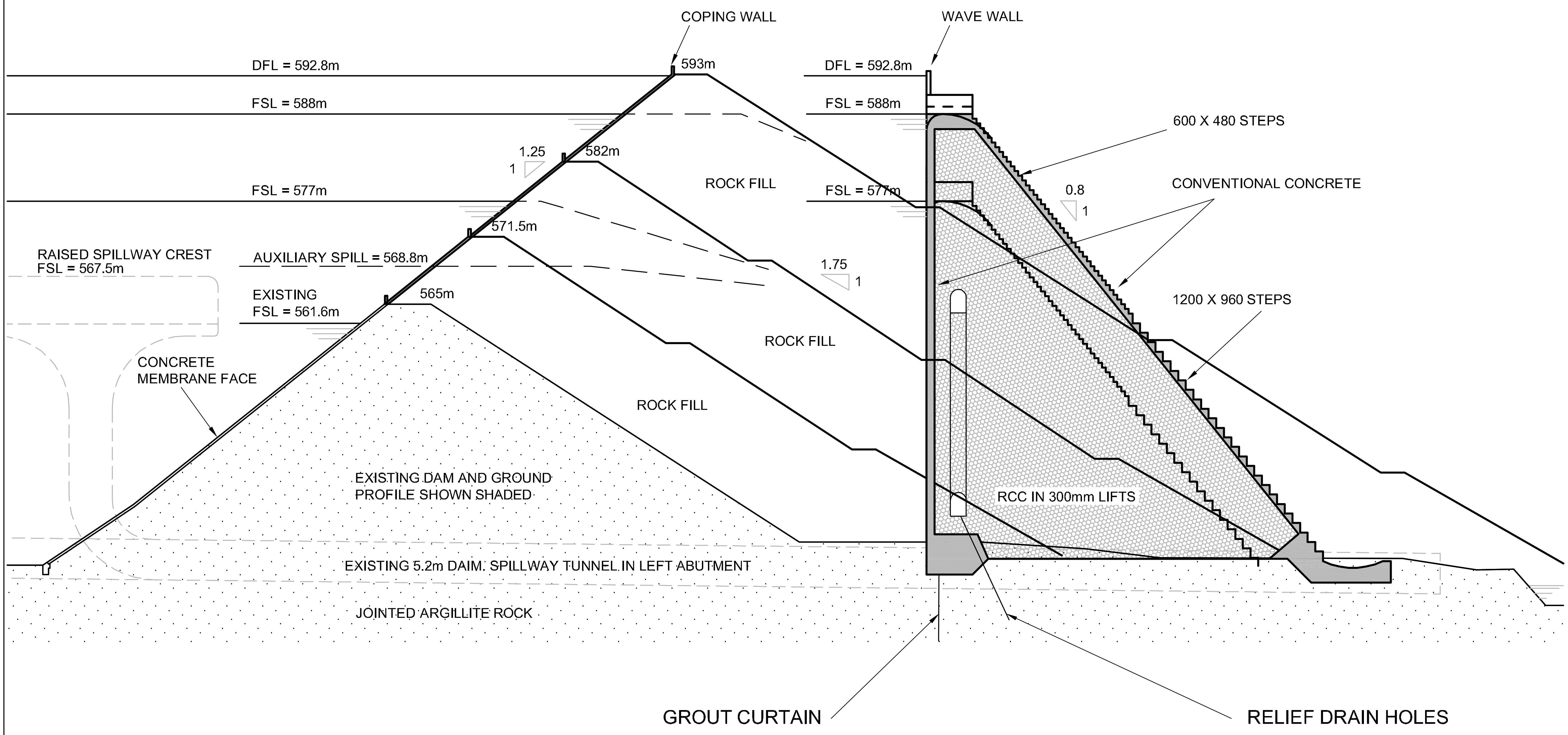

DAM SECTIONS

Figure 32

PRE-FEASIBILITY

Revision	Amendment	Approved	Revision Date

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Drawn	Designed	Approved	Revision Date
D Wilson	I G Walsh		

Project No.	Scale
6-cwi04.13 / 25dd	1:500 at A3

Project: Manuherikia River Catchment, Water Strategy Study - Stage 3, FALLS DAM REDEVELOPMENT

Sheet: ENGINEERING PREFEASIBILITY STUDY, DAM OPTIONS - SCHEMATIC

Drawing No.	Sheet No.	Revision
7 / 1071 / 1 / 1704	1	

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3.3 Dam Structural Forms

As determined in the 1930's, the site does not lend itself to the development of a zoned earthfill embankment dam which commonly achieves the most economic form of construction. However, it is well suited to the concrete faced rockfill embankment (CFRD) form that was previously developed, due to the hard rock nature of the foundation and the plentiful supply of rockfill aggregate. This form of structure generates very high hydraulic gradients around the plinth / cut off at the upstream heel, and a high degree of resistance to foundation erosion is required for successful operation. Other options that could be adapted to this site include concrete arch structures with their reliance on strong and rigid abutment rock characteristics and structural concrete, and concrete gravity structures that are less dependent the ability to carry such concentrated forces in the dam blocks and into the foundation.

The selection of suitable dam form is influenced by more than just the dam itself, as spillway and offtake works must also be taken into account. Embankment dams are generally not suited to safely accommodating flood spillways, so spillway layout must be able to be accommodated away from the dam footprint. In this case the existing morning glory spillway serves this function, but it is only suitable for moderate reservoir raising, which does not extend beyond the lowest RL567.5 FSL option being considered in the study. A separate facility will be required to be developed for any higher embankment dam option, and this would most reasonably be accommodated by a suitably proportioned rock cutting within the left abutment. This cutting could also generate the rockfill material required for both the embankment fill and for concrete aggregate for use in the facing membrane. The concept envisaged in the c1974 study [MoW 1974] of adopting spillway down Shamrock Gully from a high level saddle dam spillway has not been pursued for the highest RL588 FSL option in this study, as the environmental effects of such a major change are thought to present real consenting challenges.

Concrete gravity dams on the other hand require less total construction material and are readily adaptable to accommodate the spillway within the dam footprint. Roller compacted concrete (RCC) construction is expected to deliver the most economic form of gravity construction, as it relies upon the use of plant to place the zero slump low strength mix in a manner similar to that used in road basecourse construction.

For the purposes of this study we have not included arch dam options with their dependence upon high strength concrete and complex structural form. Although the hard rock present at the site could be expected to provide an adequate degree of stiffness and erosion resistance to support an arch form, and the aggregate able to be locally produced may also prove to be suitable for structural concrete production, the valley profile is not particularly well suited to a competitive arch form due to the relatively moderate side slopes. Arches become most competitive in tight gorge situations where conventional construction plant operation – (as would be deployed in CFRD or RCC construction) - becomes inefficient or even impractical, and this situation is not reached at this site. The original 1930's design reflects this position, as while arch dams were being constructed elsewhere in Central Otago at this time, an alternative approach was taken here. We note that this original decision was also partly due to the future raising potential of the CFRD solution adopted. Current design practice for arch dams is also much more complex than that applied to the old Central Otago irrigation arch dams. The slender single curvature (cylindrical) form previously adopted has proven to be susceptible to severe cracking particularly under differential thermal loads, and to present difficulties in establishing confidence in their seismic resilience. Modern designs adopt a double curvature (egg shell) form, which is more effective at transferring forces to the foundations without cracking. So while the arch form is not ruled out as a solution at this site, both the absence of local experience with current design practice, and absence of a strong case in its favour as a competitive alternative to CFRD and RCC forms has resulted in it not being considered further in this preliminary study.

3.4 Spillway Concepts and Freeboard

All options will need to safely discharge the adopted design flood flow of 700 m^3/s. As the existing morning glory spillway is not suited to raising beyond some 5m without very careful examination of its service performance, we have only included its retention in the RL567.5 FSL option. It was similarly retained in the previous +5m CFRD concept provided to the Falls Dam Company and illustrated earlier in this report.

Choking flow for this spillway at this moderate degree of raising is assessed to be some 470 m^3/s [OPUS 2007b] (Note; this is up from the current 430 m^3/s choking flow due to the additional head available). Any options that retain the existing spillway will therefore need to accommodate the additional discharge capacity of some 230 m^3/s through the use of an auxiliary spillway. Otherwise the full 700m^3/s discharge capacity will need to be provided, also recognising that the final design may need to accommodate even greater PMF flow without dam failure.

Freeboard requirements depend upon the nature of the flood surcharge rise, the consideration of wind generated wave action against the dam, and the consequences of varying degrees of dam overspill. For the purposes of this study we have not allowed for the possibility of a gated spillway being adopted, with its inherent ability to minimise flood surcharge rise. We are of the view that an irrigation facility of this nature requires a passive spillway concept with no reliance upon automated hardware nor human intervention to safety discharge the design flood.

Flood surcharges in this case will be well in excess of normally adopted wave run up margins for this site, so wave freeboard under FSL conditions will not be critical to design crest elevations. In considering moderate flood conditions well below the design flood, a freeboard of some 2m is expected to be required, subject to detailed assessment in due course; again this requirement is not expected to become critical to design. This leaves the freeboard to be provided under design flood conditions as the critical design consideration. For this study we have allowed for only a nominal residual freeboard beyond the flood surcharge rise above FSL, as the dam types being considered are readily able to have their wave wall height adapted to suit detailed design without incurring undue expense.

The previous +5m redevelopment option provided to FDC included a fuse plug arrangement in the auxiliary spillway to minimise freeboard and final dam crest height. This concept layout also retained a trafficable dam crest leading to the requirement for access bridging over the fuse plug embankments. For the purposes of this study we have deliberately adopted a different approach that does not involve fuse plug embankments nor bridging, and that would result in the crest no longer providing access either in flood conditions or possibly under any conditions if a track was not provided across the auxiliary spillway cutting at all. The earlier layout concepts as shown in Fig 4 and Fig 5 are still available for reference if required, and this +5m CFRD raising option has been carried forward for comparison purposes in this report.

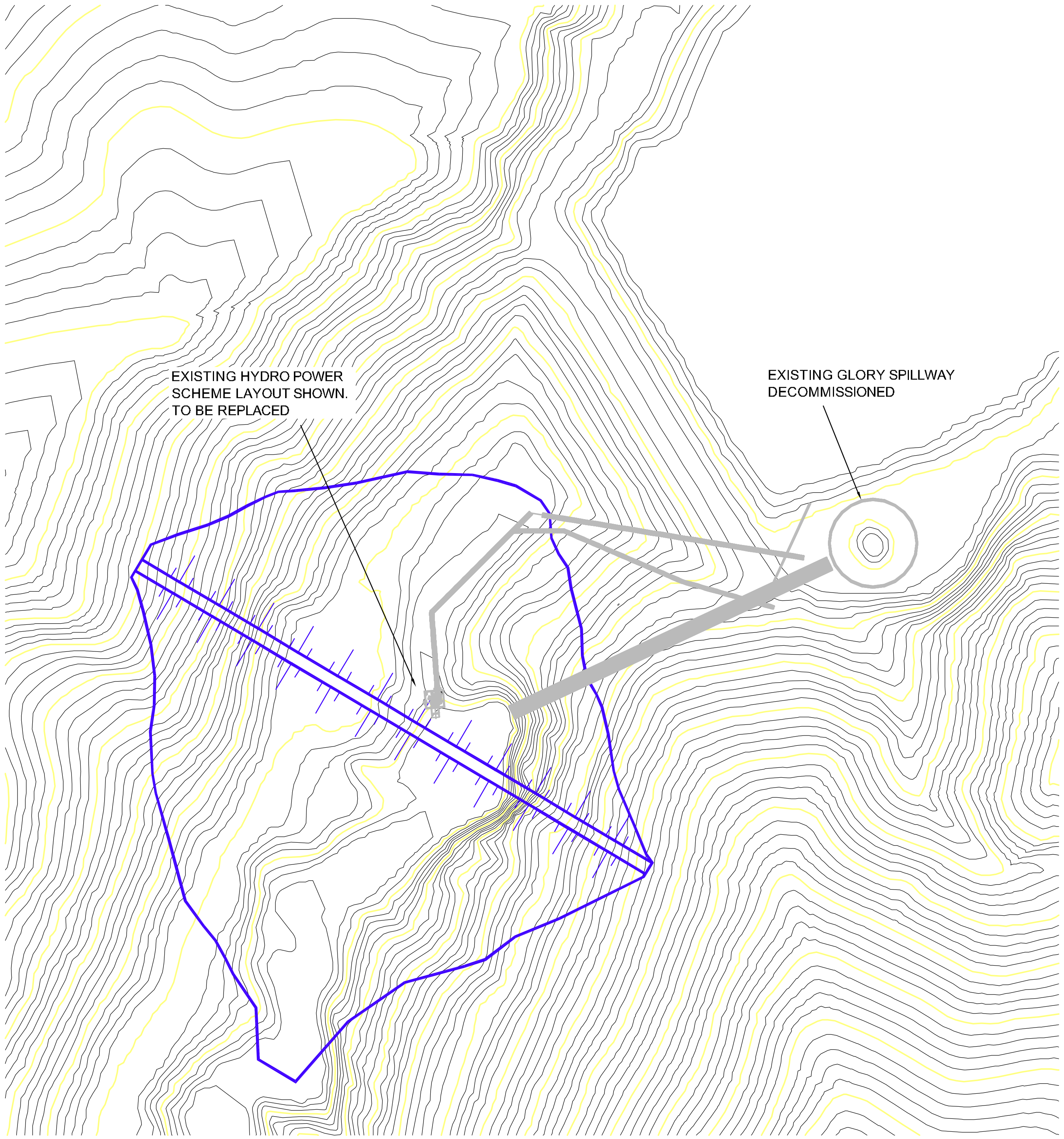
3.5 Diversion Layout Compatibility

A key driver for redevelopment being located at the current dam site is the ability to use the existing dam and offtake works for construction dewatering purposes, as this is potentially a major construction advantage compared to developing a green fields site.

The ability to accommodate various dam forms without seriously compromising the operation of the existing works has been assessed as shown on the following figures that outline some CFRD downstream layouts that do not satisfy this test, despite having the attraction of allowing totally new embankment and membrane construction free from any ageing asset design and deterioration concerns

577m PLAN

1:1000 at A1 size



588m PLAN

1:1000 at A1 size

Figure 33

PRE-FEASIBILITY

Revision	Amendment	Approved	Revision Date

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Drawn	Designed	Approved	Revision Date
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6-cwi04.13 / 25dd	As shown

Project

Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet

ENGINEERING PREFEASIBILITY STUDY
DOWNSTREAM CFRD 577m and 588m - OPTIONS

Drawing No.	Sheet.No.	Revision
7 / 1071 / 1 / 1704	13	

1:1 @ A1
1:2 @ A3
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Original Sheet Size A1 [841x594] Plot Date 04/09/12 @ 13:00 g:\miscellaneous clients\6cwi04.13 aqualinc research - manuherikia valley irrigation study\dwgs\cfrd options\7_1071_1_5704_13 ds cfrd 577 and 588 options.dwg - 13

3.6 Offtake Capacity

From the required peak offtake flow rates previously specified, and from the assumption that a mini hydropower scheme will continue to be accommodated, we have provisionally sized the conduits required to deliver these flows with acceptable head loss.

FSL	Peak Flow (m^3/s)	Nominal dia (m)
RL 588.0	11.0	2.3
RL 577.0	6.5	1.9
RL 567.5	4.0	1.5

3.7 Selected Conceptual Layouts for Prefeasibility Study

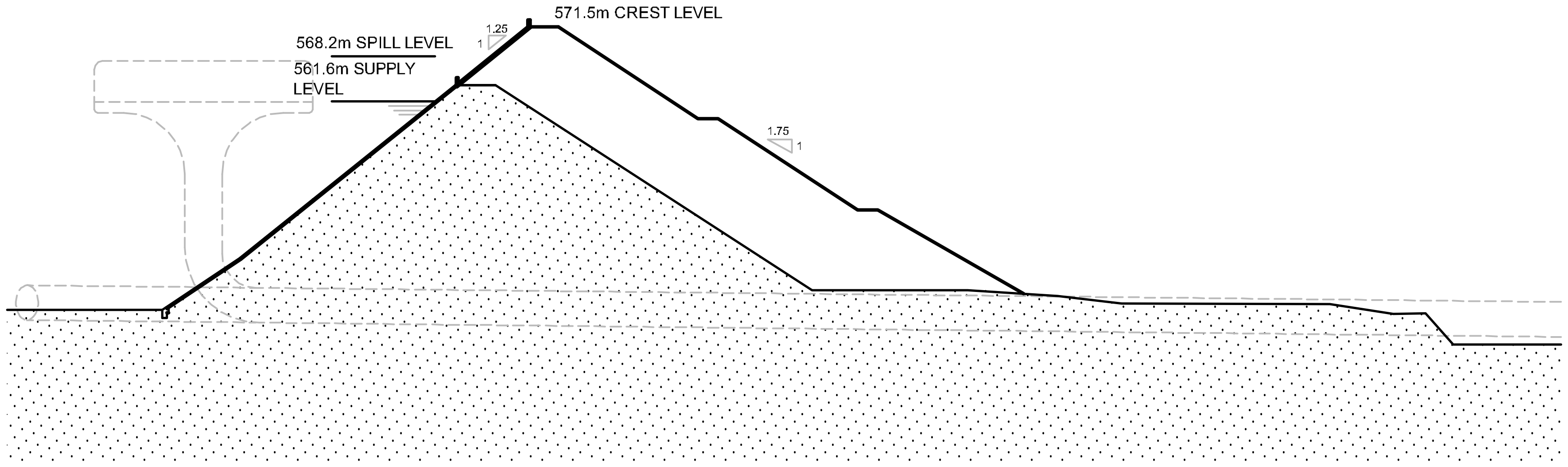
From the preliminary assessment outlined above, the stand alone CFRD options have been rejected, and following options have been carried forward for further consideration in the study.

FSL	Dam	Spillway
RL 588.0	Raised CFRD	left abutment spillway cutting
RL 588.0	RCC	Overspill, service + auxiliary stepped face
RL 577.0	Raised CFRD	left abutment spillway cutting
RL 577.0	RCC	Overspill, service + auxiliary stepped face
RL 567.5	Raised CFRD	Existing Morning Glory + Auxiliary left abutment cutting

As noted earlier, the previous +5m raised CFRD option findings have been updated and carried forward into this study discussion for comparison purposes.

These options are illustrated schematically on the figures included before Section 3.3 above, and the layouts are shown separately on the following figures including both the reservoir and the dams.

Possible reservoir margin access roading realignments are shown tentatively on the comparative reservoir inundation map included before section 3.3 above. Cut batters have generally been set at 2H:3V slope to be consistent with the similarly orientated cut face around the spillway bench that has performed satisfactorily.



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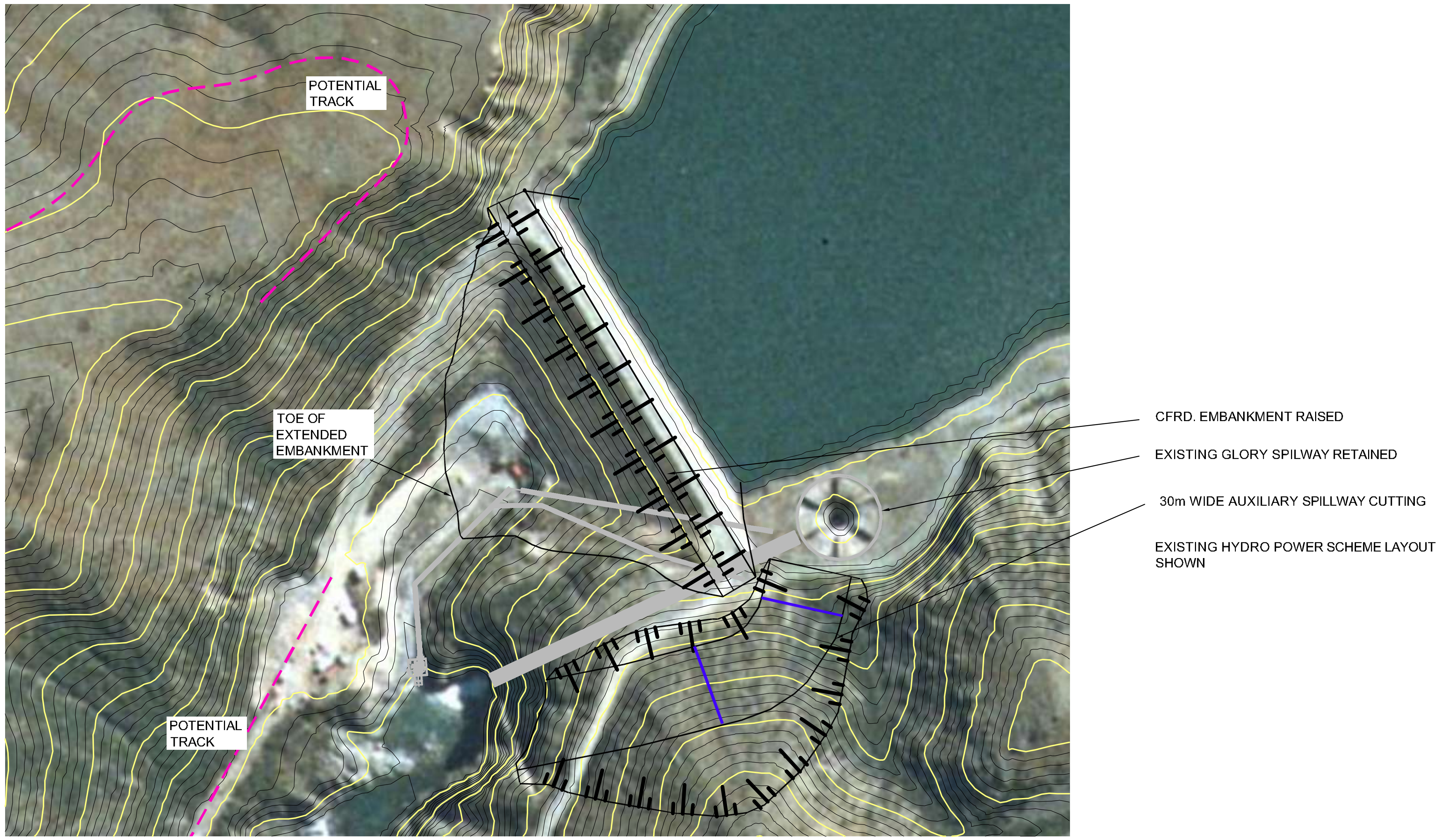

PLAN
1:1000 at A1 size

Figure 34

PRE-FEASIBILITY

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Scale: As shown

Project: Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet: ENGINEERING PREFEASIBILITY STUDY
CFRD 567.5m - OPTION

Drawing No. 7 / 1071 / 1 / 1704
Sheet No. 3
Revision

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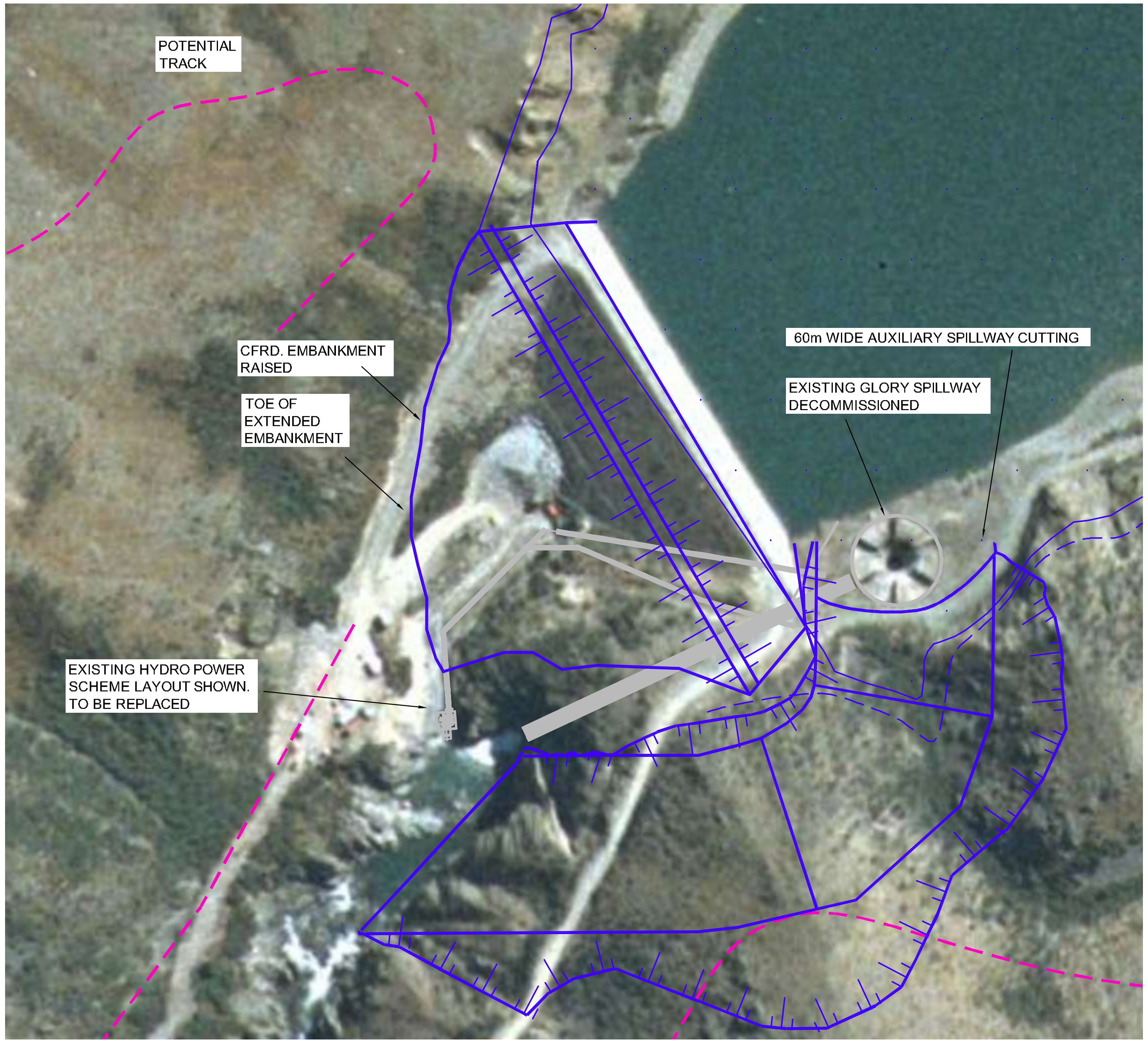
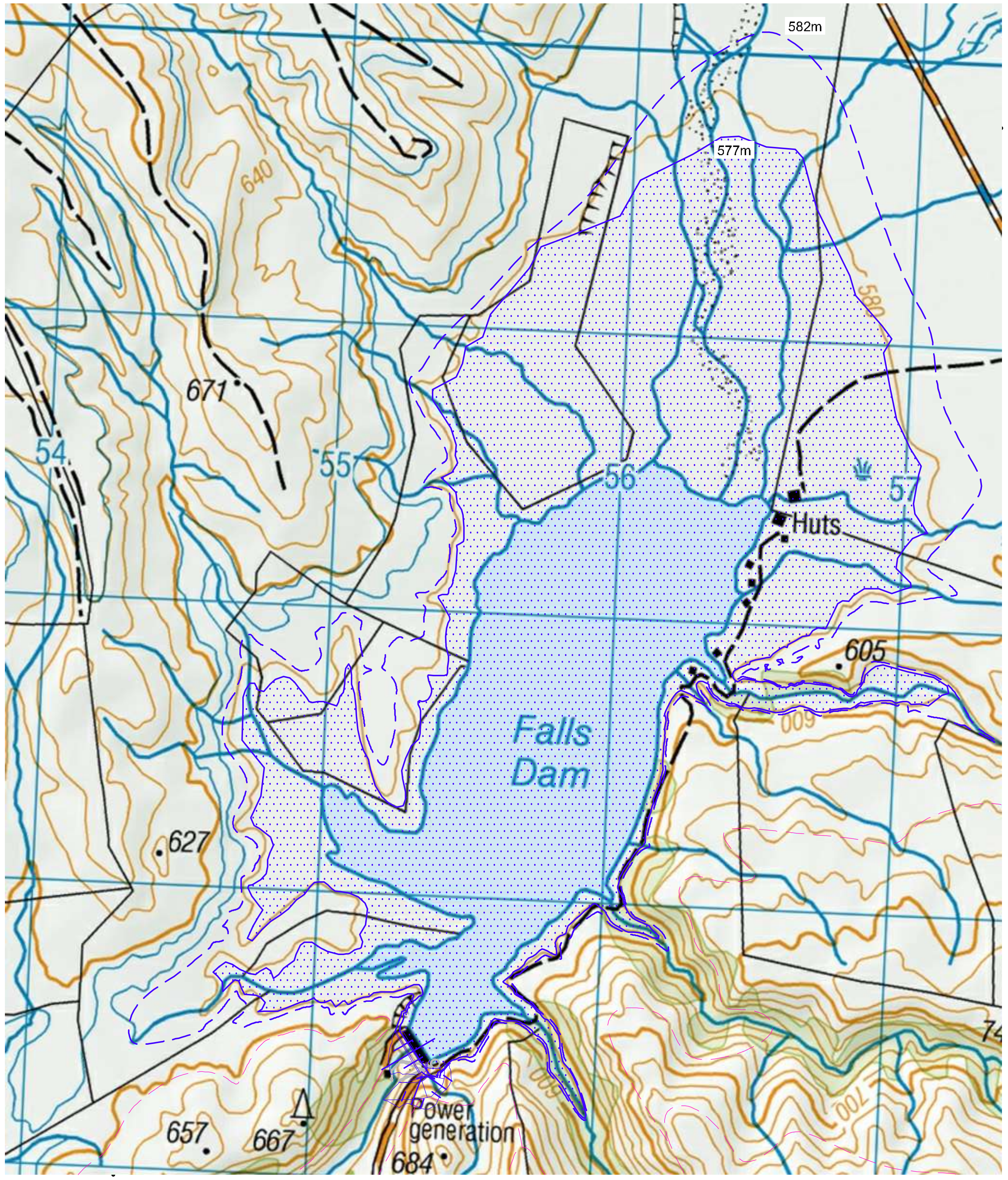


577m PLAN
1:1000 at A1 size



577m PLAN
1:10,000 at A1 size

Figure 35

PRE-FEASIBILITY

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Sheet
ENGINEERING PREFEASIBILITY STUDY
CFRD 577m - OPTION

Drawn	Designed	Approved	Revision Date
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Project No.	Scale	Drawing No.	Sheet No.	Revision
6-cwi04.13 / 25dd	As shown	7 / 1071 / 1 / 1704	4	

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1:2 @ A3
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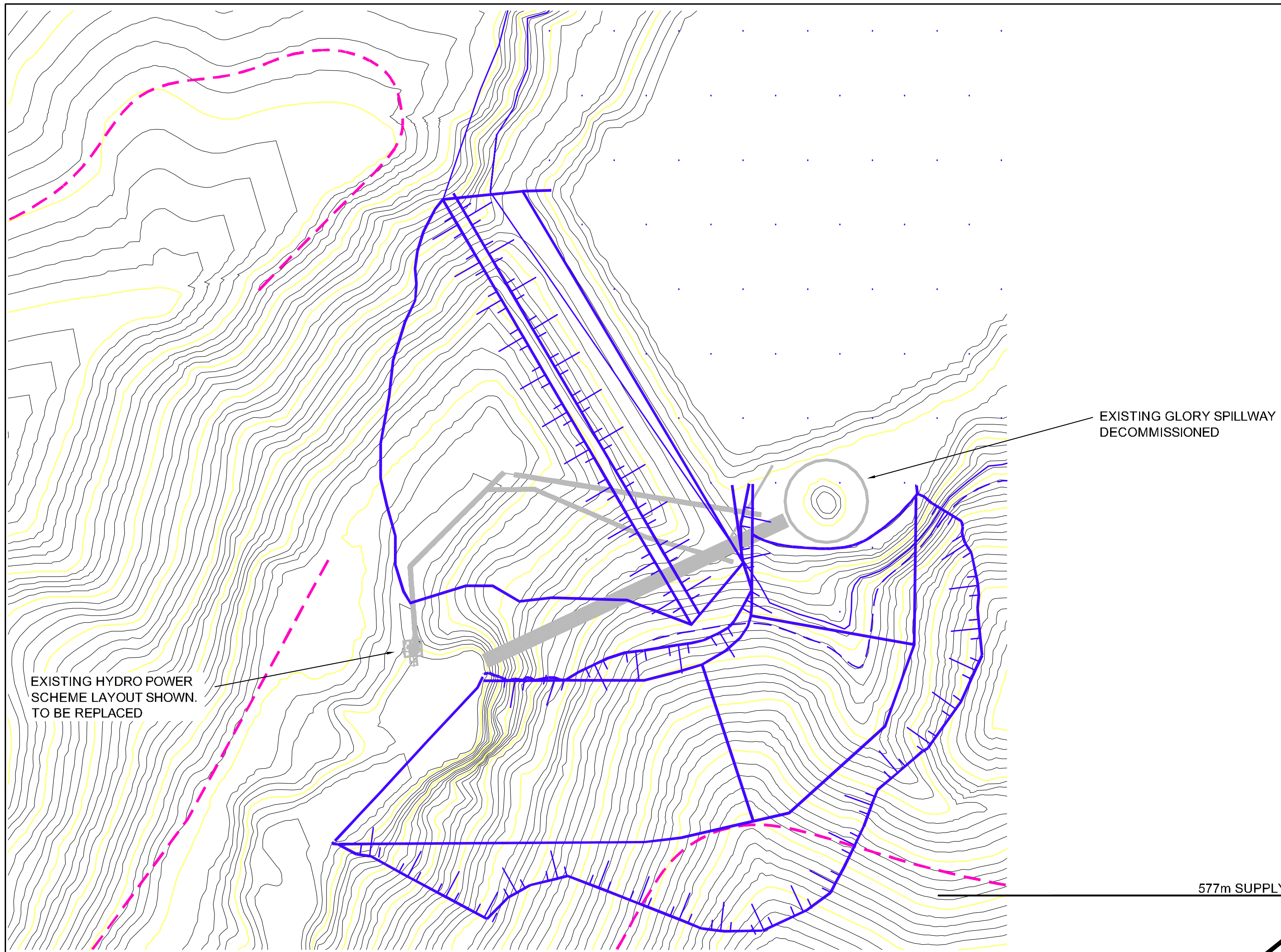
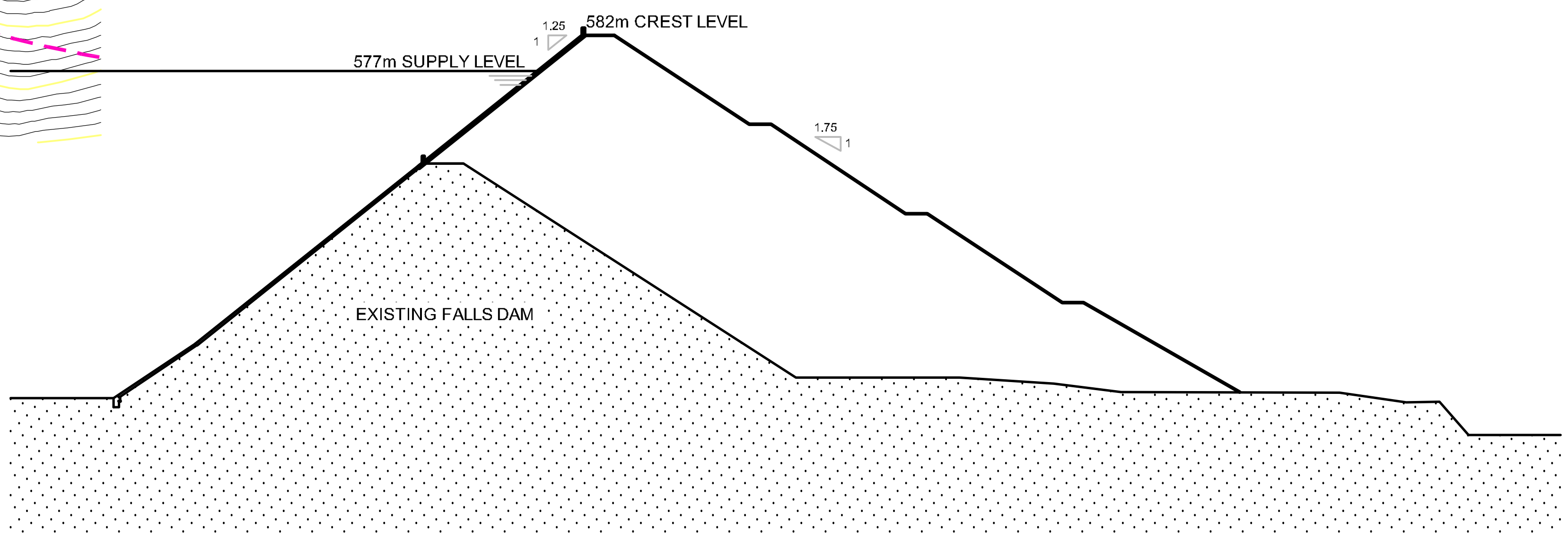
Original Sheet Size A1 [841x594] Plot Date



577m PLAN

1:1000 at A1 size



577m SECTION

1:500 at A1 size

Figure 36

PRE-FEASIBILITY

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Project No.	Scale
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Project
Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
CFRD 577m - OPTION

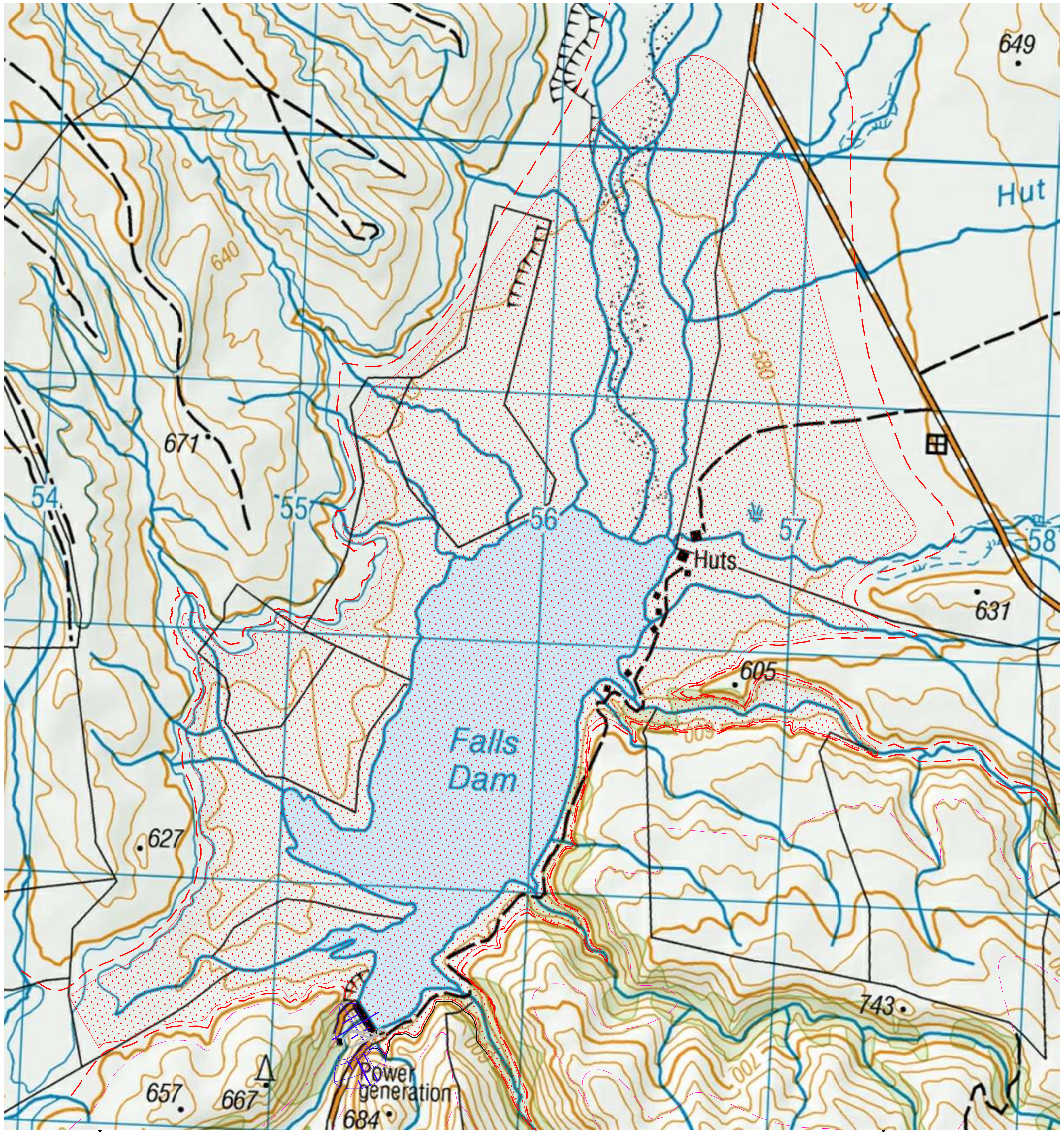
Drawing No.	Sheet. No.	Revision
7 / 1071 / 1 / 1704	5	

1:1 @ A1 1:2 @ A3 0 10 20 30 40 50 60 70 80 90 100 mm

Original Sheet Size A1 [841x594] Plot Date

588m PLAN

SCALE 1 : 10,000 (At A1 size)
SCALE 1 : 20,000 (At A3 size)



588m PLAN

SCALE 1 : 10,000 (At A1 size)
SCALE 1 : 20,000 (At A3 size)

Figure 37

PRE-FEASIBILITY

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Project: Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet: ENGINEERING PREFEASIBILITY STUDY
CFRD 588m - OPTION

Project No.	Scale	Drawing No.	Sheet. No.	Revision
6-cwi04.13 / 25dd	As shown	7 / 1071 / 1 / 1704	6	

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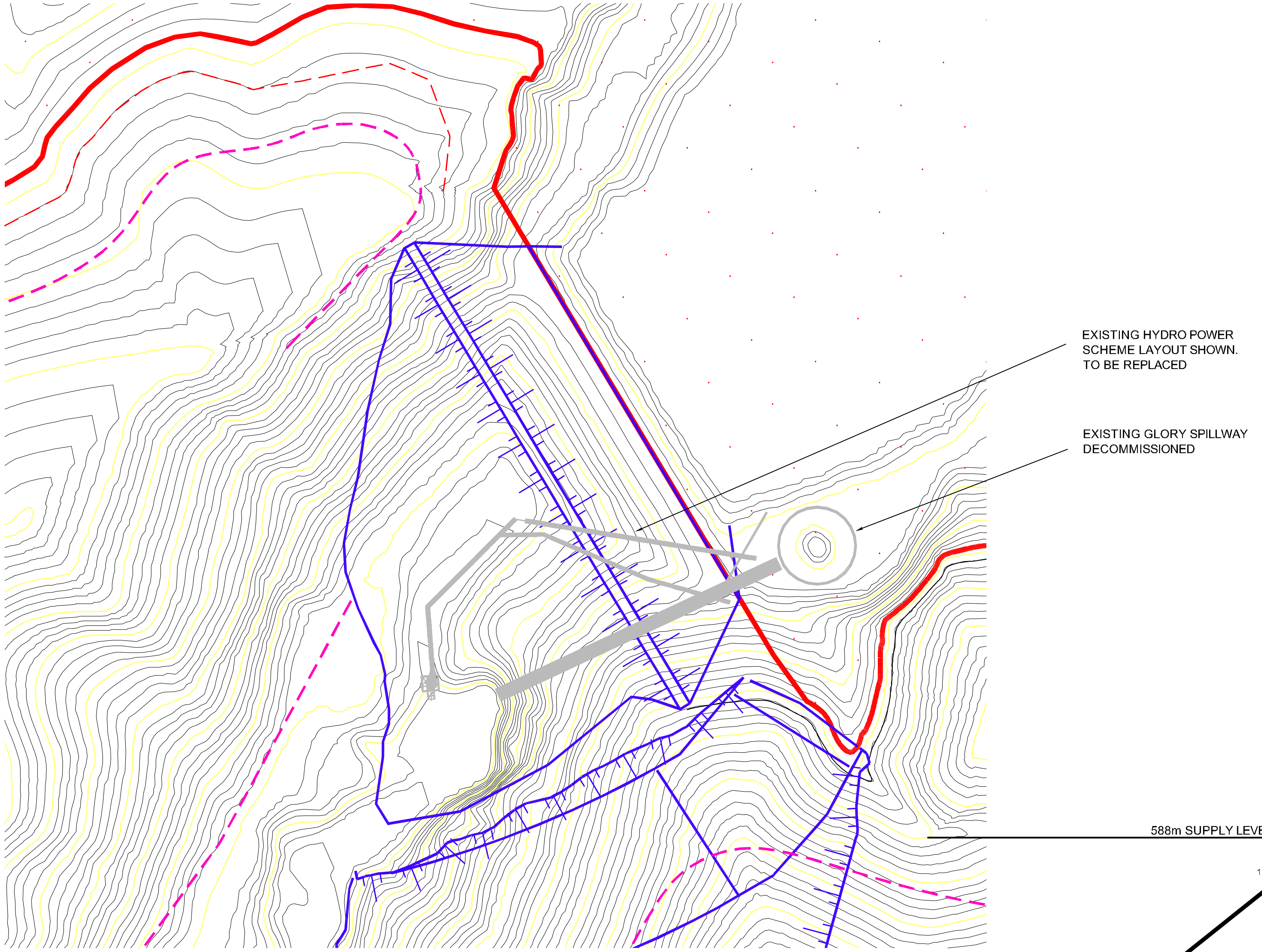
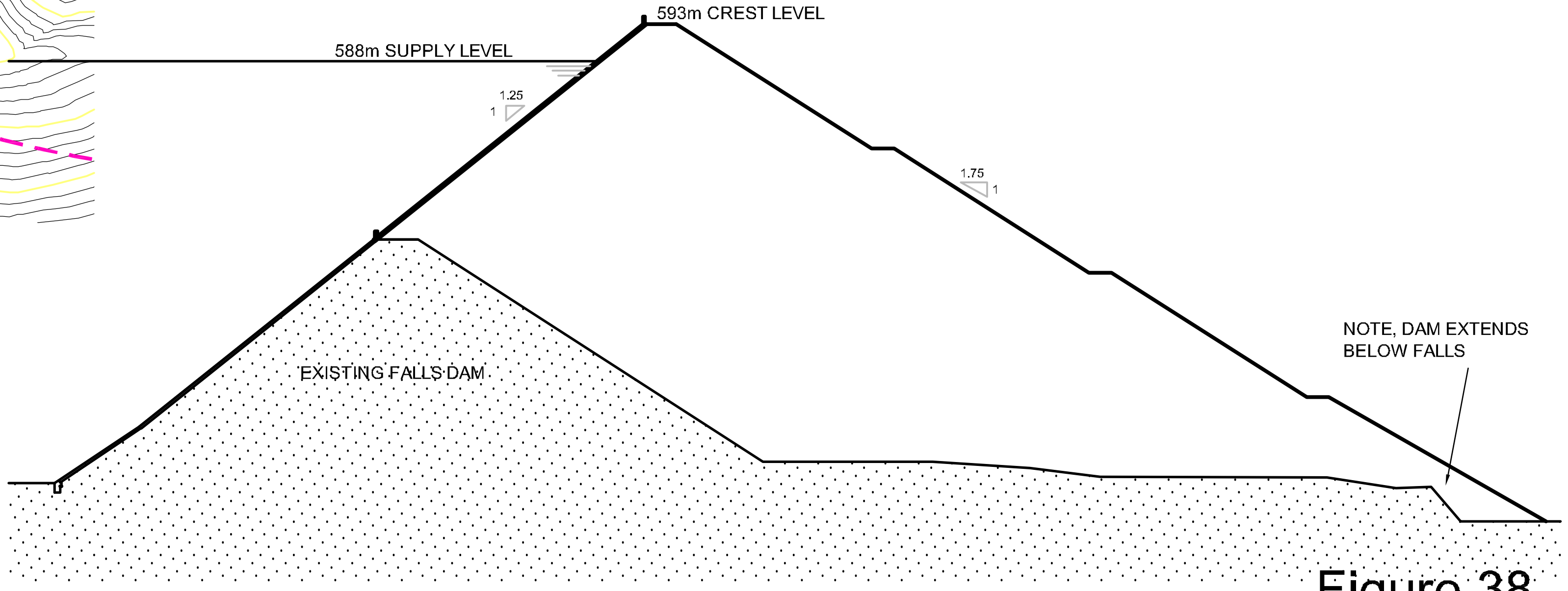
Original Sheet Size A1 [841x594] Plot Date



PLAN

1:1000 at A1 size



SECTION

1:500 at A1 size

Figure 38

PRE-FEASIBILITY

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Project No.	Scale
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Project
Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
CFRD 588m - OPTION

Drawing No.	Sheet.No.	Revision
7 / 1071 / 1 / 1704	7	

1:1 @ A1
1:2 @ A3
0 10 20 30 40 50 60 70 80 90 100 mm

Original Sheet Size A1 [841x594] Plot Date

3.8 No Storage Development ("Do Nothing") Base Option

Even in the absence of a decision to enhance storage at Falls Dam, there are some potential liabilities associated with asset deterioration and dam safety compliance obligations that need to be considered in the context of a "Do Minimum" scenario.

Definition of the "Do-Minimum" option for the Falls Dam Redevelopment Project is an important consideration insofar as it may influence subsequent option evaluation and selection processes. In the absence of any change in service delivery to water users, the "Do-Minimum" option is primarily driven by the need to provide an adequate level of asset security to deliver reliability of supply to customers, along with the dam owners public safety obligations in common law, together with those statutory responsibilities defined in the dam safety framework incorporated within the Building Act and the regional authority's policy on dangerous dams. In future, there may also be obligations arising from any resource consent conditions that may be imposed. In this context the relevant factors that affect the nature and scope of the "Do-Minimum" option include:

- Regulatory changes associated with changes to the relevant statutes and/or the associated regulations and regional authority policies. Such changes are currently anticipated from central government, but no milestone date is known by the authors at this time.

- Understanding the reliable performance expectations for the existing assets, incorporating aspects of the original engineering design standards adopted, together with in-service deterioration and any asset maintenance and upgrading work previously undertaken, along with any changes in the state of knowledge regarding hazard exposure and failure mechanisms.

- Clarification or reclassification of the potential impact classification (PIC) of the facility. The PIC is a function only of the consequences of dam failure and the resulting downstream effects or impacts on third parties of the uncontrolled release of the impoundment. The PIC has no relationship to the perceived likelihood of such a failure, so it is independent of the asset condition or original design standards. However, the current engineering design standards that apply are a function of the PIC classification, such that more stringent performance standards and resilience expectations for new work or for defining deficiencies apply as the PIC rating increases.

- Clarifying whether these performance expectations fall below the threshold defining a "dangerous dam" whereupon the special powers of the regional authority to intervene may come into effect. This threshold is set at quite low performance level, generally being defined as "in the normal course of events" or at a loading level likely to be experienced in the 50 year return period event or equivalently at an annual probability of 2% or greater.

- The need for any building consent application submission that may generate a review of safety performance standards applying to existing assets. Noting that the regional authority does not have power per se' to require existing assets to be upgraded in the situation where the dam is not classified as a “dangerous dam”, and where no other alterations or modifications requiring a building consent arise, despite the philosophy in the NZSOLD Dam Safety Guidelines that requires dam owners to be suitably informed of all deficiencies (relative to currently accepted design standards), and to have a remedial or refurbishment programme to address such deficiencies over a reasonable period of time.

- The minimum acceptable suite of performance criteria that would apply to a similar facility being developed today under the current regulatory regime, considering the degree of potential consequence of experiencing a failure (i.e uncontrolled loss of the impoundment). The difference between these currently accepted performance standards and the actual reliable performance expectations of the existing assets quantifies the degree of deficiency (if any) that needs to be addressed by the asset owner. This knowledge will be generated in the normal course of events from a comprehensive safety review (CSR) process that is expected to be commissioned by dam owners every few years (typically every 5 or 6 years).

- Any waiver of the obligation to achieve full compliance with current standards that might be accepted by the Regulator for this existing facility, when an engineering review of the facility occurs and when this matter comes formally to the regulators attention via a building consent process or similar.

- The time period stipulated by the Regulator within which the owner would be obliged to address any identified deficiencies when this matter comes formally to the regulators attention via a building consent process or similar.

In referring to performance criteria or design standards for dams of this nature, the key aspects affecting public safety objectives generally relate to resilience of the assets when subject to unusual or extreme flood and earthquake hazards. It is worth noting that these assets have been in service successfully for nearly 80 years, and while deterioration has occurred, the safety of the impoundment under normal operating conditions is not in question. The possibility of the facility being classified as a “dangerous dam” therefore seems remote in the authors opinion. Failure leading to uncontrolled release of the impoundment would almost certainly be associated with an unusual or extreme natural hazard event that exceeds the capacity of the existing assets. That is not to infer that substantial leakage might not arise, leading to the need for significant maintenance activity potentially affecting supply to customers.

While earthquake loading was not a major consideration in the 1930’s design of this dam, seismic resilience of concrete faced rockfill embankments on rock foundations has generally been very satisfactory. This behaviour is due to the embankment remaining well drained, and hydraulic forces applied to the concrete facing tending to act in a stabilising manner as the forces are transferred into the foundation. However, the steep batter slopes and uncompacted nature of the rockfill in this case does have some vulnerability to deformation around the narrow crest that could lead to loss of integrity of the joint seals in the facing and significant leakage through the rockfill. Furthermore, the copper joint seals at this site have previously failed in service during the 1980's, and the temporary repairs implemented at that time are now well past their intended service life. However, while this situation, along with freeze/thaw induced deterioration of the concrete facing panels, does present an important outstanding maintenance liability, it is not anticipated that seismically induced deformation would lead to catastrophic loss of the impoundment through internal leakage nor overtopping though loss of the substantial freeboard available. Assessment of this matter will benefit from detailed investigation of the seismic hazard at this site and detailed analysis of the embankment deformation response, but the overview outlined above is not expected to change significantly.

Flood handling capacity on the other hand is a matter that has received considerable attention at this site, due substantially to the reliance upon a morning glory spillway for all flood discharge duty. An important consideration in engineering design is the manner in which an overload condition can be handled. When working with flood discharges, the definition of a given design capacity is not a fixed limit on the flow that the dam may be subjected to. Rather, there is always the potential for a super design condition to occur, albeit at a low likelihood. A resilient design is one which can experience a super design condition without disproportionate damage or catastrophic failure, much in the same way that conventional structures are designed to act in a ductile manner with controlled deformation, rather than to experience catastrophic brittle failure.

The morning glory spillway at Falls Dam has the characteristic that it chokes at a discharge of some 430 cumecs, and it cannot convey significantly more flow as the reservoir continues heading up above this point of choking. This capacity matches the original design capacity expectation that was considered to provide for a flood with a 500 year average recurrence interval (ARI), or more correctly a flood with an annual exceedence probability (AEP) of 0.2%. This design flood event is well below that which would be adopted for a current design at this site, so the flood response to larger inflow events is an important dam safety consideration. The storage volume in the reservoir will continue to absorb any excess inflow until freeboard is lost, but this manner of providing flood capacity is not considered to be resilient to extreme flood events, and it would not be adopted for current designs without further provision for extreme flood discharge. To put the matter into a quantitative context, there are two flood studies (c1984 and c2007) that have been undertaken to gain a better insight into the flood characteristics at this site. The c1984 report essentially confirmed the original design discharge values for this catchment, and found that freeboard would be lost under a flood some 30% greater than the 0.2% AEP event. However, the probable maximum flood (PMF) relative to the 0.2% AEP event was not calculated.

My previous view of these findings has been that the situation was not acceptable in terms of establishing a defensible position for a prudent owner meeting their public safety obligations, and that the spillway capacity represented a significant design deficiency that needed to be addressed. My suggested approach advised previously to the Falls Dam Company c2003 was to construct an auxiliary overspill spillway cutting on the left abutment to provide additional discharge capacity for extreme events that exceeded the choking capacity of the morning glory spillway. So that this auxiliary facility would only come into operation during extreme events and still maintain safe freeboard, I included wash out fuse plug fills in the auxiliary spillway and an overbridge to prevent trafficking and over compaction of the fuse plugs. As the auxiliary spillway cutting would generate significant volumes of good quality rock, there was an opportunity to constructively utilise this material to raise the embankment dam slightly to improve storage volume, rather than simply dump the surplus cut to waste. The cost of the auxiliary spillway work was included along with the deferred maintenance costs within my previous c2003 minimum asset management obligation.

In 2007 a further flood study using different methods including specific focus on the PMF discharges was undertaken; this report is still in draft status as it has not been published as a final document. In reviewing the probabilistic flood events, the report came to a preliminary finding that the earlier 0.2% AEP discharge was significantly overstated, and reduced the value by some 33% to match the available flow records. The analysis undertaken showed that a 0.02% AEP flood (i.e. 5000 year ARI) could be handled by the existing spillway without choking. The full PMF flood was shown to greatly exceed the spillway capacity, and would result in overtopping failure of the dam. Even a 50%PMF flood would result in total loss of freeboard and overtopping of the embankment.

The question of the acceptability of the existing spillway capacity then hinges on the defensibility of the current position should an incident occur, or on the design standard that may be set by the dam safety Regulator in any circumstance where this action may arise. The risk to normal commercial supply obligations does not really enter into this decision as the probability of the event is so low, and is effectively limited to life safety decisions. This matter of defensibilty has not yet been determined, so the following discussion is my view on the likely position. In principle, the flood capacity performance standard to be applied is dependent upon the degree of consequences of uncontrolled release of the impoundment (as represented by the potential impact category, or PIC, for the site; Low, Medium, or High). The preliminary PIC for this site is Medium, but this could change to High when an updated assessment of dam break effects is undertaken. For a Medium PIC site the minimum design flood capacity requirement falls in the range 0.1% to 0.01% AEP (or 1,000 to 10,000year ARI). Given the Falls dam situation, the requirement would fall in the upper portion of this band, say 0.02% to 0.01% AEP (or 5,000 to 10,000 year ARI). It is therefore possible that the status quo may be acceptable if the draft c2007 flood study findings are confirmed. If the site PIC rating was to be established as High, it is most unlikely that the status quo spillway capacity would be acceptable.

So in terms of defining the “Do-Minimum” option for the Falls Dam project, I trust the above discussion highlights the nature of the uncertainty surrounding the issue. While it is possible that working through the various studies and regulatory processes could plausibly result in the existing spillway capacity being found to present an acceptable level of risk, I consider that a more cautious view should be adopted for current purposes. I suggest that it is reasonable for allowance to be made in the Do-Minimum” option for investment in providing auxiliary spillway capacity for safely handling extreme flood events. The matter of whether the “Do-Minimum” option includes for utilising the “surplus” cut to waste rock to raise the dam to increase live storage is really an executive decision matter, but in engineering and hydrological terms there is a strong case for gaining the benefits available from enhanced storage through raising levels some 3m to 6m.

In all cases the "Do-Minimum" option for the Falls Dam project should include for all deferred maintenance liabilities that have been reported elsewhere. These liabilities can be partially or fully avoided of course for options that entail replacement of the existing dam and spillway assets.

4 Raised Concrete Faced Rockfill Dam Options

4.1 Previous +5m Option

The previous +5m option was developed to balance the auxiliary spillway cutting volumes to the embankment fill demand. This necessitated moving the spillway cut close to the left abutment such that cut volumes were kept down, and surplus rock material was not needing to be cut to waste.

The scope of repairs to the existing dam established for this scenario are applicable to all CFRD raising cases, and included both membrane joint repair and concrete repairs. Specific joint repair details have not been subject to detailed design, so the precise nature of the repair technique is yet to be developed. The matter of access to the section of membrane underwater is also a key consideration, as it has been assumed that this repair will be undertaken when the reservoir is sufficiently low at the end of an irrigation season to make this a practical proposition. There are clearly some risks and uncertainty involved in this work, which directly carry over to all the CFRD raising options being considered, and which remain key design and construction factors in all these options that depend on retaining the existing dam asset.

Raising the existing spillway sill was based upon the use of 48 precast segments placed upon a new insitu concrete ring beam at the perimeter of the spillway. The sketches have been kept simple for illustrative purposes, but further details covering extension to the nappe, guide vanes, and aeration details were also provisionally allowed for. These considerations transfer directly into the RL567.5 FSL CFRD option, albeit with more attention to the higher spillway raising involved. Careful analysis of the hydraulic behaviour of this concept was highlighted as a factor for design attention, and this view carries over to the current RL567.5 CFRD option.

As mentioned above, the auxiliary spillway for this +5m scenario relied upon fuse plug fills that were intended to erode out when extreme flood rise generated overtopping. This detail has been replaced with a simple broad crested spill weir rock cutting in the current options, with no access retained over the dam crest during flood discharge conditions. Access in normal reservoir conditions for the CFRD 567.5m option would be possible utilising a simple track constructed across the spillway cutting, but this approach would be of limited practicality for the higher CFRD options that do not retain the operational morning glory spillway. Specific access to the hydropower assets is to be covered is a more focussed separate study on the hydropower aspects arising from each development option, but a bridge crossing of the river downstream of the dam site appears to offer some advantages, particularly if such a facility was to be constructed as part of the construction methodology anyway, and was simply to be upgraded to a permanent feature. This aspect will require more development in any subsequent feasibility study where access works in the spillway area can be assessed against alternative routes, and where construction methodology receives further detailed consideration.

The existing hydropower scheme supply works were seen as being retained in the +5m scenario, and this situation has been carried over to the 567.5 FSL CFRD option, albeit with greater likelihood of detailed design conflicts arising in terms loading conditions on the trenched steel pipes, access for future pipe maintenance etc.

The increase in embankment height was identified as placing demands on the strength and stiffness of the existing dumped rock fill, particularly under seismic loading. This was one of the key reasons for keeping the new imposed loads associated with raising to a moderate value. This is just a matter of degree, but it flags a need to apply much more caution and engineering rigour as the loads are increased, especially beyond +5m. These CFRD dams are intrinsically stable under water load, and have been constructed elsewhere to a great height, but the stiffness is also important in terms of supporting the stiff concrete membrane and the panel joints. The +5m scenario included a steep coping wall detail at the crest to minimise overall downstream fill requirements. We have avoided this potential stress raising detail for the options in the current study, and adopted a more generously proportioned embankment.

4.2 Spillway and Freeboard

Rating curves for a raised morning glory spillway have been previously developed [OPUS 2007b], where an indicated 470 m^3/s may be passed by the service spillway. We have set the auxiliary sill 1.2m above the raised service spillway to minimise spill down this path in other than major flood events, and thereby allow a lower standard of serviceability than would apply to a facility in regular use. A 30m wide broad crested weir has been adopted for the RL567.5 FSL CFRD auxiliary spillway, requiring a surcharge of not less than some 3.6m, giving a total potential flood rise of around 4.8m above FSL.

The other higher CFRD options require at least a 60m wide broad crested weir main spillway rock cutting, at FSL which in turn will lead to a flood surcharge of at least 4.8m.

The surcharge figures for the broad crested weir are sensitive to the discharge coefficient used, which in turn is dependent upon the detailed geometry of the spillway. This aspect will require careful attention at later design, to determine if the preliminary Cd value of 1.12 adopted is appropriate for the dimensions in this case. It is also appropriate to note that the previous +5m scenario was not directly based on the 700 m^3/s discharge capacity requirement used in this study, although it was similar.

As previously mentioned we have allowed for little freeboard above the design flood surcharge in this study, as the wave wall dimensions can be readily increased to suit detailed design requirements.

The degree of in service damage that may reasonably be sustained by the rock cutting will be a key design consideration. We have assumed limited concrete works for the spillway cutting in all cases, other than localised face treatment and structural crest works. Kinematic stability of the discrete rock blocks under progressive erosion mechanisms will require careful attention during design.

4.3 Dam Cross Sections

We have assumed that the existing upstream batter of the raised embankment will need to follow the existing steep 1.25H:1V slope, despite this being somewhat steeper than might be adopted for seismic resilience under todays design methods.

We have reduced the downstream batter to an effective slope of 1.75H:1V to ensure that it performs well under extreme seismic loading. The greater density and interlock achievable in the rockfill with modern compaction equipment will also improve seismic performance in this regard.

Fill present at the true right toe of the dam was placed and compacted there in 2003 as part of the excavation work undertaken for construction of the mini hydropower scheme. The placing was undertaken in the knowledge that it may in future form part of an extended embankment.

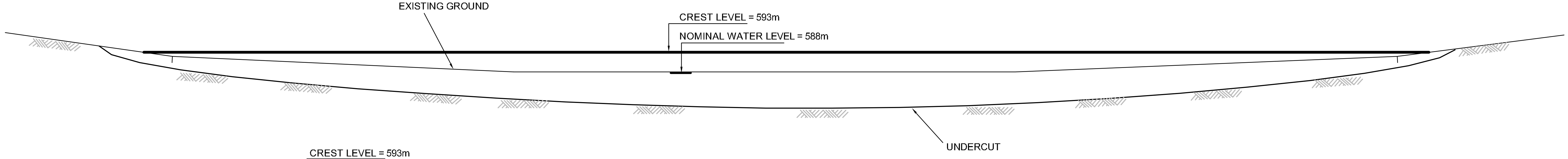
We have allowed for limited internal zoning of the raised embankment; only providing for face preparation under the extended membrane to achieve uniform rigid support. This matter may benefit from further detailed consideration during design, to see if there are benefits to be gained by further processing of the rock into selected zones. It is quite common to adopt a lower permeability zone on the upstream of these embankments to limit leakage flow in the event of membrane joint failure.

Structural concrete facing of not less than 300mm thickness has been adopted for preliminary scoping for the membrane, and a simple coping wall with suitable wave control has also been assumed. The panels would probably be slip formed in alternate bays rather than using the panel approach as in the past.

4.4 Plan Layouts and Footprint

The presented layout drawings included above show the general effect on the existing works, including the degree of encroachment on the existing access adit, increased loading on trenched pipelines etc. Generally the adopted layouts appear practical, although the RL588 FSL CFRD option does encroach well downstream into the pool below the falls. It would be desirable to avoid this condition if possible, and thus reduce the risk of toe erosion under extreme flood discharge.

The RL588 FSL options also lead to the need to construct a retaining dam at the Shamrock Gully saddle, some 1.6km from the main dam site. A possible dam concept is shown on the figure below, based on very preliminary scoping of the work. A zoned earthfill design has been selected as probably most suitable for this site, although no foundation information has been sighted nor detailed site layout considered. This structure has the potential to become quite a significant work in its own right, and it should be investigated thoroughly if the high FSL option is to be progressed. While the Figure overleaf is drawn to scale, the topographical surface and foundation preparation elevations are very approximate and not based on any specific site information other than the available 20m interval contour mapping.



LONGITUDINAL SECTION
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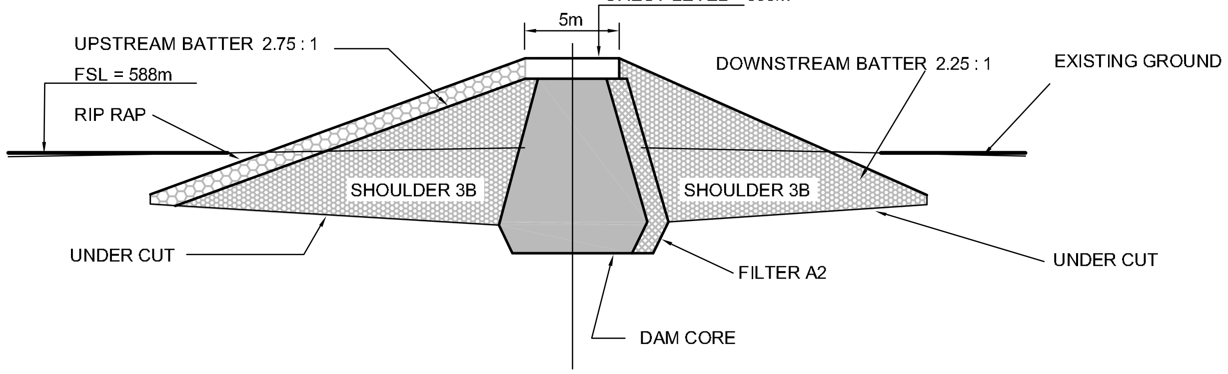

NOTE: FOUNDATION CONDITIONS NOT DETERMINED

SECTION 1 - 1
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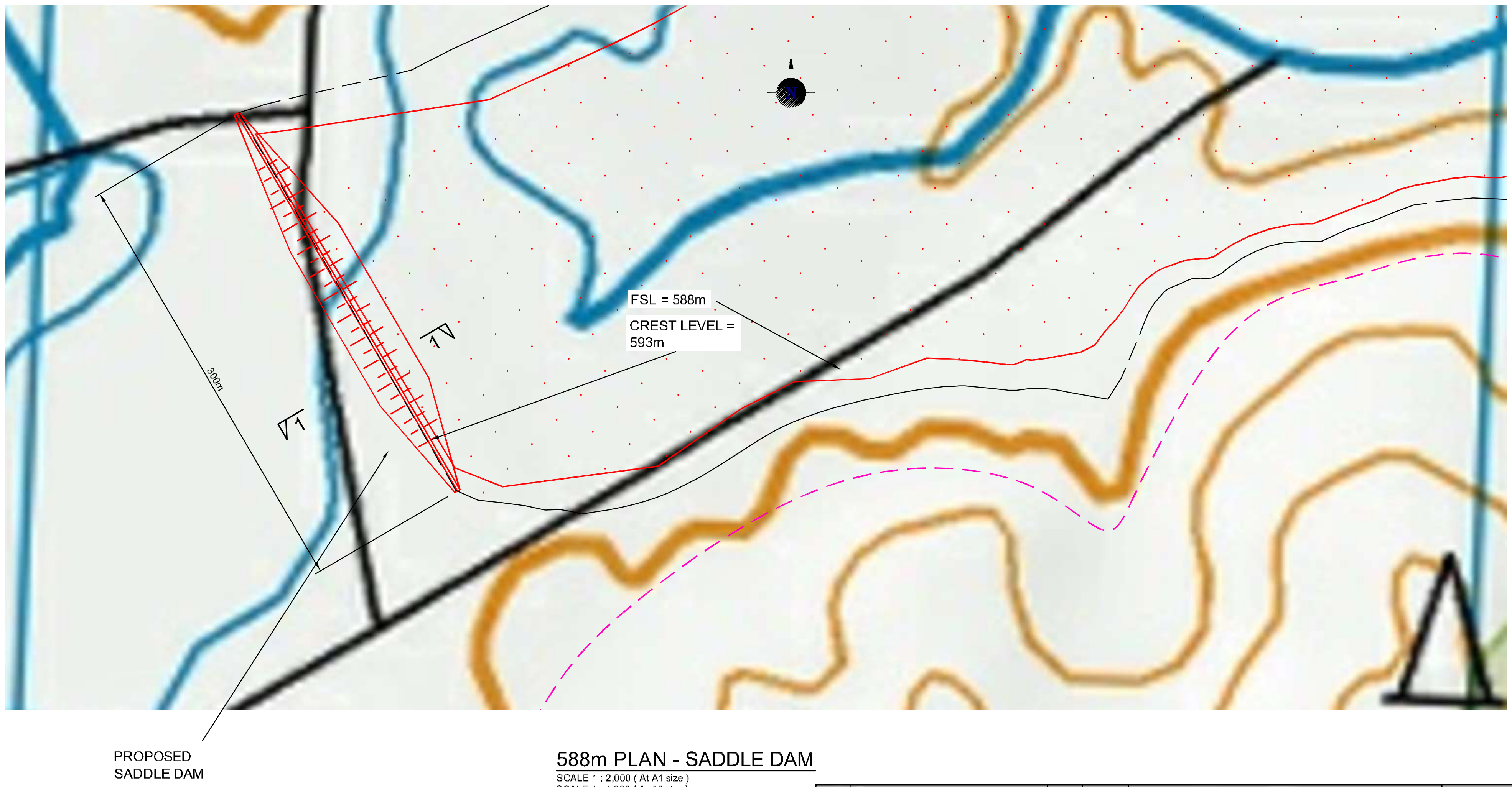

588m PLAN - SADDLE DAM
SCALE 1 : 2,000 (At A1 size)
SCALE 1 : 4,000 (At A3 size)

Figure 39

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Project
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FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
588m - OPTION - SADDLE DAM

Drawing No.	Sheet. No.	Revision
7 / 1071 / 1 / 1704	12	

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1:2 @ A3
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4.5 Foundation Treatment

Foundation treatment for the CFRD dam will entail minor stripping of the footprint, and more detailed preparation on the line of the extended plinth works on each abutment. The plinth may be associated with either a cut off trench as before, or a structural concrete slab well anchored to the rock surface.

Injection grouting at the plinth line is expected to be required, although the original construction experience was that grout takes were generally very low. There may be increased foundation permeability experienced with increasing elevation, reflecting the degree of relaxation of the rock mass due to the progressive river down cutting process. This aspect should be specifically investigated by subsurface downstage packer testing at selected locations.

Otherwise little preparation effort is envisaged as being required over the remaining footprint. Seepage from the original now isolated construction drain under the embankment, and subsurface drainage from the access adit has been directed to a new collector pipeline and directed to the V notch measuring weir at the power house. These features along with the existing penstock pipelines will need to be examined in detail for treatment or removal under the various options.

No permanent foundation drainage works are envisaged to be required at the dam embankment site, but the increased water pressures arising from the reservoir may lead to grouting and/or relief measures being needed at features such as underground concrete linings etc. This aspect will need to be specifically investigated, probably through the installation of groundwater instrumentation (piezometers) to inform detailed design.

4.6 Rockfill Borrow Sources, Construction Methods and Working Areas

The 3D modelling work undertaken for this study shows that for the option layouts selected the spillway cut volumes are sufficient to supply all the rock fill requirements. On this basis layout of the working areas will need to suit this material handling process. There is very little working area available to accommodate processing equipment and stockpiling, so there will be a strong incentive to minimise such processing between borrow and fill placing.

Solid volumes for the various options are included in the Appended scope quantification and cost estimation.

We have not produced a construction programme as part of this preliminary study, but the sequence of work will entail;

- foundation preparation and plinth construction, followed by
- quarrying and embankment filling, then
- concrete facing after the embankment has settled in response to the additional loading, and finally,

- permanent spillway concrete works.

We have assumed that the current spillway and some form of offtake works will continue in operation over the construction period until the concrete membrane is up to full height and can safely act as the primary impoundment. There will then be a period during which the existing spillway can be modified to suit the various development options.

Volumes of material to be handled are included in net solid volume terms in Section 6.5 and in Appendix. A

4.7 Offtake works and Hydropower Scheme

Various layout options for the offtake works associated with CFRD raising were considered during this study, and two concepts were arrived at for scoping the project. The lower RL567.5 FSL option retains the existing spillway tunnel in service, and has no increase in peak offtake capacity. On this basis the offtake works layout remains essentially unchanged from the current arrangement, leading to the need to maintain access into the access adit and underground valve chamber. To achieve this ongoing access, a corrugated steel plate arch structure has been adopted to daylight the existing adit to the new embankment toe. Apart from detailed attention to penstock pipe loading conditions etc., the mini hydropower scheme remains substantially unchanged, with reservoir water being conveyed via the current pipelines, subject to detailed analysis of the structural implications of increased pressures.

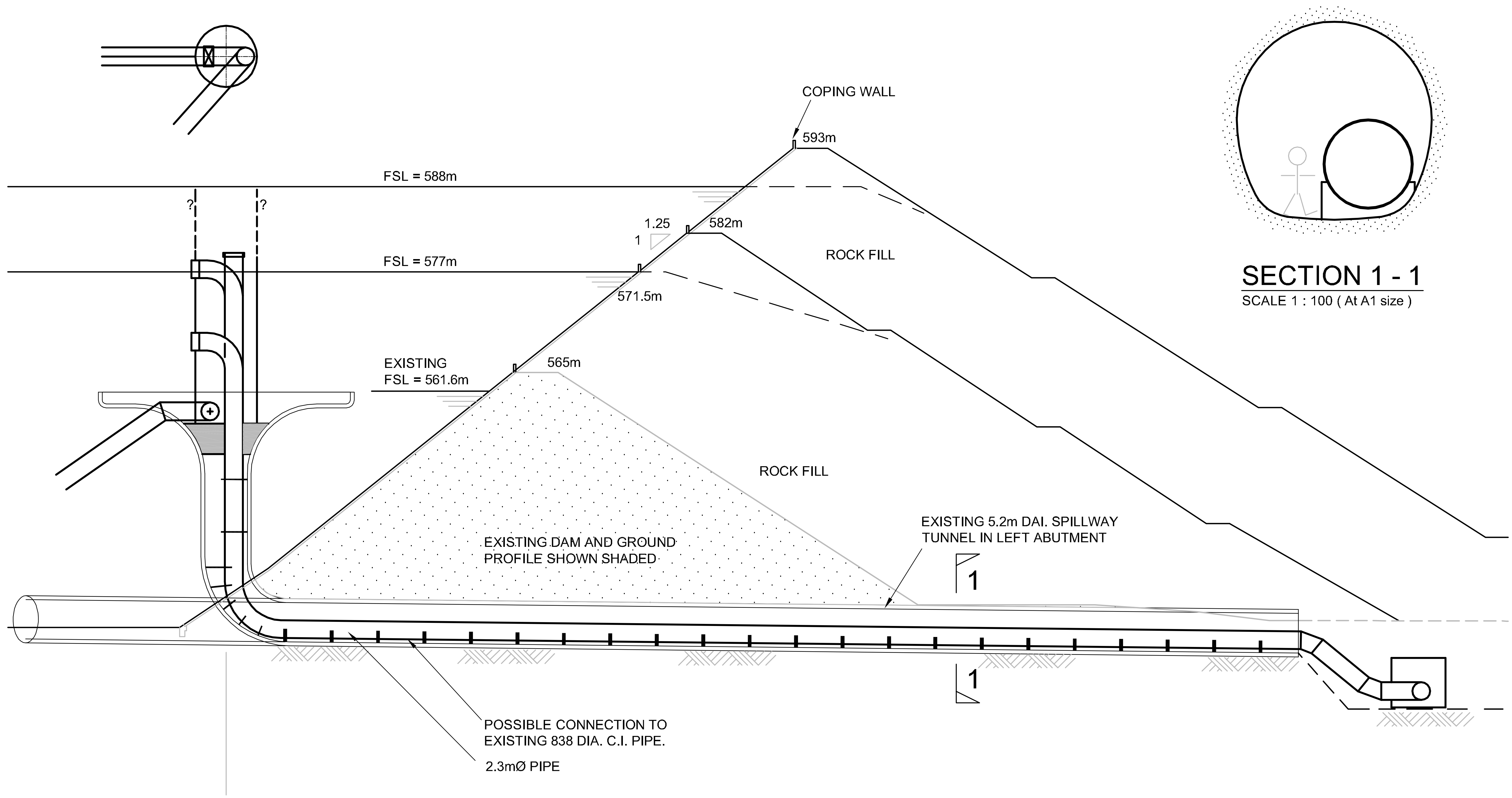
The higher RL577 FSL and RL588 FSL CFRD options present quite different challenges, as the existing spillway tunnel will become decommissioned and isolated from the reservoir for these cases. This situation presents opportunities to use the existing tunnel for other purposes, such as housing conveyance pipelines. Furthermore, the increased offtake capacity associated with these higher FSL options effectively renders the existing hydropower scheme obsolete in its present configuration, although some of the plant may be able to be deployed in a future scheme; probably elsewhere.

The adopted concept is shown on the figure below for the RL588 FSL option, although the layout will be similar for the RL 577 FSL case, just scaled down accordingly.

The concept is based upon seeking to provide access to pipelines and valve gear wherever possible for maintenance purposes. To this end, a dry valve tower is located over the existing morning glory spillway, housing the multiple offtake valves with actuators inside the chamber. A syphonic low level intake is also provided to allow drawdown to around RL 555m in a manner similar to the current smaller syphonic penstock servicing the hydropower scheme. By this means the limitation of the existing tunnel bulkhead pipework is avoided, although the structural capacity of these original works under increased reservoir level will need to be fully analysed.

For the higher RL577 FSL and RL588 FSL CFRD conceptual layouts, we have allowed for a new power house to be constructed in the invert of the tunnel outlet portal to gain access to the lowest TWL to achieve maximum potential generation, as at present.

We note that the hydropower implications of all CFRD development options are to be the subject of a separate specific study, and this report has only considered the conceptual factors insofar as they are needed to define a demarcation point for the supply works that would either deliver irrigation discharge or supply the hydropower scheme. We have assumed this demarcation point for the higher CFRD options to be near the diversion tunnel outlet. These supply works upstream of the demarcation point are included in the scope of the preliminary option costings in this report, whereas any investment in the hydropower scheme downstream of this demarcation point is excluded; to be considered on its individual commercial merits. In reality the integration of hydropower and irrigation objectives is more complex than this oversimplification infers, as the use of the water resource as represented by the scheme operating rules will require careful definition in any further feasibility studies.



DAM SECTIONS

Figure 40

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Project
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FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
HYDROELECTRIC OPTION - SCHEMATIC

Drawing No.	Sheet. No.	Revision
7 / 1071 / 1 / 1704	14	

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4.8 Permanent Access Tracks

Construction access around the immediate site to facilitate construction activity is included in the scoping of options and preliminary cost estimating.

However, given that land and property considerations are not included in the scope of this engineering study, access for land owners, and possibly the public, is not included in the scope of works for cost estimation at this stage.

Notwithstanding this exclusion, possible layouts for scoping future feasibility assessments along with land and property considerations are illustrated conceptually on Figure 31 above showing the various raised reservoir extents, some of which will not be required as the low level track (6km.) may be replaced with the one at a higher level. There is potentially a track length of 20.5km required, and up to 20 culverts, two of which may be bridges. The scope of this work is not insignificant to the project, and the possible layouts should be examined in some detail as part of land and property considerations.

We have also not allowed for permanent access across the dam crest in these options as was previously allowed for in the +5m raising option.

4.9 Potential for Progressive Development

Given the dominance of the spillway concept to the CFRD raising options, it is this factor that dictates the practicality any progressive raising stages to match future growth in water demand. Once any of the current CFRD options are adopted, there is a spillway cutting that will need to be raised significantly in the event of further development. This is not a particularly attractive situation, and it brings into question the appropriateness of any of these options if progressive development is seriously considered. While there is little constraint to staged raising of the CFRD embankment, the cutting of spillways below any "final" level should really be avoided if at all possible.

It may be theoretically possible to achieve this objective by providing excess freeboard that can be utilised to route extreme floods by storage, but this concept will need to be carefully analysed to ensure that safety standards are met. I do not envisage that regulatory authorities will be amenable to an argument that relies upon adopting a reduced flood capacity standard based upon a limited exposure period before "final" development is undertaken, so any interim design will need to meet full flood handling standards.

Quarrying for rockfill could still occur in the "final" spillway cutting, but the sill would remain well above FSL until the final stage of development. Conceptually an additional tunnel may satisfy the hydraulic objectives, but the costing of such works would need careful consideration, as any investment made in such features would presumably need to be written off in terms of the "final" development

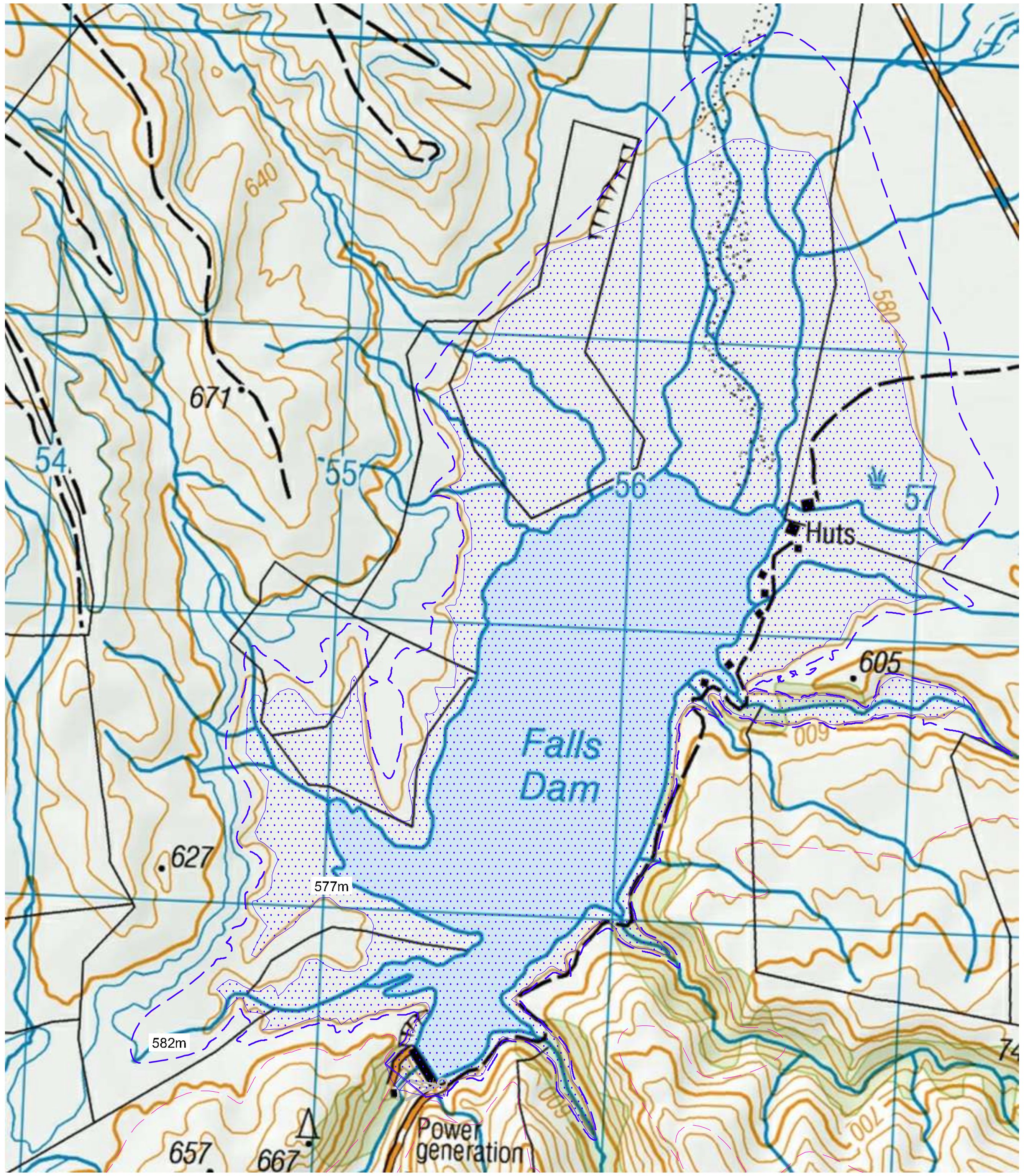
4.10 Key CFRD development challenges

Several design and construction factors condense out from the preliminary discussion presented above.

- Develop spillway hydraulic and geotechnical design to suit rock mass quality, especially where the morning glory spillway is to be retained in a raised form in the path of the auxiliary spillway flow.
- Develop the offtake works general arrangement with a view to determining the specific information needed to input to the feasibility study.
- Develop construction site layouts such that practical access roading and working areas can be established.
- Extent of processing of rock to meet fill requirements in the context of a bulk spillway cut source.
- Stiffness differentials, original vs new embankment fills
- Deformed shape of existing membrane due to accumulated settlement over the life of the embankment
- Repair technique for the degraded membrane joints and exposed concrete, including the existing spillway lining for the lowest option, especially given access constraints arising from the in-service reservoir.
- Saddle dam foundation conditions, especially the depth of stripping required.
- Influence of increased reservoir elevation on the integrity of existing works such as the tunnel bulkhead and tunnel lining subjected to water pressure loading.

577m PLAN

SCALE 1 : 1000 (At A1 size)
SCALE 1 : 2000 (At A3 size)



PLAN

SCALE 1 : 10,000 (At A1 size)
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Figure 41

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Sheet	
ENGINEERING PREFEASIBILITY STUDY RCC. 577m - OPTION	

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Project No.	Scale	Drawing No.	Sheet. No.	Revision
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PLAN
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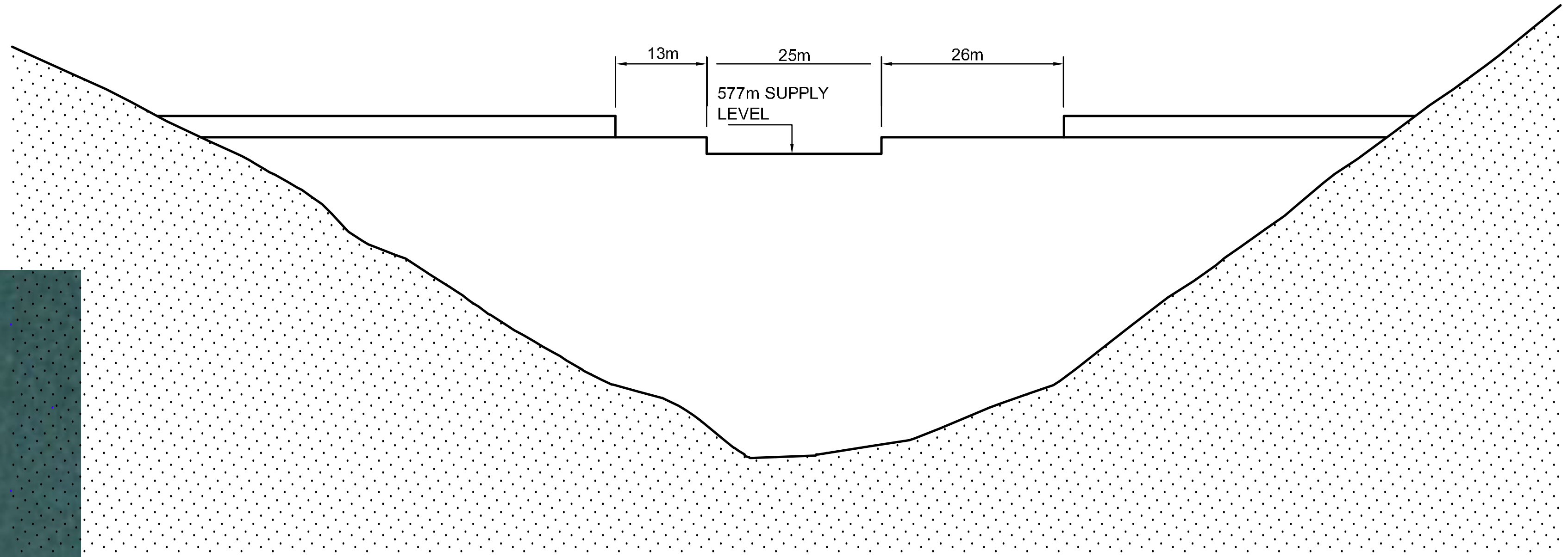

ELEVATION LOOKING DOWNSTREAM
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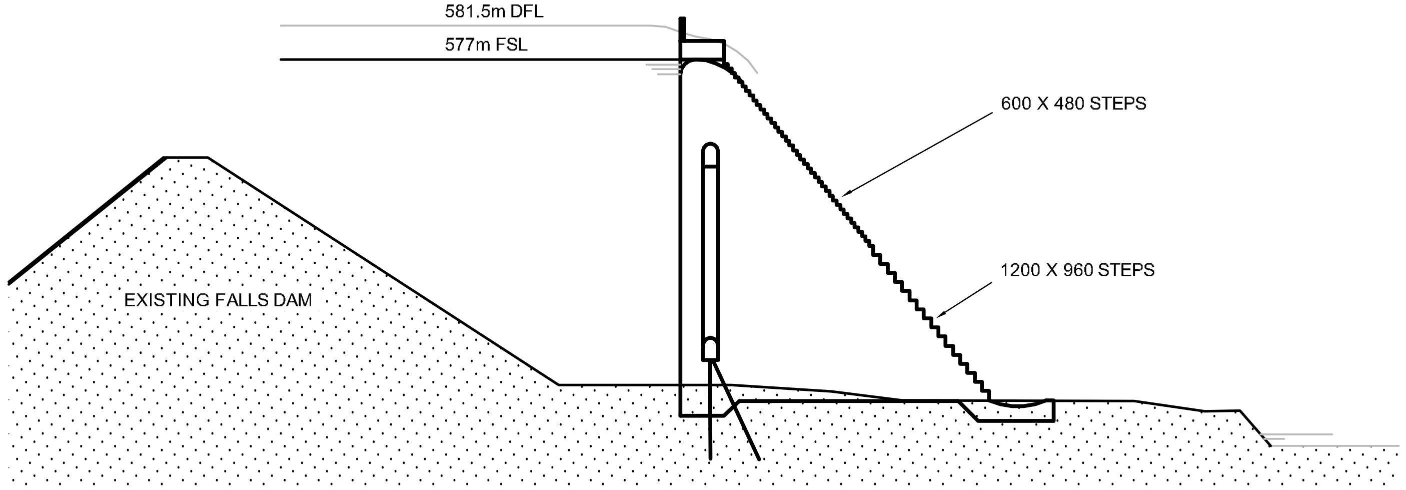

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Figure 42

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Sheet
ENGINEERING PREFEASIBILITY STUDY
RCC. 577m - OPTION

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588m PLAN
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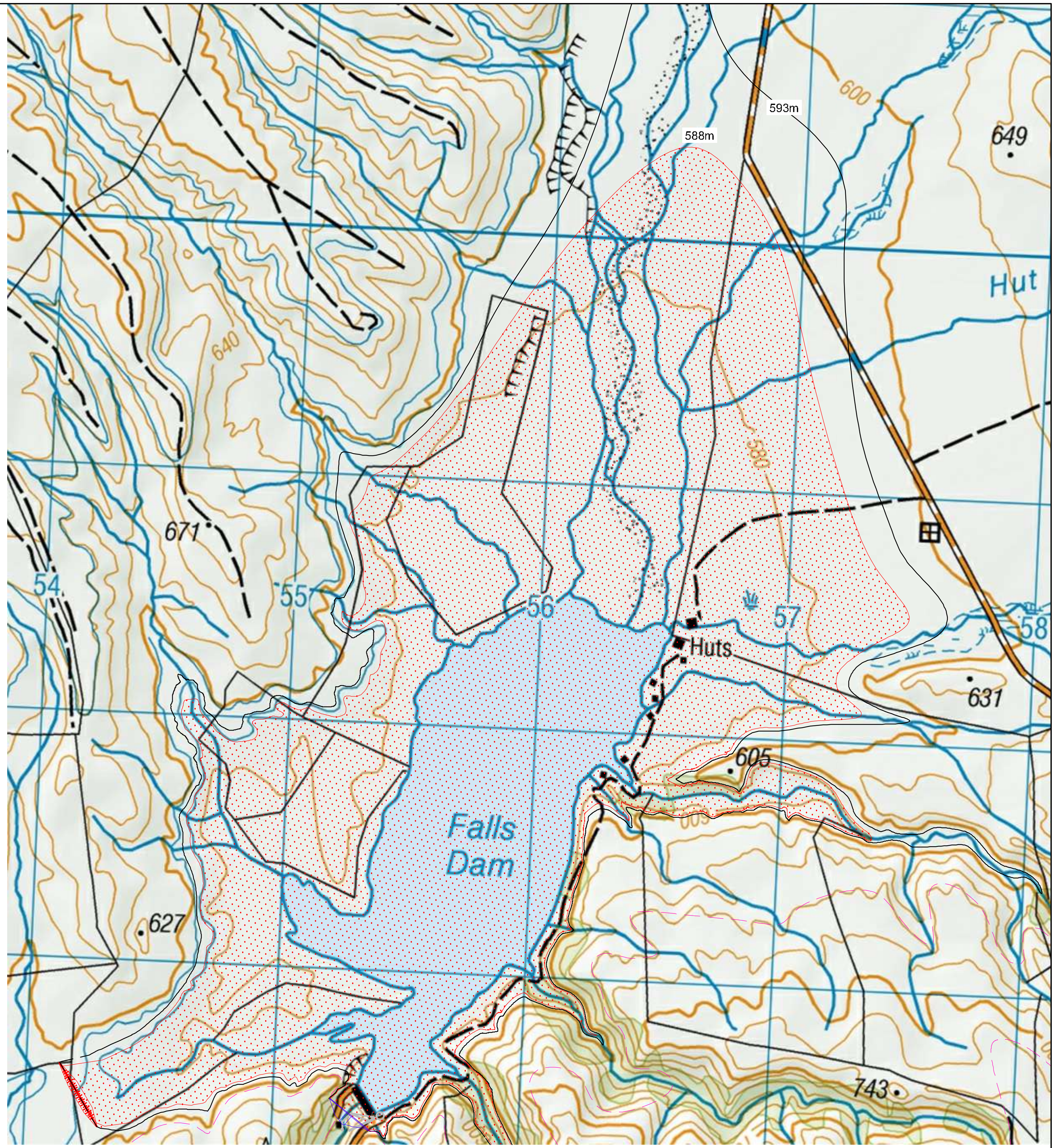
SCALE 1 : 2000 (At A3 size)



577m PLAN
SCALE 1 : 10,000 (At A1 size)
SCALE 1 : 20,000 (At A3 size)

Figure 43

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Water Strategy Study - Stage 3
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Sheet: ENGINEERING PREFEASIBILITY STUDY
RCC. 588m - OPTION

Project No.	Scale	Drawing No.	Sheet. No.	Revision
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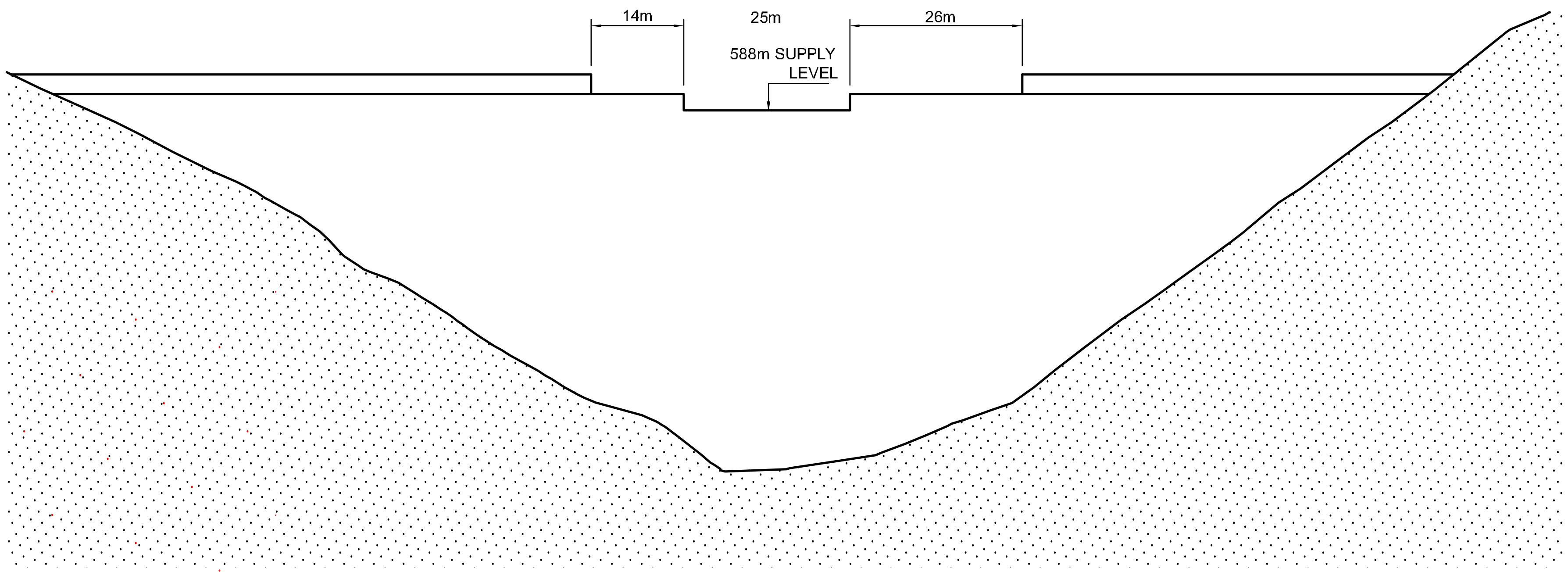

ELEVATION LOOKING DOWNSTREAM
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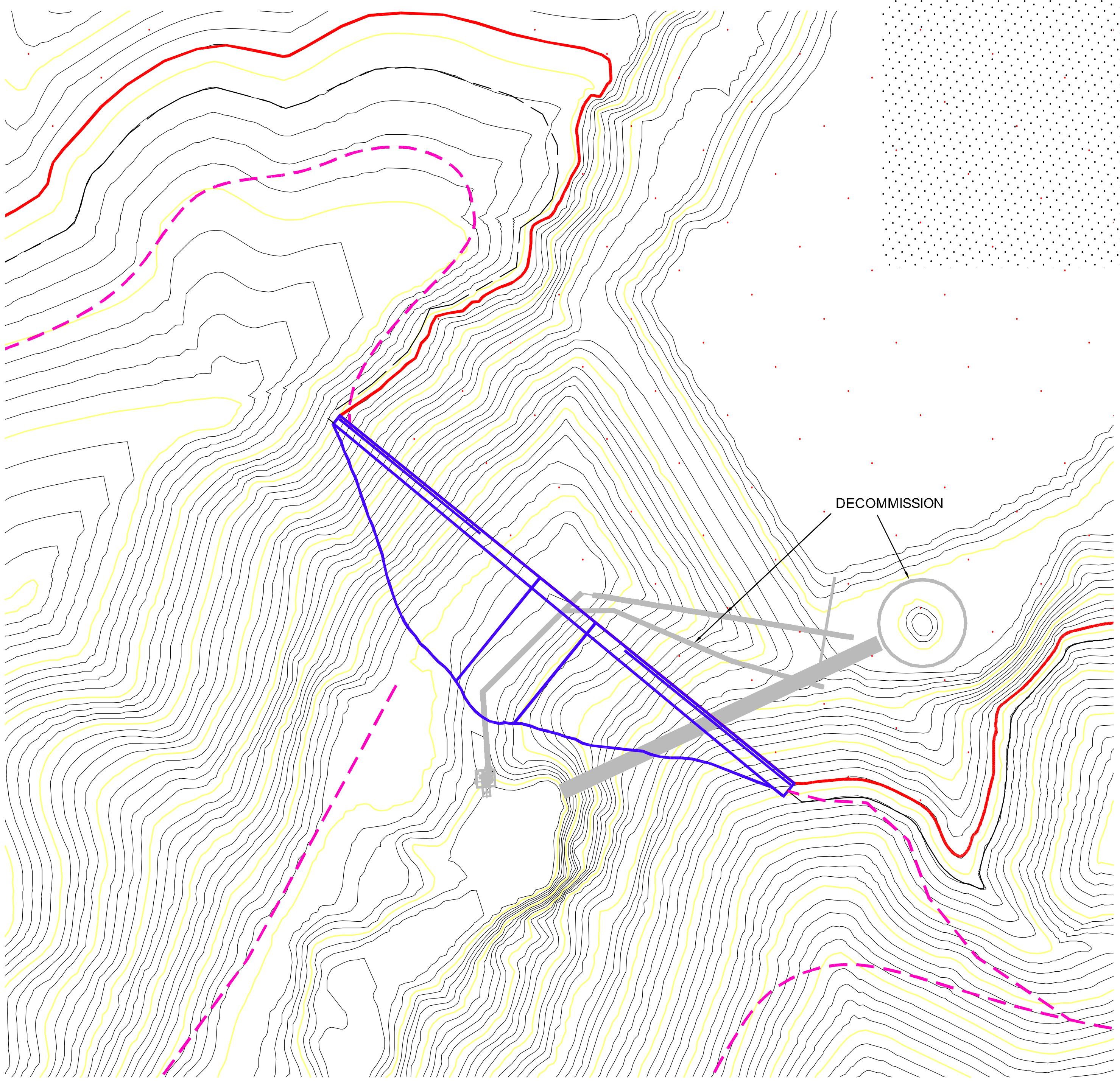

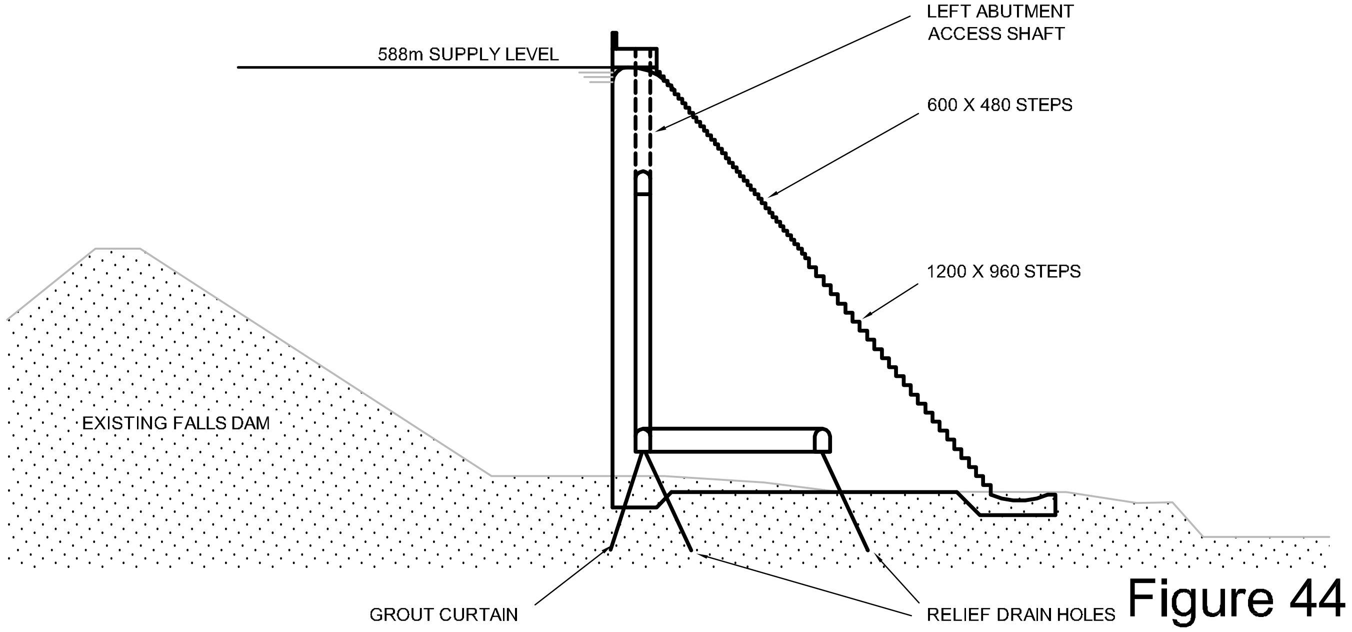

Figure 44

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PRE-FEASIBILITY

PLAN
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FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
RCC. 588m - OPTION

Project No.	Scale	Drawing No.	Sheet. No.	Revision
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5 Roller Compacted Concrete Dam Options

5.1 Spillway and Freeboard

As discussed in the previous section on CFRD options, the morning glory spillway will be decommissioned for both the RL 577 FSL RCC and RL 588 FSL RCC options being considered in this study. The existing spillway will only need to remain in service until the RCC dam reaches a level such that uncontrolled flood breach is not possible. A key advantage of the RCC concept is that over spilling can safely occur with an incomplete dam, and the construction diversion demands apply to a lower level than with an embankment dam, where temporary flood handling must be allowed up to near full embankment height.

The argument previously presented against the adoption of gated spillways at this facility applies also to these RCC options.

It is common practice to incorporate a service spillway and an auxiliary spillway into RCC overspill dams to control the discharge of water into the receiving channel most favourably, while not generating an excessive flood surcharge. This is the approach we have adopted for inclusion in this study. The service spillway can incorporate a smooth curved ogee crest profile with higher efficiency (Cd), and the auxiliary spillway can operate as a broad crested weir with lower efficiency (Cd). Hydraulic control is achieved at the crest as critical depth develops in the normal manner, but RCC construction lends itself to the use of a stepped face where substantial energy dissipation can be achieved before the flow returns to the receiving river channel. This factor is very relevant to this site where the RCC toe terminates near the original falls, and tailwater conditions may be inadequate to achieve the desired stable transition to subcritical flow.

For the target design discharge capacity of 700 m3/s, we have sized the RCC spillway with a 25m central service spillway section, and lateral auxiliary spillways totalling a further 40m crest length, for a total combined length of 65m. The preliminary rating for this arrangement is shown in the figure below, where the design surcharge is some 4.5m above FSL.

The specific discharge for this arrangement is well within normal guideline values for spillways of this type, but detailed analysis may indicate that a longer auxiliary crest may be beneficial.

We have adopted provisional step heights of 600mm and 1200mm in our concept designs, but this aspect will also be subject to detailed analysis in due course. This has little effect on scoping and cost estimation at this early stage.

We have also allowed for a structural concrete apron at the base of the service spillway, although the need for and scope of this feature will be subject to detailed assessment of the rock mass characteristics in this area.

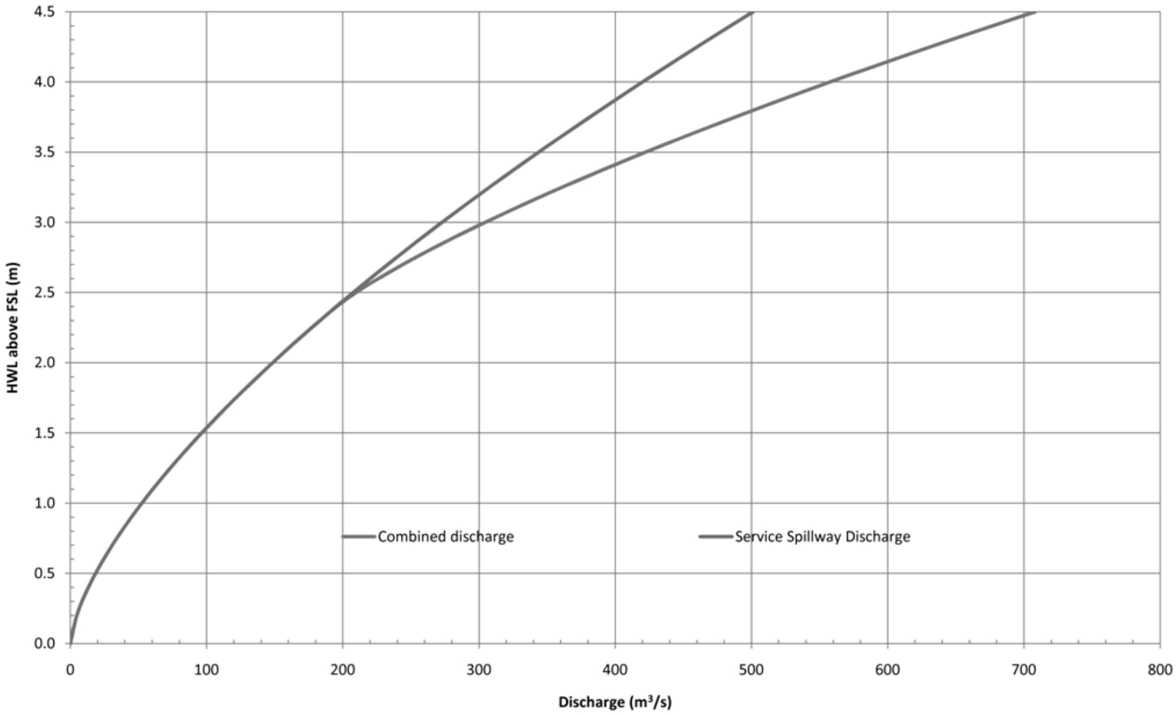


Figure 45 RCC Spillway rating curve

5.2 Dam Cross Sections

The RCC cross sections shown in the above figures are based on a vertical upstream face and an effective 0.8H:1V stepped downstream face. These proportions are typical for dams of this size and type, but specific design will be required to finalise these dimensions as appropriate under all load cases.

As RCC is relatively low strength concrete, the outer "skin" of the dam is shown formed from conventional concrete with a combination of placed in situ and precast elements. This outer skin provides enhanced erosion and freeze thaw resistance, and allows for incorporation of water stops at contraction joints created in the dam at defined points to relieve hydration and thermal shrinkage.

Conventional concrete is also used in the crest and wave wall elements, where conventional formwork is used.

As the stability of gravity dams is influenced by the presence of water pressure on the base, we have allowed for the forming of galleries within the dam to provide access for drilled drains to relieve any seepage pressure. These galleries can also be utilised for access to any internal offtake pipework control valves, and to this end a vertical formed shaft with crest crane access has been included.

5.3 Plan Layouts and Footprint

The plan layouts presented show the dam footprints can be readily accommodated within the available area between the existing dam and the original falls.

However, given the discharge path of the new spillway, it is clear that the existing power house location will not remain tenable. This is not really a major factor as any new hydropower scheme suited to these development options would require changed capacity supply works and plant, such that the existing power house would be inadequately sized.

The RL588 FSL option leads to the need to construct a retaining dam at the Shamrock Gully saddle, some 1.6km from the main dam site as previously identified for the similar height CFRD option. The possible saddle dam concept previously shown is also directly relevant to the 588 FSL RCC option.

5.4 Foundation Treatment

Foundation treatment for the dam will entail minor stripping of the footprint, and more detailed preparation around the heel trench and potentially at the spillway toe.

Injection grouting at the heel trench, is envisaged in a similar manner to the CFRD options, either on the exposed foundation or possibly from within the galleries, although this is likely to be more expensive. Although the original construction experience was that grout takes were generally very low, there may be increased foundation permeability experienced with increasing elevation, reflecting the degree of relaxation of the rock mass due to the progressive river down cutting process. As for the CRD options this aspect should be specifically investigated by subsurface downstage packer testing at selected locations.

Otherwise little preparation effort is envisaged as being required over the remaining footprint. Seepage from the original now isolated construction drain under the embankment, and subsurface drainage from the access adit has been directed to a new collector pipeline and directed to the V notch measuring weir at the power house. These features along with the existing penstock pipelines will need to be examined in detail for sealing or removal.

Permanent foundation drainage works are envisaged to be required at the dam site to relieve any uplift pressures that may develop. These will take the form of drilled drain holes discharging to the internal galleries and in turn to downstream. As for the CRD options, the increased water pressures arising from the reservoir may lead to grouting and/or relief measures being needed at features such as underground concrete linings etc. This aspect will need to be specifically investigated, probably through the installation of groundwater instrumentation (piezometers) to inform detailed design.

5.5 Source of RCC Aggregates, Site Working Areas and RCC Mix Design

The local argillite rock is expected to produce an aggregate product that is very suitable for both RCC and conventional concrete production. Normal investigation of mineral properties for concrete production should be undertaken to confirm this opinion, but the behaviour of the exposed rockfill in the existing dam is certainly consistent with a durable and stable material. The rock is very hard, so wear on handling and crushing equipment can be expected to be at the upper end of the scale of local materials. The grading of aggregate for RCC production is very similar to that used in premium roading basecourse [Mulvihill & Walsh 2001], with a top size of 40mm achieved by crushing. The ability to manufacture sand fractions will need to be established, and it may prove to be necessary to import some quartz sand for blending.

The location of the aggregate quarry will need to be selected to suit the optimised handling of the whole production chain, and this is likely to place it as close to the RCC production plant as possible. There is little working area available at the dam site, so the plant may need to be set up down the gorge a little where flat working area is available. The fresh no-slump RCC is readily transported in on or off road aggregate trucks, and the conventional concrete would be moved by agitators if the batching plant cannot be located adjacent to the dam site and a conveyor system. The in place costs will be strongly influenced by this handling efficiency and plant productivity, so this aspect should receive careful attention during the design process. A potential complicating factor is the possible use of aggregate won from the existing dam embankment once the new RCC is completed up to a suitable level. This option may provide some cost advantages given the effective partial processing that has already been undertaken in creating this substantial “stockpile”.

Volumes of material required are included in net solid volume terms in the Section 6.5 and in Appendix A.

Another factor in the production of RCC is the source of cementitious material. It is common practice elsewhere to substitute some Portland cement with flyash or other suitable products, but there are no local sources of such material and the only other local RCC dam was constructed using cement alone to achieve a 90 day compressive strength of 15MPa [Mulvihill & Walsh 2001]. This matter will benefit from further investigation, as the heat of hydration of a cement only product may become problematic on a project of this scale, and potential cost savings may not be realised if substitute products are not identified.

5.6 Offtake works and Hydropower Scheme

Various layout options for the offtake works associated with RCC dam options were considered during this study, and the simple provision of a conduit embedded in the dam was found to be the most cost efficient approach.

A key factor in this arrangement is the coupling to the reservoir at low levels below the current RL565 embankment crest level. Lowering of the existing dam crest will be required to allow drawdown below this level, and this factor ties in with the opportunity to borrow aggregate for RCC production from the dam.

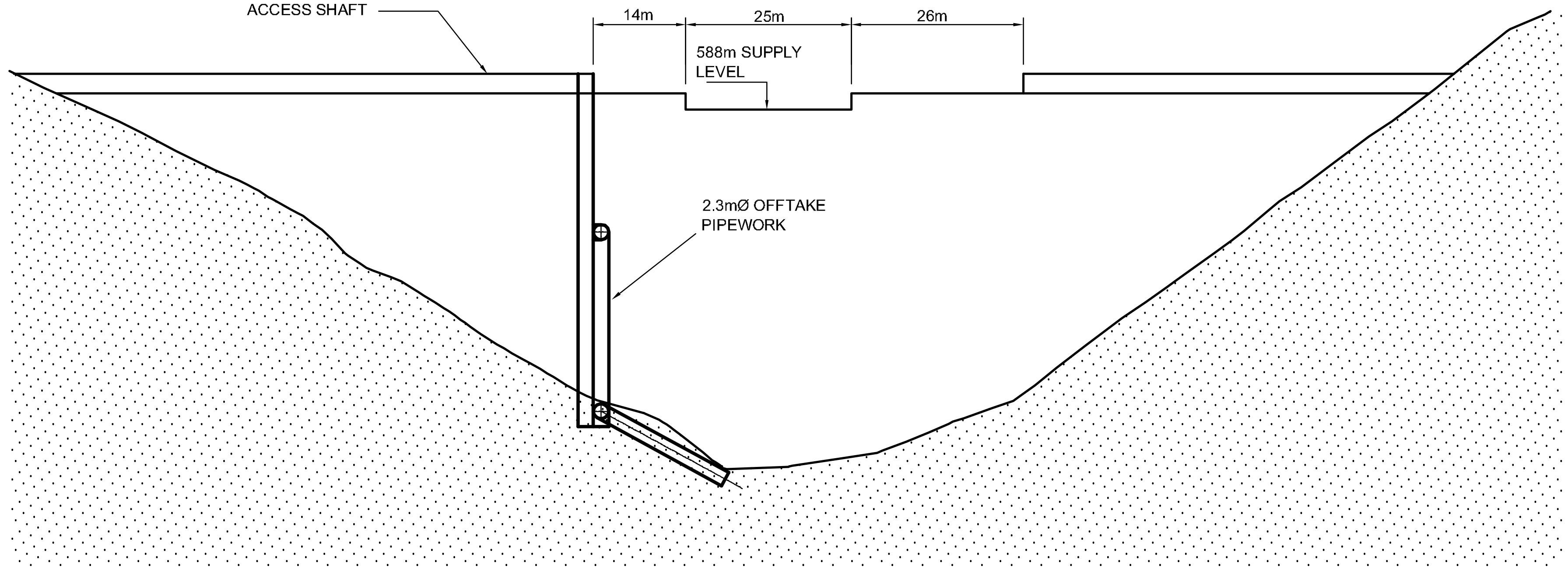
The existing spillway tunnel will become decommissioned and isolated from the reservoir for these options, where a concrete plug at the throat of the spillway shaft has been assumed to be used. The increased offtake capacity associated with these higher FSL options effectively renders the existing hydropower scheme obsolete in its present configuration, although some of the plant may be able to be deployed in a future scheme; probably elsewhere.

The adopted concept is shown on the figure below for the RL588 FSL option, although the layout will be similar for the RL 577 FSL case, just scaled down accordingly.

The concept is based upon seeking to provide access to pipelines and valve gear wherever possible for maintenance purposes. To this end, a two level dry valve chamber has been included within the dam, with the intake screen located on the dam face.

In the conceptual layouts for the RL577 and 588 FSL RCC schemes we have allowed for a new power house to be constructed on the true left river bank around the point from its present location, away from the direct path of the spillway flow above the original falls to gain access to the lowest TWL to achieve maximum potential generation, as at present. The penstock will be laid in a rock trench backfilled with concrete within the spillway zone.

We note that the hydropower implications of these RCC development options are to be the subject of a separate specific study, and this report has only considered the conceptual factors insofar as they are needed to define a demarcation point for the supply works that are to be incorporated within the RCC dam footprint. These supply works upstream of the demarcation point are included in the scope of the preliminary option costings, whereas any investment in the hydropower scheme downstream of this demarcation point is excluded; to be considered on its individual commercial merits. In reality the integration of hydropower and irrigation objectives is more complex than this oversimplification infers, as the use of the water resource as represented by the scheme operating rules will require careful definition in any further feasibility studies.



ELEVATION LOOKING DOWNSTREAM
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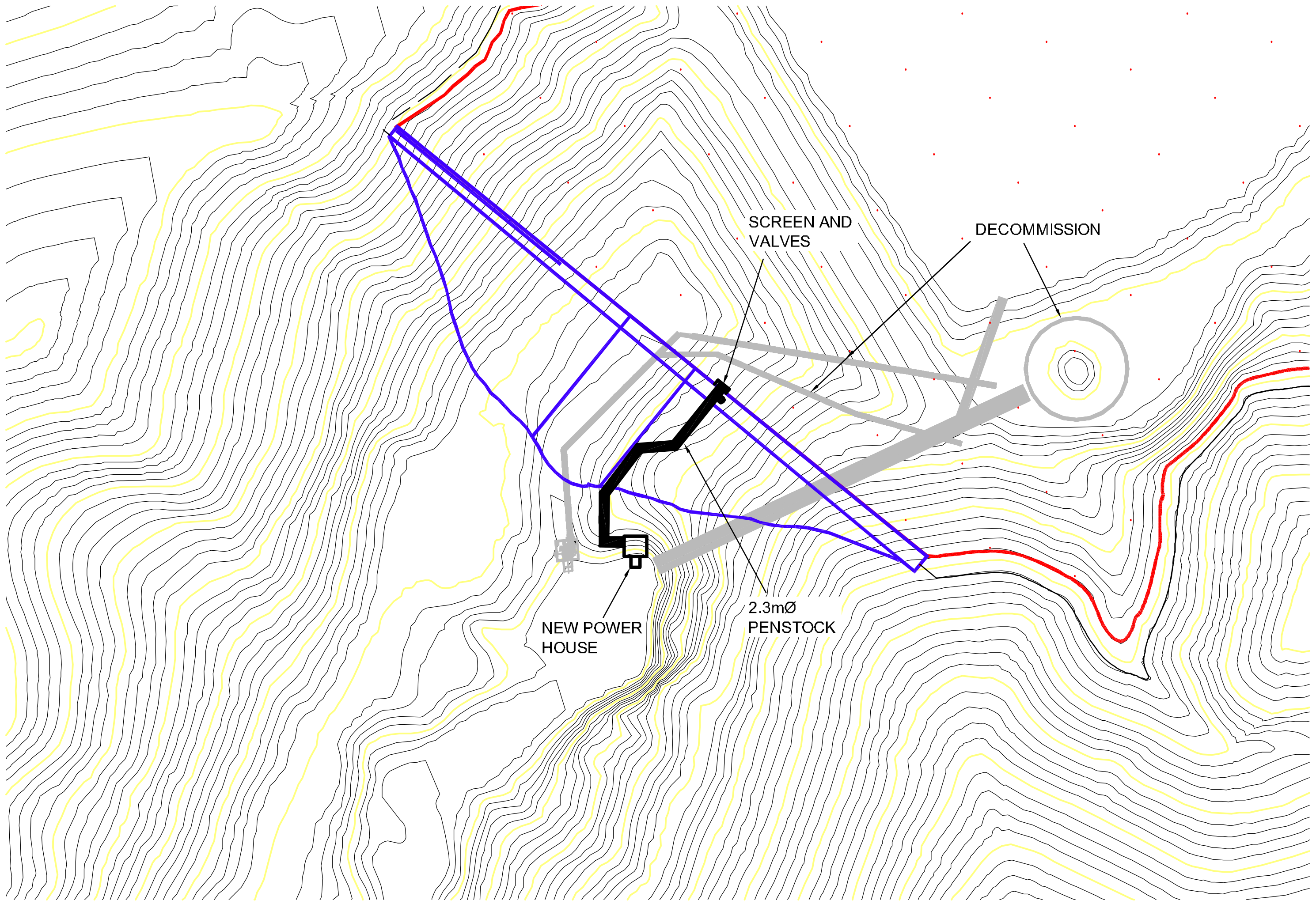

PLAN
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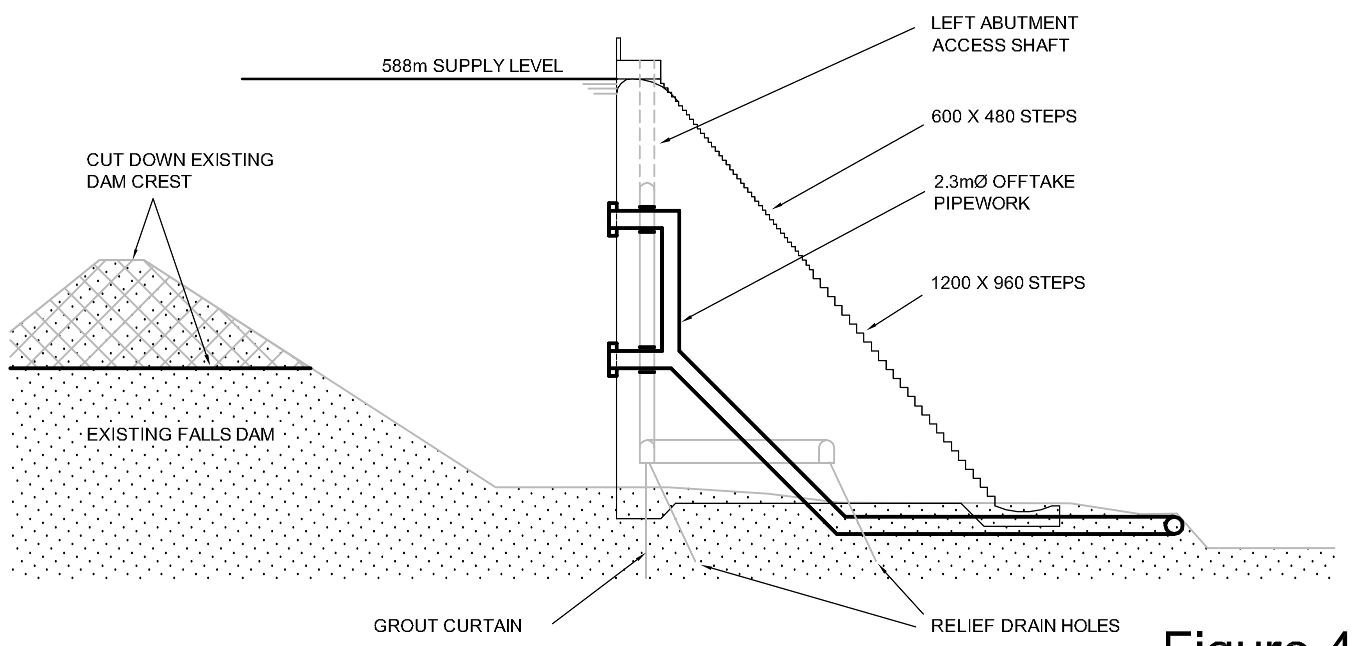

SECTION
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Figure 46
PRE-FEASIBILITY

Revision	Amendment	Approved	Revision Date

OPUS
Dunedin Office
Private Bag 1913
Dunedin 9016, New Zealand
+ 64 3 471 5500

Drawn	Designed	Approved	Revision Date
D Wilson	I G Walsh		30-8-2012

Project No.	Scale
6-cwi04.13 / 25dd	As shown

Project
Manuherikia River Catchment
Water Strategy Study - Stage 3
FALLS DAM REDEVELOPMENT

Sheet
ENGINEERING PREFEASIBILITY STUDY
RCC. HYDROELECTRIC POWER OPTION

Drawing No.	Sheet No.	Revision
7 / 1071 / 1 / 1704	15	

1:1 @ A1 1:2 @ A3 0 10 20 30 40 50 60 70 80 90 100 mm

Original Sheet Size A1 [841x594] Plot Date 04/09/12 @ 11:50 g:\miscellaneous clients\6cwi04.13 aqualinc research - manuherikia valley irrigation study\dwgs\section montage\7_1071_1_1704_15 rcc power option.dwg - 15

5.7 Permanent Access Track

Construction access around the immediate site to facilitate construction activity is included in the scoping of options and preliminary cost estimating.

We have also not allowed for permanent access across the dam crest in these options as was previously allowed for in the +5m CFRD raising option.

However, as for the CFRD options already outlined, land and property considerations are not included in the scope of this engineering study, and access requirements for land owners, and possibly the public, are not included in the scope of works for cost estimation at this stage.

Notwithstanding this exclusion, possible layouts for scoping future feasibility assessments along with land and property considerations are illustrated conceptually on Figure 31 above showing the various raised reservoir extents, some of which will not be required as the low level track (6km.) may be replaced with the one at a higher level. There is potentially a track length of 20.5km required, and up to 20 culverts, two of which may be bridges. The scope of this work is not insignificant to the project, and the possible layouts should be examined in some detail as part of land and property considerations.

5.8 Potential for Progressive Development

The RCC concept presents the most flexibility for progressive development. It would be quite practical to develop the dam to an intermediate elevation and raise this at a future date. The dam could would be extended on the downstream face, so even embedded conduits could be duplicated at the later time, but it may be advantageous to incorporate the ultimate capacity primary conduit in the foundation as part of the initial development for the moderate marginal cost that could be incurred.

There are design issues to be considered in terms of effectively connecting the old and new RCC into a monolithic whole as required for gravity dam stability, but these can be worked through with appropriate detailing, including the possibility of deploying tendons to enhance the connection.

While it would not be a dam safety concern, reliance upon the RCC spillway to discharge floods during a future construction period could impact significantly upon productivity and costs. It may be beneficial to include some future option to recommission the existing morning glory spillway temporarily during this latter construction period, before finally decommissioning the facility permanently. The means of achieving this in a manner that would not compromise short to medium term safety will require careful examination, but it may be possible if the temporary decommissioning method is well thought through. Some form of removable plug that can be supported without placing unacceptable stresses on the shaft lining and surrounding rock, and sealed in a manner that can be practically removed is not too difficult to conceive. The real challenge will be in the detail.

Alternatively the construction phase disruption risk may be adequately mitigated by simply incorporating an over capacity offtake conduit in the dam that would enable adequate control of the reservoir flood rise in most realistic conditions when seasonal factors are taken into account. Some relatively straightforward hydrological modelling would soon give quantitative clarity to these risks, such that engineering solutions could be developed for incorporation into the initial development at an acceptable investment cost. The key to successfully executing this progressive development will be to develop a clear understanding of the final development when designing the initial stage.

5.9 Key development challenges

Several design and construction factors condense out from the preliminary discussion presented above.

- Develop spillway hydraulic and geotechnical design to suit rock mass quality, especially the transition into the receiving river channel
- Develop the offtake works general arrangement with a view to determining the specific information needed to input to the feasibility study.
- Develop construction site layouts such that practical access roading and working areas can be established, especially the preferred quarry site(s) and RCC aggregate processing and mixing area(s).
- Investigation of RCC materials able to be economically produced on site.
- Investigation of potential cementitious materials to complement Portland cement.
- Nature and scope of decommissioning work required for the existing dam and tunnel assets, including the lowering of the existing embankment and the long term integrity of the tunnel lining and bulkhead under increased reservoir elevation.
- Saddle dam foundation conditions, especially the depth of stripping required.

6 Preliminary Construction Cost Estimates

6.1 Estimate Compilation Methodology & Purpose

The principal sources of costing information used in preparing the preliminary estimates are:

- Development of realistic design concepts and models from an experienced perspective,
- Reliable quantity take off of primary construction quantities from 3D modelling of the various options,
- Analysis of actual all inclusive contract rates used on the only other RCC dam constructed in NZ c1998-9, (albeit at a much smaller scale than this proposal),
- Review of contractor estimate provided for previous CFRD raising at Falls Dam based upon similar design approach to this proposal but smaller in scale, and
- Analysis of steel pipe in place costs from recent projects using similar sizes.

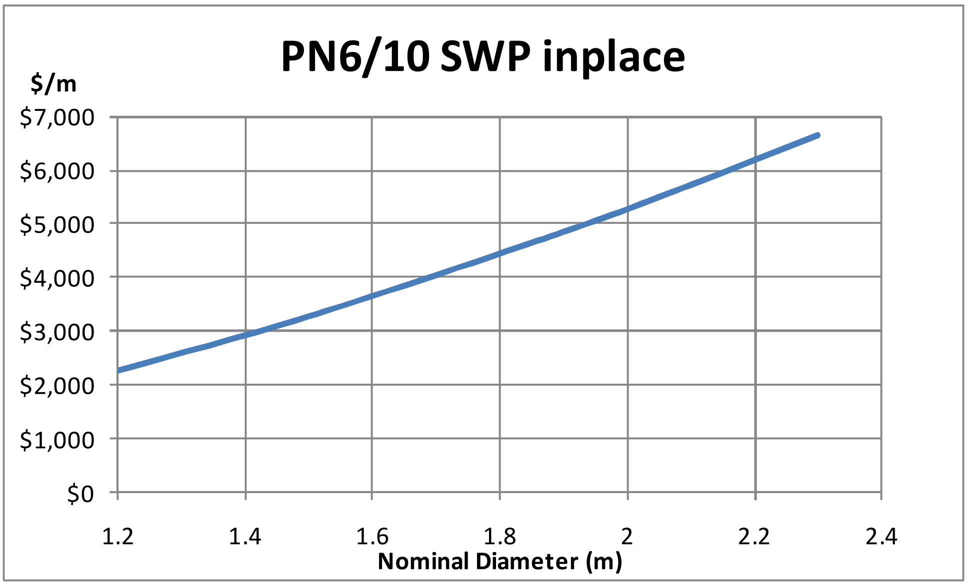

Figure 47 Installed Cost Rates for Welded & Painted Steel Pipework

The costs have not been compiled from first principle construction methodology, productivity, resource demand analysis, and overhead cost allocation. This would be the appropriate cost estimation method to apply to more developed designs in due course. The purpose of this costing exercise is to provide a realistic preliminary assessment of the expected order of costs such that decisions on committing to further development of any given option can be well informed.

6.2 Assumptions and Exclusions

The cost estimates currently exclude everything other than the direct construction investment. Consenting costs, land and property costs, hydropower scheme costs other than the supply conduit are excluded, along with all financing costs and tax liabilities including GST. Allowance is included for engineering design and construction supervision costs, but extensive investigation programme costs are not specifically allowed for.

Sunk costs are not included.

6.3 Cost Risk and Uncertainty

We have sought to present our assessment of the expected construction cost as established from the current state of knowledge reached. Of necessity the costs at this early stage of the project development process must include a degree of conservatism, but the presented figures are not intended to be interpreted as worst case values. Neither are they best case outcomes.

The scope of this commission does not extend to preparing systematic risk adjusted programmes and cost budgets, but we have been cognisant of the risk profile that typically applies to work of this scope and nature when compiling the option estimates. Contingency allowances have been applied at an aggregate level to reflect the swings and roundabouts influences on construction work.

6.4 Construction Cost Escalation Index

In adjusting rates from relevant previous projects and estimates for use in the preliminary current cost estimates (2nd QTR 2012), an escalation correction needs to be applied. The following index trend has been compiled from a weighted combination of cost statistics, to arrive at suitable cost index curve (note the semi-log plot presents a constant escalation rate as a straight sloping line).

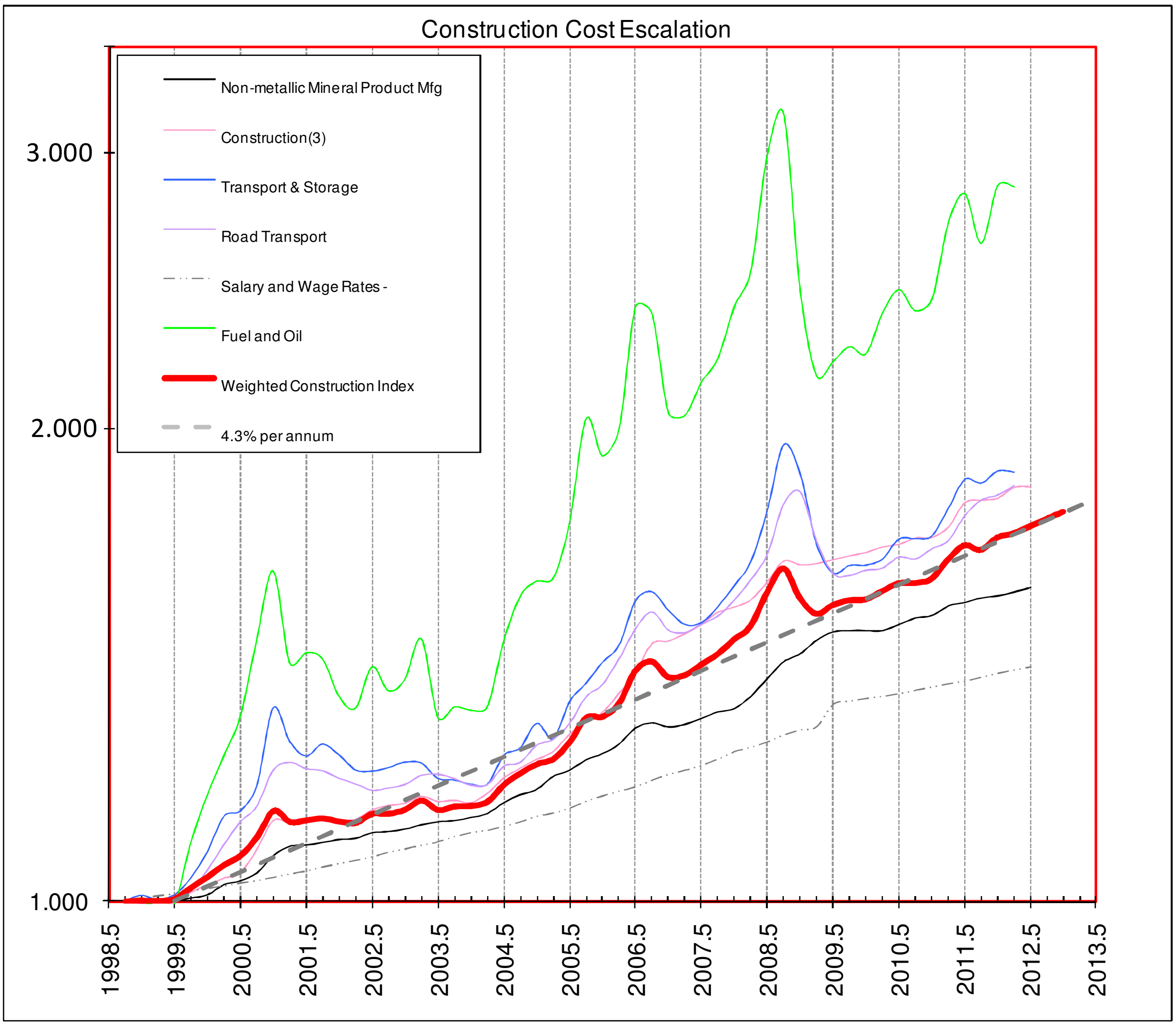

Figure 48 Construction Cost Escalation Index

Fuel and oil cost escalation is running up to 8-9% per annum over this 15 year period, and labour rate escalation is running much lower. Weighted construction index is 4.3% per annum smoothed. For example, over a 10 year interval, the smoothed weighted index shows a 52% increase in $ of the day terms. These values exclude the effects of GST. The chosen weighting values are most suited to the inputs associated with RCC construction, so CFRD construction with its lesser reliance on labour inputs may not be fully adjusted by this methodology.

The presented option costs are in 2nd QTR 2012 terms, and should be regularly updated to reflect escalation until replaced by later estimates. It is clear that actual $ costs at some point in the future when construction is committed will be greater than these figures due to the ongoing effects of escalation. Funding streams for any future construction will obviously need to make allowance for cost escalation beyond the estimate date.

6.5 Preliminary Scope of Work and Quantities

6.5.1 CFRD Options

The three concrete faced rockfill embankment dam development options incorporating the existing dam as described previously have been modelled in the digital terrain model for the site to obtain preliminary quantities for construction cost estimation purposes. The net values tabulated below relate to the dam model imposed on the existing dam profile and ground surface. For scoping purposes, these values need to be adjusted to allow for the effects of site preparation such as foundation excavation.

CFRD Option	Embankment net additional solid fill volume (m^3)	Net additional embankment footprint plan area (m^2)	Spillway cut solid volume (m^2)	New concrete membrane area (m^2)	Net crest length (m)
FSL=588.0	459,000	18,100	455,000	8,790	212
FSL=577.0	212,200	8,600	290,000	4,760	185
FSL=567.5	74,000	3,600	105,000	2,110	173

6.5.2 RCC Options

The two roller compacted concrete dam development options described previously have been modelled in the digital terrain model for the site to obtain preliminary quantities for construction cost estimation purposes. The net values tabulated below relate to the dam model imposed on the existing ground surface. For scoping purposes, these values need to be adjusted to allow for the effects of site preparation such as foundation excavation.

RCC Option	Dam net concrete volume (m^3)	Dam footprint plan area (m^2)	Stepped face net surface area (m^2)	Crest wave wall length (m)
FSL=588.0	134,100	5,850	112,000	195
FSL=577.0	78,600	4,200	81,000	155

6.5.3 "Do Nothing: Base Scenario

The "no storage development scenario" discussed in Section 3.8 arrived at the view that there are significant liabilities associated with ongoing defensible management of the existing facility. There is also a strong case to include a degree of storage enhancement by utilising the "surplus" rock excavation that would be obtained from constructing an auxiliary spillway cutting. While these dam safety management decisions are still to be made, the range of potential cost implications are presented below for consideration of the marginal costs for the defined storage development options covered by this report.

6.6 Expected Current Value Costs of the Selected Development Options

The originating costing schedules for each option considered are included in the appendix, and the summary cost make up is tabulated below with P&G distributed across the items:

			CFRD options Excl Powerhouse & Plant			
Work Description		Development Option	+5m (566.5FSL), MG+fuse plug	+6m 567.5FSL MG+auxil	+15m 577FSL +cutting	+26m 588FSL +cutting & saddle dam
	design peak offtake capacity		4.0	4.0	6.5	11.0
Repairs and Maintenance	membrane		$910,000	$910,000	$910,000	$910,000
Repairs and Maintenance	spillway		$694,000	$694,000	$0	$0
Access Portal Extension			$243,000	$339,000	$0	$0
Access Roading			$569,000	$566,000	$560,000	$560,000
Membrane Plinth			$266,000	$303,000	$592,000	$1,200,000
Spillway Rock Cutting			$3,193,000	$5,367,000	$13,872,000	$16,344,000
Dam Embankment			$1,212,000	$1,936,000	$5,056,000	$10,411,000
Concrete Membrane Extension			$639,000	$827,000	$1,807,000	$3,078,000
Spillway Cutting Crest Works			$1,131,000	$183,000	$377,000	$376,000
Glory Hole Spillway Works			$1,347,000	$1,881,000	$3,738,000	$5,356,000
Additional / Misc Items			$1,281,000	$3,145,000	$8,584,000	$15,865,000
Expected total incl. unscheduled, engineering fees and contingencies			$11,485,000	$16,151,000	$35,496,000	$54,100,000
		$M	**$11.5**	**$16.2**	**$35.5**	**$54.1**
Gross storage			1.86E+07	2.07E+07	4.93E+07	$105,403,000
less existing			1.03E+07	1.03E+07	1.03E+07	1.03E+07
Increase in gross storage			8.26E+06	1.04E+07	3.90E+07	9.51E+07
		$/m3	**$1.39**	**$1.56**	**$0.91**	**$0.57**

			RCC options Excl Powerhouse & Plant			
Work Description		Development Option			+15m 577 FSL overspill	+26m 588 FSL overspill + saddle dam
	peak offtake capacity				6.5	11.0
Repairs and Maintenance	membrane				$0	$0
Repairs and Maintenance	spillway				$0	$0
Seal Access Adit					$38,000	$38,000
Access Roading					$476,000	$476,000
Partial Demolition of Membrane					$12,000	$12,000
Aggregate Production					$8,087,000	$13,084,000
Foundation Preparation & Treatment					$3,532,000	$4,822,000
RCC & Concrete Construction					$29,140,000	$45,378,000
Offtake works @ RCC					$1,363,000	$1,947,000
Glory Hole Spillway Isolation Works					$268,000	$269,000
Additional / Misc Items					$0	$1,935,000
Expected total incl. unscheduled, engineering fees and contingencies					$42,916,000	$67,961,000
		$M			**$42.9**	**$68.0**
Gross storage					4.93E+07	1.05E+08
less existing					1.03E+07	1.03E+07
Increase in gross storage					3.90E+07	9.51E+07
		$/m3			**$1.10**	**$0.71**

The RCC options show a 20% to 25% premium over the equivalent CFRD items, which is consistent with the proportion of existing assets being retained in service in these options.

6.7 Expected Liabilities of the Existing Assets

As discussed in Section 3.8, there are significant potential liabilities arising from owning the existing assets even in the absence of storage development associated with the options considered in this report.

The magnitude of this potential liability and the probability and timing of it arising are subject to many factors which are still open to a number of influences. In this sense we have presented the potential liability as a range of values in similar term to the above tabulation of costs. These costs can be deducted from the costs tabulated in Section 6.6 above to identify the marginal investment proposition, as the tabulated development costs for the options include for addressing these existing potential liabilities.

This liability is estimated to lie in the range **$7.2M to $11.5M**, where the lower limit involves dumping the auxiliary spillway excavation as cut to waste, and the upper limit is simply the +5m CFRD raising option presented in section 6.6. My previously expressed (and current) view is that the marginal cost involved by committing the additional $4.3M is an attractive investment to gain some 8.26Mm^3 additional storage capacity. This corresponds to a marginal investment cost of some $0.87/m^3 .

In terms of adjusting the cost estimates of the new development options tabulated in Section 6.6 back to marginal investment requirements, the $7.2M lower limit on the potential liability could reasonably be deducted. We stress that the assumptions and opinions outlined in Section 3.8 should be referred to when making such a deduction, as there may be other view points on the existing liabilities to be taken into account.

6.8 Influence of Progressive Development on Costs

As discussed above, only the RCC options present a practical upgrade pathway without significant changes to the option definitions to accommodate an alternative means of handling extreme floods.

Assessment of the additional costs incurred by building in upgrade capability to the RCC dam requires a clear definition of the actual upgrade scenario(s), but as a generalisation it appears that such cost penalty is not expected to be excessive.

Further development of the matter requires analysis of the flood handling risk and treatment options, and further definition of the specific upgrade scenarios.

References

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Opus 2003-06 "Falls Dam Raising" Unpublished reports for Falls Dam Company addressing preliminary water balance and dam engineering aspects of increased storage up to 6m above existing FSL and retaining 4 cumec peak discharge capacity to enhance to security of supply to existing users. Incorporates results of synthesised inflow record.

Opus 2005 "Falls Dam – Asset management" unpublished report for Falls Dam Company including deferred maintenance and safety compliance obligations arising from the requirements of the Building Act 2004.

Opus 2006 "Falls Dam Replacement Cost" unpublished report for Falls Dam Company provided for asset insurance purposes, including scoping of RCC alternatives to CFRD asset.

Opus 2007a "Falls Dam Raising - Implications on Power Generation" Draft report for Pioneer Generation. Examines effects on output of existing generation assets of raising Falls Dam up to 5m.

Opus 2007b "Falls Dam Probable Maximum Flood" Draft report for Falls Dam Company; not peer reviewed or finalised.

Ellis D 2009 "The Small Dams of Central Otago" Includes a summary of the construction history of Falls Dam.

Opus 2010 "Falls Dam Redevelopment - Overview of Potential New Dam Concepts" prepared for Falls Dam Company, covering supply reliability and construction costs for reservoir raising up to 12m above existing FSL, and peak offtake flows up to 6 cumecs.

Walsh I G, 2010 "Some NZ Examples of Enhancing Small Hydropower Assets", Paper submitted at Renewable Energy Asia Conference - Singapore Nov 2010, including the Falls Dam retrofitted mini hydropower project and its syphonic over dam supply penstock system.

Opus 2011 "Falls Dam Raising – Scoping of Raising Existing by 8m" communication to Falls Dam Company, covering design implications and scope of work for extending the +5m option to +8mg up to 12m above

Raineffects 2012 "Falls Dam Inflows", Updating of the synthesised long term daily inflow record for Manuherikia at Falls Dam prepared for Opus / Aqualinc.

Aqualinc 2012 "Manuherikia Valley: Detailed Hydrology". Study of water resources available for irrigation development, incorporating results from the updated synthesised inflow record for Falls dam.

Appendix - Preliminary Cost Estimate Schedules

Falls Dam CFRD Raising Concept
Preliminary Construction Cost Estimate
FSL=567.5m

OPUS International Consultants
Prepared by Ian Walsh
Date 30-Aug-12
Status ROC Rev5
File 6CWI04.13

Assumptions
Repairs to existing assets not directly studied
Auxillary spillway unlined (to tolerate some damage in
Exclusions
Land & Property Issues
RMA Consent Costs
Pioneer Generation Ltd Asset Impacts
All volumes are solid in place

Item	Description	Unit	Quantity	Original Nov04 rates Rate	Amount Nov04 $	Subtotals Nov-04 $	Subtotals Jun-12 $	P&G redistributed and amounts dajusted to EV	$
								P&G	
1	**Preliminary & General**		1	$520,000	$520,000	$520,000	$737,447	6.0%	$0
2	**Repairs and Maintenance**								
2.01	Membrane perimetric joint above WL	m	201.41	$300	$60,423	$397,172	$563,257		$901,207
2.02	Membrane perimetric joint below WL	m	0	-					
2.03	Membrane panel joint above WL	m	683.22	$250	$170,805				
2.04	Membrane panel joint below WL	m	0	-					
2.05	Membrane concrete surface repair	m2	760.05	$85	$64,604				
2.06	Membrane concrete surface sealing	m2	5067	$20	$101,340				
2.07	Glory Hole concrete lining CJ repair	m	150	$510	$76,500	$303,500	$430,414		$688,659
2.08	Glory Hole concrete lining surface repair	m2	260	$250	$65,000				
2.09	Tunnel concrete lining CJ repair	m	100	$510	$51,000				
2.10	Tunnel concrete lining surface repair	m2	444	$250	$111,000				
3	**Access Portal Extension**					$149,610	$212,173		$339,475
3.1	Reinf concrete footing & invert slab	m3	17.55	$1,000	$17,550				
3.2	Connection to existing portal	LS	1	$3,000	$3,000				
3.3	CSP multiplate 2.5m dia arch	t	6.670026	$18,000	$120,060				
3.4	Mitre bends - multi plate specials	ea	2	$3,000	$6,000				
3.5	Reinf concrete portal stiffener ring & security gate	LS	1	$3,000	$3,000				
4	**Access Roading**					$249,293	$353,538		$565,659
4.1	TL weathered rock cut to TR turning platform formation fill	m3	4000	$14.00	$56,000				
4.2	TL fresh rock cut to dam toe fill	m3	3700	$22.00	$81,400				
4.3	TL pavement shaping and running course	m2	800	$4.50	$3,600				
4.4	TR rock cut to lower platform fill	m3	4500	$22.00	$99,000				
4.5	TR pavement shaping and running course	m2	1200	$4.50	$5,400				
4.6	Crest pavement shaping and running course	m2	865	$4.50	$3,893				
5	**Membrane Plinth**					$133,685	$189,587		$303,338
5.1	Loose rock excavation	m3	68.82042	$120	$8,258				
5.2	Insitu rock excavation	m3	45.88028	$160	$7,341				
5.3	Sealing at penstock cutting	PS	1	$10,000	$10,000				
5.4	Install passive 3.6m grouted rock anchors	ea	45	$650	$29,250				
5.5	Demolish existing plinth/membrane terminations	ea	2	$2,000	$4,000				
5.6	Foundation grouting including drilling	t	11.47007	$4,500	$51,615				
5.7	Concrete plinth (reinforced)	m	46	$250	$11,500				
5.8	Perimetric joint	m	46	$120	$5,520				
5.9	Demolish existing coping wall & hand rail	m	155	$40	$6,200				
6	**Auxiliary Spillway Cutting**					$2,365,500	$3,354,674		$5,367,457
6.1	Site stripping etc	LS	1	$11,000	$11,000				
6.2	Rock cutting to waste	m3	41,000	$33	$1,353,000				
6.3	Rock cutting to embankment fill	m3	64,000	$12	$768,000				
6.4	Dental concrete including local undercutting (unreinforced)	m3	200	$500	$100,000				
6.5	110mm Shotcrete including mesh & grouted dowel anchors	m2	500	$120	$60,000				
6.6	3.6m grouted rock anchors including drilling - to secure blocks	ea	30	$650	$19,500				
6.7	Consolidation grouting including drilling - to lock in invert blocks	t	12	$4,500	$54,000				
7	**Dam Embankment**					$853,280	$1,210,093		$1,936,142
7.1	Foundation preparation	m2	3960	$1	$3,960				
7.2	Progressive benching of existing face	LS	1	$21,000	$21,000				
7.3	Place and compact selected general (through grizzly if required) rockfill from cuts	m3	66615	$8	$532,920				
7.4	Place and compact selected processed rockfill from cuts in upstream crest zone	m3	7385	$40	$295,400				
8	**Concrete Membrane Extension**					$364,444	$516,842		$826,945

8.1	New rockfill face preparation	m2	2110	$2	$4,220			
8.2	Horizontal panel joints	m	155	$120	$18,600			
8.3	Vertical panel joints	m	115.2	$120	$13,824			
8.4	Slipformed 300mm thick reinforced concrete membrane panels	m2	2110	$140	$295,400			
8.5	New coping wall & handrail	m	162	$200	$32,400			
9	**Auxiliary Spillway Crest Works (not fuse plug concept)**					$80,640	$114,361	$182,977
9.1	base reinf slab	m3	120	$600	$72,000			
9.2	Fuse plug apron reinf slab	m3	14.4	$600	$8,640			
9.3	Abutment reinf walls	m3	0	$800	$0			
9.4	Reinf pier	m3	0	$1,200	$0			
9.5	Bridge deck 15m DHC post tensioned units including transverse tie bars	ea	0	$10,000	$0			
9.6	Bridge side rails, joints etc	LS	0	$10,000	$0			
9.7	Spillway guide wall anchors	ea	0	$800	$0			
9.8	Spillway guide wall reinf concrete	m3	0	$1,000	$0			
9.9	Supply and place zoned fuse plug fill	m3	0	$100	$0			
10	**Glory Hole Spillway Works**					$829,195	$1,175,936	$1,881,491
10.1	Temporary access track	LS	1	$5,000	$5,000			
10.2	Foundation preparation/excavation	LS	1	$3,000	$3,000			
10.3	Reinforced footing including water stops - primary conc	m3	255.4956	$1,200	$306,595			
10.4	Foundation grouting including drilling	t	10	$4,500	$45,000			
10.5	Drainage relief holes	ea	12	$600	$7,200			
10.6	Supply precast 1.7m wide segments - 48 of	m3	202.00	$1,200.00	$242,400.00			
10.7	Reinf plunge slab - 300 thick including dow	m3	60.00	$800.00	$48,000.00			
10.8	Install precast segments including 0.15-0.3	ea	48.00	$1,500.00	$72,000.00			
10.9	Footing secondary reinf concrete	m3	21.00	$800.00	$16,800.00			
10.10	Aeration pipework detail	PS	1.00	$40,000.00	$40,000.00			
10.11	Extend reinf concrete guide vanes - 6 of	m3	54.00	$800.00	$43,200.00			
10.12								
						$1,386,000	$1,965,579	$3,144,915
11	**Additional / Misc Items**							
	(add any further identified items)				$0			
11.1	extra over 6.3 for R2 rock 33%	m3	34650	$40	$1,386,000			
11.2								
11.3					$0			
					Sub Total Net Construction Est	$7,632,319	$10,823,902	
	Unscheduled items	8%			Unscheduled Allowance	$610,585	$865,912	
					Sub Total Base Construction Est	$8,242,904	$11,689,814	
	Engineering Fees	10%			Engineering Costs	$824,290	$1,168,981	
					Sub Total Design & Construction Base Est	$9,067,194	$12,858,795	
	Contingency to Most Likely Outturn	25%			Plus 50%ile Contingency	$2,266,799	$3,214,699	
					Expected Mean Outturn	$11,333,993	$16,073,494	$16,138,265
	Additional Contingency to 80%ile Outturn	15%			Plus 50-80%ile Contingency	$1,700,099	$2,411,024	
					80%ile outturn	$13,034,092	$18,484,518	
					Total Excl GST	$13,034,092	$18,484,518	

Falls Dam CFRD Raising Concept
Preliminary Construction Cost Estimate
FSL=577.0m

OPUS International Consultants
Prepared by Ian Walsh
Date 30-Aug-12
Status ROC Rev5
File 6CWI04.13

Assumptions
Repairs to existing assets not directly studied
Auxillary spillway unlined (to tolerate some damage in
Exclusions
Land & Property Issues
RMA Consent Costs
Pioneer Generation Ltd Asset Impacts
All volumes are solid in place

Item	Description	Unit	Quantity	Original Nov04 rates Rate	Amount Nov04 $	Subtotals Nov-04 $	Subtotals Jun-12 $	P&G redistributed and amounts dajusted to EV	$
1	**Preliminary & General**		1	$1,000,000	$1,000,000	$1,000,000	$1,418,167	P&G 6.1%	$0
2	**Repairs and Maintenance**					$397,172	$563,257		$890,671
2.01	Membrane perimeteric joint above WL	m	201.41	$300	$60,423				
2.02	Membrane perimeteric joint below WL	m	0	-					
2.03	Membrane panel joint above WL	m	683.22	$250	$170,805				
2.04	Membrane panel joint below WL	m	0	-					
2.05	Membrane concrete surface repair	m2	760.05	$85	$64,604				
2.06	Membrane concrete surface sealing	m2	5067	$20	$101,340				
2.07	Glory Hole concrete lining CJ repair	m	0	$510	$0	$0	$0		$0
2.08	Glory Hole concrete lining surface repair	m2	0	$250	$0				
2.09	Tunnel concrete lining CJ repair	m	0	$510	$0				
2.10	Tunnel concrete lining surface repair	m2	0	$250	$0				
3	**Access Portal Extension**					$0	$0		$0
3.1	Reinf concrete footing & invert slab	m3	0	$1,000	$0				
3.2	Connection to existing portal	LS	0	$3,000	$0				
3.3	CSP multiplate 2.5m dia arch	t	0	$18,000	$0				
3.4	Mitre bends - multi plate specials	ea	0	$3,000	$0				
3.5	Reinf concrete portal stiffener ring & security gate	LS	0	$3,000	$0				
4	**Access Roading**					$249,563	$353,921		$559,652
4.1	TL weathered rock cut to TR turning platform formation fill	m3	4000	$14.00	$56,000				
4.2	TL fresh rock cut to dam toe fill	m3	3700	$22.00	$81,400				
4.3	TL pavement shaping and running course	m2	800	$4.50	$3,600				
4.4	TR rock cut to lower platform fill	m3	4500	$22.00	$99,000				
4.5	TR pavement shaping and running course	m2	1200	$4.50	$5,400				
4.6	Crest pavement shaping and running course	m2	925	$4.50	$4,163				
5	**Membrane Plinth**					$263,880	$374,226		$591,759
5.1	Loose rock excavation	m3	147.1224	$120	$17,655				
5.2	Insitu rock excavation	m3	98.0816	$160	$15,693				
5.3	Sealing at penstock cutting	PS	1	$10,000	$10,000				
5.4	Install passive 3.6m grouted rock anchors	ea	98	$650	$63,700				
5.5	Demolish existing plinth/membrane terminations	ea	2	$2,000	$4,000				
5.6	Foundation grouting including drilling	t	24.5204	$4,500	$110,342				
5.7	Concrete plinth (reinforced)	m	98.0816	$250	$24,520				
5.8	Perimetric joint	m	98.0816	$120	$11,770				
5.9	Demolish existing coping wall & hand rail	m	155	$40	$6,200				
6	**Spillway Cutting**					$6,185,696	$8,772,349		$13,871,616
6.1	Site stripping etc	LS	1	$20,000	$20,000				
6.2	Rock cutting to waste	m3	105,652	$33	$3,486,522				
6.3	Rock cutting to embankment fill	m3	184,348	$12	$2,212,174				
6.4	Dental concrete including local undercutting (unreinforced)	m3	400	$500	$200,000				
6.5	110mm Shotcrete including mesh & grouted dowel anchors	m2	1000	$120	$120,000				
6.6	3.6m grouted rock anchors including drilling - to secure blocks	ea	60	$650	$39,000				
6.7	Consolidation grouting including drilling - to lock in invert blocks	t	24	$4,500	$108,000				
7	**Dam Embankment**					$2,254,408	$3,197,127		$5,055,582
7.1	Foundation preparation	m2	8600	$1	$8,600				
7.2	Progressive benching of existing face	LS	1	$21,000	$21,000				

7.3	Place and compact selected general (through grizzly if required) rockfill from cuts	m3	195474.7	$8	$1,563,798			
7.4	Place and compact selected processed rockfill from cuts in upstream crest zone	m3	16525.26	$40	$661,011			
8	**Concrete Membrane Extension**					$805,752	$1,142,690	$1,806,923
8.1	New rockfill face preparation	m2	4721.504	$2	$9,443			
8.2	Horizontal panel joints	m	347	$120	$41,640			
8.3	Vertical panel joints	m	472.1504	$120	$56,658			
8.4	Slipformed 300mm thick reinforced concrete membrane panels	m2	4721.504	$140	$661,011			
8.5	New coping wall & handrail	m	185	$200	$37,000			
9	**Auxiliary Spillway Crest Works (not fuse plug concept)**					$168,000	$238,252	$376,745
9.1	base reinf slab	m3	240	$600	$144,000			
9.2	Fuse plug apron reinf slab	m3	0	$600	$0			
9.3	Abutment reinf walls	m3	30	$800	$24,000			
9.4	Reinf pier	m3	0	$1,200	$0			
9.5	Bridge deck 15m DHC post tensioned units including transverse tie bars	ea	0	$10,000	$0			
9.6	Bridge side rails, joints etc	LS	0	$10,000	$0			
9.7	Spillway guide wall anchors	ea	0	$800	$0			
9.8	Spillway guide wall reinf concrete	m3	0	$1,000	$0			
9.9	Supply and place zoned fuse plug fill	m3	0	$100	$0			
10	**Glory Hole Spillway Works**					$1,667,074	$2,364,190	$3,738,466
10.1	Temporary access track	LS	1	$5,000	$5,000			
10.2	Foundation preparation/excavation	LS	1	$3,000	$3,000			
10.3	Reinforced footing including water stops - primary conc	m3	0	$1,200	$0			
10.4	Foundation grouting including drilling	t	10	$4,500	$45,000			
10.5	Drainage relief holes	ea	12	$600	$7,200			
10.6	concrete spillway plug	m3	115.45	$700.00	$80,817.47			
10.7	exposed 24m high steel pipe 4.4m dia towe	t	65.27	$5,000.00	$326,356.60			
10.8	paint	m2	663.50	$40.00	$26,540.17			
10.9	1.9m supply pipe	m	159.00	$4,840.00	$769,560.00			
10.10	1.6m syphon pipe	m	40.00	$3,640.00	$145,600.00			
10.11	Valves 2.0 @ RL 565	ea	1.00	$240,000.00	$240,000.00			
10.12	Actuators	ea	1.00	$18,000.00	$18,000.00			
11	**Additional / Misc Items**					$3,828,000	$5,428,743	$8,584,410
	(add any further identified items)				$0			
11.1	extra over 6.3 for R2 rock 33%	m3	95700	$40	$3,828,000			
11.2					$0			
11.3					$0			
					Sub Total Net Construction Est	$16,819,544	$23,852,922	
	Unscheduled items	8%			Unscheduled Allowance	$1,345,564	$1,908,234	
					Sub Total Base Construction Est	$18,165,108	$25,761,156	
	Engineering Fees	10%			Engineering Costs	$1,816,511	$2,576,116	
					Sub Total Design & Construction Base Est	$19,981,619	$28,337,271	
	Contingency to Most Likely Outturn	25%			Plus 50%ile Contingency	$4,995,405	$7,084,318	
					Expected Mean Outturn	$24,977,023	$35,421,589	$35,475,824
	Additional Contingency to 80%ile Outturn	15%			Plus 50-80%ile Contingency	$3,746,554	$5,313,238	
					80%ile outturn	$28,723,577	$40,734,827	
					Total Excl GST	$28,723,577	$40,734,827	

Falls Dam CFRD Raising Concept
Preliminary Construction Cost Estimate
FSL=588.0m

OPUS International Consultants

Prepared by	Ian Walsh
Date	30-Aug-12
Status	ROC Rev5
File	6CWI04.13

Assumptions
- Repairs to existing assets not directly studied
- Auxillary spillway unlined (to tolerate some damage in

Exclusions
- Land & Property Issues
- RMA Consent Costs
- Pioneer Generation Ltd Asset Impacts

All volumes are solid in place

Item	Description	Unit	Quantity	Original Nov04 rates Rate	Amount Nov04 $	Subtotals Nov-04 $	Subtotals Jun-12 $	P&G redistributed and amounts dajusted to EV	$
1	**Preliminary & General**		1	$1,500,000	$1,500,000	$1,500,000	$2,127,250	P&G 6.0%	$0
2	**Repairs and Maintenance**								
2.01	Membrane perimeretic joint above WL	m	201.41	$300	$60,423	397172.25	563256.5542		$889,248
2.02	Membrane perimeretic joint below WL	m	0	-					
2.03	Membrane panel joint above WL	m	683.22	$250	$170,805				
2.04	Membrane panel joint below WL	m	0	-					
2.05	Membrane concrete surface repair	m2	760.05	$85	$64,604				
2.06	Membrane concrete surface sealing	m2	5067	$20	$101,340				
2.07	Glory Hole concrete lining CJ repair	m	0	$510	$0	$0	$0		$0
2.08	Glory Hole concrete lining surface repair	m2	0	$250	$0				
2.09	Tunnel concrete lining CJ repair	m	0	$510	$0				
2.10	Tunnel concrete lining surface repair	m2	0	$250	$0				
3	**Access Portal Extension**					$0	$0		$0
3.1	Reinf concrete footing & invert slab	m3	0	$1,000	$0				
3.2	Connection to existing portal	LS	0	$3,000	$0				
3.3	CSP multiplate 2.5m dia arch	t	0	$18,000	$0				
3.4	Mitre bends - multi plate specials	ea	0	$3,000	$0				
3.5	Reinf concrete portal stiffener ring & security gate	LS	0	$3,000	$0				
4	**Access Roading**					$250,170	$354,783		$560,118
4.1	TL weathered rock cut to TR turning platform formation fill	m3	4000	$14.00	$56,000				
4.2	TL fresh rock cut to dam toe fill	m3	3700	$22.00	$81,400				
4.3	TL pavement shaping and running course	m2	800	$4.50	$3,600				
4.4	TR rock cut to lower platform fill	m3	4500	$22.00	$99,000				
4.5	TR pavement shaping and running course	m2	1200	$4.50	$5,400				
4.6	Crest pavement shaping and running course	m2	1060	$4.50	$4,770				
5	**Membrane Plinth**					$535,865	$759,946		$1,199,774
5.1	Loose rock excavation	m3	311.5381	$120	$37,385				
5.2	Insitu rock excavation	m3	207.6921	$160	$33,231				
5.3	Sealing at penstock cutting	PS	1	$10,000	$10,000				
5.4	Install passive 3.6m grouted rock anchors	ea	207	$650	$134,550				
5.5	Demolish existing plinth/membrane terminations	ea	2	$2,000	$4,000				
5.6	Foundation grouting including drilling	t	51.92302	$4,500	$233,654				
5.7	Concrete plinth (reinforced)	m	207.6921	$250	$51,923				
5.8	Perimetric joint	m	207.6921	$120	$24,923				
5.9	Demolish existing coping wall & hand rail	m	155	$40	$6,200				
6	**Spillway Cutting**					$7,300,000	$10,352,619		$16,344,328
6.1	Site stripping etc	LS	1	$20,000	$20,000				
6.2	Rock cutting to waste	m3	61,000	$33	$2,013,000				
6.3	Rock cutting to embankment fill	m3	400,000	$12	$4,800,000				
6.4	Dental concrete including local undercutting (unreinforced)	m3	400	$500	$200,000				
6.5	110mm Shotcrete including mesh & grouted dowel anchors	m2	1000	$120	$120,000				
6.6	3.6m grouted rock anchors including drilling - to secure blocks	ea	60	$650	$39,000				
6.7	Consolidation grouting including drilling - to lock in invert blocks	t	24	$4,500	$108,000				
7	**Dam Embankment**					$4,649,849	$6,594,262		$10,410,776
7.1	Foundation preparation	m2	18100	$1	$18,100				
7.2	Progressive benching of existing face	LS	1	$21,000	$21,000				
7.3	Place and compact selected general (through grizzly if required) rockfill from cuts	m3	429664.1	$8	$3,437,313				
7.4	Place and compact selected processed rockfill from cuts in upstream crest zone	m3	29335.91	$40	$1,173,437				
8	**Concrete Membrane Extension**					$1,374,820	$1,949,725		$3,078,152

8.1	New rockfill face preparation	m2	8381.69	$2	$16,763			
8.2	Horizontal panel joints	m	347	$120	$41,640			
8.3	Vertical panel joints	m	838.169	$120	$100,580			
8.4	Slipformed 300mm thick reinforced concrete membrane panels	m2	8381.69	$140	$1,173,437			
8.5	New coping wall & handrail	m	212	$200	$42,400			
9	**Auxiliary Spillway Crest Works (not fuse plug concept)**					$168,000	$238,252	$376,143
9.1	base reinf slab	m3	240	$600	$144,000			
9.2	Fuse plug apron reinf slab	m3	0	$600	$0			
9.3	Abutment reinf walls	m3	30	$800	$24,000			
9.4	Reinf pier	m3	0	$1,200	$0			
9.5	Bridge deck 15m DHC post tensioned units including transverse tie bars	ea	0	$10,000	$0			
9.6	Bridge side rails, joints etc	LS	0	$10,000	$0			
9.7	Spillway guide wall anchors	ea	0	$800	$0			
9.8	Spillway guide wall reinf concrete	m3	0	$1,000	$0			
9.9	Supply and place zoned fuse plug fill	m3	0	$100	$0			
10	**Glory Hole Spillway Works**					$2,392,186	$3,392,519	$5,355,983
10.1	Temporary access track	LS	1	$5,000	$5,000			
10.2	Foundation preparation/excavation	LS	1	$3,000	$3,000			
10.3	Reinforced footing including water stops - primary conc	m3	0	$1,200	$0			
10.4	Foundation grouting including drilling	t	10	$4,500	$45,000			
10.5	Drainage relief holes	ea	12	$600	$7,200			
10.6	concrete spillway plug	m3	115.45	$700.00	$80,817.47			
10.7	exposed 32m high steel pipe 4.4m dia towe	t	93.05	$5,000.00	$465,231.75			
10.8	paint	m2	884.67	$40.00	$35,386.90			
10.9	2.3m supply pipe	m	159.00	$6,650.00	$1,057,350.00			
10.10	1.8m syphon pipe	m	40.00	$4,430.00	$177,200.00			
10.11	Valves 2.0 RL 565 & RL580	ea	2.00	$240,000.00	$480,000.00			
10.12	Actuators	ea	2.00	$18,000.00	$36,000.00			
11	**Additional / Misc Items**					$7,085,885	$10,048,967	$15,864,935
	(add any further identified items)				$0			
11.1	extra over 6.3 for R2 rock 33%	m3	151800	$40	$6,072,000			
11.2	Saddle dam (62,000m3 E/W)	LS	1	$1,013,885	$1,013,885			
11.3					$0			
					Sub Total Net Construction Est	$25,653,947	$36,381,580	
	Unscheduled items	8%			Unscheduled Allowance	$2,052,316	$2,910,526	
					Sub Total Base Construction Est	$27,706,263	$39,292,107	
	Engineering Fees	10%			Engineering Costs	$2,770,626	$3,929,211	
					Sub Total Design & Construction Base Est	$30,476,890	$43,221,317	
	Contingency to Most Likely Outturn	25%			Plus 50%ile Contingency	$7,619,222	$10,805,329	
					Expected Mean Outturn	$38,096,112	$54,026,646	$54,079,459
	Additional Contingency to 80%ile Outturn	15%			Plus 50-80%ile Contingency	$5,714,417	$8,103,997	
					80%ile outturn	$43,810,529	$62,130,643	
					Total Excl GST	$43,810,529	$62,130,643	

Falls Dam RCC replacement Concept
Preliminary Construction Cost Estimate
FSL=577.0m

OPUS International Consultants

Prepared by	Ian Walsh
Date	30-Aug-12
Status	ROC Rev5
File	6CWI04.13

Assumptions
- Repairs to existing assets not Required, use as diversion works and keep operational during Construction
- Decommission Glory Hole

Exclusions
- Land & Property Issues
- RMA Consent Costs
- Pioneer Generation Ltd Asset Impacts

All volumes are solid in place

Item	Description	Unit	Quantity	Amount	Respread P&G	Adjust Base to EV
					P&G	
1	**Preliminary & General**		1	$2,073,354	6.5%	
2	**Repairs and Maintenance**					
2.01	Membrane perimeteric joint above WL	m	0			
2.02	Membrane perimeteric joint below WL	m	0			
2.03	Membrane panel joint above WL	m	0			
2.04	Membrane panel joint below WL	m	0			
2.05	Membrane concrete surface repair	m2	0			
2.06	Membrane concrete surface sealing	m2	0			
2.07	Glory Hole concrete lining CJ repair	m	0			
2.08	Glory Hole concrete lining surface repair	m2	0			
2.09	Tunnel concrete lining CJ repair	m	0			
2.10	Tunnel concrete lining surface repair	m2	0			
3	**Access Adit**			$28,363	$30,319	$38,154
3.1	Reinf concrete footing & invert slab	m3	0			
3.2	Connection to existing portal	LS	0			
3.3	CSP multiplate 2.5m dia arch	t	0			
3.4	Mitre bends - multi plate specials	ea	0			
3.5	Bulkhead plug seal	m3	20			
4	**Access Roading**			$353,921	$378,326	$476,085
4.1	TL weathered rock cut to TR turning platform formation fill	m3	4000			
4.2	TL fresh rock cut to dam toe fill	m3	3700			
4.3	TL pavement shaping and running course	m2	800			
4.4	TR rock cut to lower platform fill	m3	4500			
4.5	TR pavement shaping and running course	m2	1200			
4.6	Crest pavement shaping and running course	m2	925			
5	**Membrane Plinth**			$8,793	$9,399	$11,828
5.1	Loose rock excavation	m3	0			
5.2	Insitu rock excavation	m3	0			
5.3	Sealing at penstock cutting	PS	0			
5.4	Install passive 3.6m grouted rock anchors	ea	0			
5.5	Demolish existing plinth/membrane terminations	ea	0			
5.6	Foundation grouting including drilling	t	0			
5.7	Concrete plinth (reinforced)	m	0			
5.8	Perimetric joint	m	0			
5.9	Demolish existing coping wall & hand rail	m	155			
6	**Aggregate Production (P&G include)**			$6,036,849	$6,426,263	$8,086,809
6.1	Quarry Site stripping etc	LS	1			

6.2	Win and process rock to Concrete grades from quarry rate as per finished concrete measure	m3	59,137			
6.3	Win and process rock to Concrete grades from existing embankment	m3	23,200			
	Foundation Preparation & Treatment (P&G INCLUDE)					
	Foundation preparation includes grouting	m2	4188	$2,636,975	$2,807,076	$3,532,424
	110mm Shotcrete including mesh & grouted dowel anchors	m2	4000			
	3.6m grouted rock anchors including drilling - to secure blocks	ea	0			
	Consolidation grouting including drilling - to lock in invert blocks	t	0			
7	**RCC Dam (P&G in these rates)**			$21,753,328	$23,156,550	$29,140,203
7.2	place conventional concrete unreinf	m3	22464			
7.3	mix & place RCC unreinf includes joints and lift prep excludes aggregate production	m3	56425			
7.4	form galleries	m2	5670			
7.5	precast and cast in situ reinf concrete	m3	3448			
7.6						
7.7						
7.8						
7.9		m	0			
7.1		m2	0			
7.11		m	0			
9	**Auxiliary Spillway Crest Works (not fuse plug concept)**			$0	$0	$0
9.1	base reinf slab	m3	0			
9.2	Fuse plug apron reinf slab	m3	0			
9.3	Abutment reinf walls	m3	0			
9.4	Reinf pier	m3	0			
9.5	Bridge deck 15m DHC post tensioned units including transverse tie bars	ea	0			
9.6	Bridge side rails, joints etc	LS	0			
9.7	Spillway guide wall anchors	ea	0			
9.8	Spillway guide wall reinf concrete	m3	0			
9.9	Supply and place zoned fuse plug fill	m3	0			
10	**Glory Hole Spillway Works**			$199,536	$213,295	$268,410
10.1	Temporary access track	LS	1			
10.2	Foundation preparation/excavation	LS	1			
10.3	Reinforced footing including water stops - primary conc	m3	0			
10.4	Foundation grouting including drilling	t	10			
10.5	Drainage relief holes	ea	12			
10.6	concrete spillway plug	m3	115.00			
10.7	exposed 24m high steel pipe 4.4m dia tower	t	0.00			
10.8	paint	m2	0.00			
11.0	**Offtake Works in RCC**			$1,013,600	$1,083,492	$1,363,466
10.9	1.9m supply pipe	m	140.00			
10.10	1.6m syphon pipe	m	0.00			
10.11	Valves 1.6 @ RL 565 & RL550	ea	2.00			
10.12	Actuators	ea	2.00			
12	**Additional / Misc Items** (add any further identified items)			$0	$0	$0
12.1	extra over 6.3 for R2 rock 33%	m3	0			
12.2						

12.3					
			$34,104,719	$34,104,719	
	Unscheduled items	4%	$1,364,189	$1,364,189	
			$35,468,908	$35,468,908	
	Engineering Fees	10%	$3,546,891	$3,546,891	
			$39,015,799	$39,015,799	
	Contingency to Most Likely Outturn	10%	$3,901,580	$3,901,580	
			$42,917,379	$42,917,379	$42,917,379
	Additional Contingency to 80%ile Outturn	15%	$6,437,607		
			$49,354,986		

Falls Dam RCC replacement Concept
Preliminary Construction Cost Estimate
FSL=588.0m

OPUS International Consultants

Prepared by	Ian Walsh
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Assumptions
- Repairs to existing assets not Required, use as diversion works and keep operational during Construction
- Decommission Glory Hole

Exclusions
- Land & Property Issues
- RMA Consent Costs
- Pioneer Generation Ltd Asset Impacts

All volumes are solid in place

Item	Description	Unit	Quantity	Amount	Respread P&G	Adjust Base to EV
					P&G	
1	**Preliminary & General**		1	$3,289,874	6.5%	
2	**Repairs and Maintenance**					
2.01	Membrane perimeteric joint above WL	m	0		$0	$0
2.02	Membrane perimeteric joint below WL	m	0			
2.03	Membrane panel joint above WL	m	0			
2.04	Membrane panel joint below WL	m	0			
2.05	Membrane concrete surface repair	m2	0			
2.06	Membrane concrete surface sealing	m2	0			
2.07	Glory Hole concrete lining CJ repair	m	0		$0	$0
2.08	Glory Hole concrete lining surface repair	m2	0			
2.09	Tunnel concrete lining CJ repair	m	0			
2.10	Tunnel concrete lining surface repair	m2	0			
3	**Access Adit**			$28,483	$30,331	$38,168
3.1	Reinf concrete footing & invert slab	m3	0			
3.2	Connection to existing portal	LS	0			
3.3	CSP multiplate 2.5m dia arch	t	0			
3.4	Mitre bends - multi plate specials	ea	0			
3.5	Bulkhead plug seal	m3	20			
4	**Access Roading**			$355,417	$378,472	$476,270
4.1	TL weathered rock cut to TR turning platform formation fill	m3	4000			
4.2	TL fresh rock cut to dam toe fill	m3	3700			
4.3	TL pavement shaping and running course	m2	800			
4.4	TR rock cut to lower platform fill	m3	4500			
4.5	TR pavement shaping and running course	m2	1200			
4.6	Crest pavement shaping and running course	m2	925			
5	**Membrane Plinth**			$8,830	$9,403	$11,832
5.1	Loose rock excavation	m3	0			
5.2	Insitu rock excavation	m3	0			
5.3	Sealing at penstock cutting	PS	0			
5.4	Install passive 3.6m grouted rock anchors	ea	0			
5.5	Demolish existing plinth/membrane terminations	ea	0			
5.6	Foundation grouting including drilling	t	0			
5.7	Concrete plinth (reinforced)	m	0			
5.8	Perimetric joint	m	0			
5.9	Demolish existing coping wall & hand rail	m	155			
6	**Aggregate Production (P&G include)**			$9,763,929	$10,397,302	$13,083,965
6.1	Quarry Site stripping etc	LS	1			
6.2	Win and process rock to Concrete grades from quarry rate as per finished concrete measure	m3	59,137			
6.3	Win and process rock to Concrete grades from existing embankment	m3	23,200			
	Foundation Preparation & Treatment (P&G INCLUDE)					

	Foundation preparation includes grouting	m2	5848	$3,598,468	$3,831,896	$4,822,058
	110mm Shotcrete including mesh & grouted dowel anchors	m2	5000			
	3.6m grouted rock anchors including drilling - to secure blocks	ea	0			
	Consolidation grouting including drilling - to lock in invert blocks	t	0			
7	**RCC Dam (P&G in these rates)**			$33,863,342	$36,060,013	$45,377,920
7.2	place conventional concrete unreinf	m3	31867			
7.3	mix & place RCC unreinf includes joints and lift prep excludes aggregate production	m3	104047			
7.4	form galleries	m2	6210			
7.5	precast and cast in situ reinf concrete	m3	4192			
7.6						
7.7						
7.8						
7.9		m	0			
7.1		m2	0			
7.11		m	0			
9	**Auxiliary Spillway Crest Works (not fuse plug concept)**			$0	$0	$0
9.1	base reinf slab	m3	0			
9.2	Fuse plug apron reinf slab	m3	0			
9.3	Abutment reinf walls	m3	0			
9.4	Reinf pier	m3	0			
9.5	Bridge deck 15m DHC post tensioned units including transverse tie bars	ea	0			
9.6	Bridge side rails, joints etc	LS	0			
9.7	Spillway guide wall anchors	ea	0			
9.8	Spillway guide wall reinf concrete	m3	0			
9.9	Supply and place zoned fuse plug fill	m3	0			
10	**Glory Hole Spillway Works**			$200,379	$213,378	$268,514
10.1	Temporary access track	LS	1			
10.2	Foundation preparation/excavation	LS	1			
10.3	Reinforced footing including water stops - primary conc	m3	0			
10.4	Foundation grouting including drilling	t	10			
10.5	Drainage relief holes	ea	12			
10.6	concrete spillway plug	m3	115.00			
10.7	exposed 24m high steel pipe 4.4m dia tower	t	0.00			
10.8	paint	m2	0.00			
11.0	**Offtake Works in RCC**			$1,453,115	$1,547,376	$1,947,218
10.9	2.3m supply pipe	m	140.00			
10.10	1.8m syphon pipe	m	0.00			
10.11	Valves 2.0 @ RL580 & RL550	ea	2.00			
10.12	Actuators	ea	2.00			
12	**Additional / Misc Items** (add any further identified items)			$1,443,934	$1,537,600	$1,934,916
12.1	extra over 6.3 for R2 rock 33%	m3	0			
12.2	Saddle dam (62,000m3 E/W)	LS	1.0			
12.3						
				$54,005,770	$54,005,770	
	Unscheduled items	4%		$2,160,231	$2,160,231	
				$56,166,001	$56,166,001	
	Engineering Fees	10%		$5,616,600	$5,616,600	
				$61,782,601	$61,782,601	
	Contingency to Most Likely Outturn	10%		$6,178,260	$6,178,260	
				$67,960,861	$67,960,861	$67,960,861
	Additional Contingency to 80%ile Outturn	15%		$10,194,129		
				$78,154,991		

Offtake / hydropower supply pipework scoping estimate breakdown

Assuming a powerscheme on outlet at tunnel portal - no terminal discharge valve

CFRD 588m via tunnel	unit	Q	rate		$2,331,986.12
concrete spillway plug	m3	115.45	$700.00	$80,817.47	
exposed 32m high steel pipe 4.4m dia tower	t	93.05	$5,000.00	$465,231.75	
paint	m2	884.67	$40.00	$35,386.90	
2.3m supply pipe	m	159.00	$6,650.00	$1,057,350.00	
1.8m syphon pipe	m	40.00	$4,430.00	$177,200.00	
Valves 2.0 RL 565 & RL580	ea	2.00	$240,000.00	$480,000.00	
Actuators	ea	2.00	$18,000.00	$36,000.00	

CFRD 577m via tunnel	unit	Q	rate		$1,606,874.25
concrete spillway plug	m3	115.45	$700.00	$80,817.47	
exposed 24m high steel pipe 4.4m dia tower	t	65.27	$5,000.00	$326,356.60	
paint	m2	663.50	$40.00	$26,540.17	
1.9m supply pipe	m	159.00	$4,840.00	$769,560.00	
1.6m syphon pipe	m	40.00	$3,640.00	$145,600.00	
Valves 2.0 @ RL 565	ea	1.00	$240,000.00	$240,000.00	
Actuators	ea	1.00	$18,000.00	$18,000.00	

Need to mine 15m out of existing emabnkment

RCC 588 In dam	unit	Q	rate		$1,527,817.47
concrete spillway plug	m3	115.45	$700.00	$80,817.47	
exposed 24m high steel pipe 4.4m dia tower	t	0.00	$5,000.00	$0.00	
paint	m2	0.00	$40.00	$0.00	
2.3m supply pipe	m	140.00	$6,650.00	$931,000.00	
1.8m syphon pipe	m	0.00	$4,430.00	$0.00	
Valves 2.0 @ RL580 & RL550	ea	2.00	$240,000.00	$480,000.00	
Actuators	ea	2.00	$18,000.00	$36,000.00	

Need to mine 15m out of existing emabnkment

RCC 577 In dam	unit	Q	rate		$1,094,417.47
concrete spillway plug	m3	115.45	$700.00	$80,817.47	
exposed 24m high steel pipe 4.4m dia tower	t	0.00	$5,000.00	$0.00	
paint	m2	0.00	$40.00	$0.00	
1.9m supply pipe	m	140.00	$4,840.00	$677,600.00	
1.6m syphon pipe	m	0.00	$3,640.00	$0.00	
Valves 1.6 @ RL 565 & RL550	ea	2.00	$150,000.00	$300,000.00	
Actuators	ea	2.00	$18,000.00	$36,000.00	

Morning glory stays in service

CFRD 567.5 use existing penstock or enhanced for 6	unit	Q	rate		$837,000.00
concrete spillway plug	m3	0.00	$700.00	$0.00	
exposed 24m high steel pipe 4.4m dia tower	t	0.00	$5,000.00	$0.00	
paint	m2	0.00	$40.00	$0.00	
duplicate 1.2 syphonic penstock	m	250.00	$2,260.00	$565,000.00	
duplicate 1.2m valves and chamber	LS	1.00	$200,000.00	$200,000.00	
rebuild vacuum pump house	ea	1.00	$60,000.00	$60,000.00	
Actuators	ea	1.00	$12,000.00	$12,000.00	

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