WAIMEA WATER AUGMENTATION COMMITTEE

Lee Valley Storage Dam
Engineering Feasibility Report

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

REPORT

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Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE

Report prepared by:

TONKIN & TAYLOR LTD

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Executive summary

This report describes and summarises the work undertaken to assess the engineering feasibility of a water augmentation storage dam on the Lee River, forming part of the Waimea Water Augmentation Project. The engineering assessment is complemented by geotechnical and water resource investigations, reported in separate volumes.

Twelve sites on the Lee River were selected for preliminary engineering comparison purposes. The locations were between chainage 10,200 m and chainage 12,400 m. Earth embankment, concrete faced rockfill, and roller compacted concrete dams were considered. The size of dams at each location were estimated based on storage-elevation curves, and the approximate cost for each type of dam estimated, to show the relativity between sites. Initial evaluation of geotechnical conditions at each site were also undertaken. The evaluation indicated that a dam located at chainage 11, 010 m offered the most economic solution.

Subsequent geotechnical investigations at that site revealed poor founding conditions on the right abutment of the dam, and potentially unstable slopes on the left bank of the reservoir. When viewed cumulatively, these issues had an adverse effect on potential cost and programme in relation to a dam at chainage 11,010 m. A decision was subsequently taken to investigate an alternative site located between chainage 12,100 m and 13,000 m.

On the basis of preliminary engineering geological mapping, consideration of earthworks volumes, and construction materials availability a site based on approximately Ch 12,400m was selected for drilling investigations. The preliminary dam design reported in this report is based on a concrete faced rockfill dam at Ch 12,430m.

Dams in New Zealand are categorised based on their Potential Impact Category (PIC). Three levels of PIC are set for large dams: Low, Medium and High. The New Zealand Dam Safety Guidelines provide guidance for selection of PIC based on the social, economic and environmental consequences of a hypothetical failure. The Building Act also now provides requirements for selection of PIC, although these requirements are now under review and may be revised.

Selection of the PIC leads to definition of the extreme events that a dam should withstand. Selection of PIC for the proposed Lee Dam has been based on a dambreak assessment undertaken specifically for this project which concluded that the PIC would be High. The design standards for the dam have therefore been selected on this basis.

The arrangement of a dam includes an almost infinite number of combinations of spillway type, embankment type, freeboard allowance, and outlet systems. The study included assessment of options for components, including the following:

Embankment types:	Zoned earthfill
	Concrete faced rockfill
	Roller compacted concrete

Spillway types:	Combination primary and auxiliary spillway
	Ogee weir primary
	Labyrinth weir primary
	Bell-mouth primary with dropshaft

A summary of the arrangement and specifications for the selected dam and spillway are listed in Table 1.

Table 1: Summary and Specifications

Embankment Characteristics		
Normal top water level (NTWL)	RL 197	m
Embankment type	Concrete faced rock fill (CFRD)	
Crest elevation	RL 201	m
Maximum flood water level	RL 201.58	m
Maximum dam height (from riverbed to dam crest)	52	m
Crest length	220	m
Wave wall height	1	m
Spillway Characteristics		
Total peak outflow OBF	372	m^3/s
Total peak outflow MDF	1036	m^3/s
Primary spillway type	Ogee Weir	
Primary spillway width	22.3	m
Peak outflow OBF for primary spillway component	372	m^3/s
Peak outflow MDF for primary spillway component	449	m^3/s
Auxiliary spillway type	Fuseable Embankment	
Auxiliary spillway width	19.5	m
Peak outflow MDF for auxiliary spillway component	606	m^3/s
Spillway Chute and Energy Dissipation Characteristics		
Chute length (plan)	105	m
Chute width, wide section	22.3	m
Chute width, narrow section	10	m
Chute minimum wall height	4	m
Dissipation type	Flip Bucket	
Flip bucket radius	25	m
Bucket lip level	RL 156.58	m

Outlet Characteristics	
Number of outlets	2
Outlet type	Sloping outlet conduits on upstream face with removable screens and gate control.
Outlet level – Upper	RL 185 m
Outlet level – Lower	RL 167 m
Control gate type	Radial
Control gate size	1 x 1 m
Conveyance conduit size (under embankment)	2.5 x 5 m
Number of conveyance conduits	2 (access via third)
Conveyance conduit downstream protection	Stoplogs

A Concrete Faced Rockfill Dam (CFRD) has been selected, which uses a concrete slab on the upstream face as a waterproofing element.

Linking the upstream face into the foundation is a critical component of this type of dam. This is achieved by the plinth which is a concrete slab cast against the prepared foundation surface and tied to the foundation with grouted reinforcing bars. Grout is also injected into the foundation where necessary to reduce leakage to acceptable amounts.

The internal zoning of the dam is arranged to minimise settlement of the upstream face during first reservoir filling, and to manage leakage in the event cracks form through the upstream concrete face. The zoning also makes most economical use of the materials which are available locally at the dam site, and preferably from excavations required for the spillway and other related activities.

The dam foundation is formed by in-situ rock of various weathering grades, as determined during the geotechnical investigations. The target depth for subexcavation varies across the footprint as different parts of the dam require different quality materials as a foundation. The dam plinth requires the best foundation to minimise potential leakage. The size of the plinth is related to the foundation quality. This will require significant excavation in some areas, especially at the left abutment.

The foundation under the body of the dam has a lower requirement for quality. The main objective in this area is to remove material which could result in additional settlement of the dam embankment, or form weak planes (shear surfaces) under the embankment.

The internal zoning of the dam serves a number of objectives, including:

- using the available materials to most economic effect,
- controlling settlement of the dam to amounts that will not cause distress of the upstream concrete face,
- allowing seepage flow through the dam body without the formation of a high phreatic surface, both in the case of normal operation and if cracking forms in the upstream face allowing larger leakage,
- providing a bedding layer for formation of the upstream face and
- providing stability against static and seismic loadings.

These objectives are sometimes conflicting and a compromise must be reached in developing the internal zoning.

Diversion of the Lee River during construction is a critical process. Diversion is proposed through two or three concrete culverts located underneath the embankment. Each culvert has internal dimensions of 2.5m width and 5.0m height. A concrete starter dam at the upstream toe will form the coffer dam for directing flow through the culverts.

When the embankment is at design height, the culverts will no longer be required for flood diversion. They will then be converted for use as discharge of irrigation flow, and person access to the gate/valve control chamber at the upstream toe.

An outline of the anticipated construction methodology for the dam, including requirements for construction facilities, borrow areas, disposal areas, etc. has been developed. A critical part of this is careful control of river diversion for the safety of the dam during construction.

The construction process is a continuum, but has been broken into several nominal stages. The details are included in the body of this report. In summary the stages are:

1. Preparation for and construction of diversion culverts;
2. Construction of the diversion culverts;
3. Diversion of the river through the culverts;
4. Construction of the plinth and commencement of the dam embankment;
5. Completion of the embankment and construction of the spillway;
6. Placement of the filter zones and forming the upstream concrete face;
7. Construction of upstream face intake conduits and intake gate structures, followed by plugging of the diversion. Passage of residual flows will be maintained through the irrigation outlets;
8. Construction of the fish passage structures and site restoration.

Estimation of the capital cost for construction of the dam and for potential hydro-electric power generator add-on has been undertaken. The estimation process for the dam alone has been more robust and detailed than that for the hydro-power component add-on.

The process has involved estimation of quantities of materials and items, and identification of likely rates. Percentages have been allowed for contingencies (20%), contractors' preliminary and general (15%), and design (10%).

Costs were estimated for cases of two and three diversion culverts (as discussed earlier in the report). A significant portion of cost is attributed to diversion during construction as shown by this assessment. The actual requirement for diversion will need to be developed during detailed design and construction methodology development as part of a risk assessment, including contractor inputs. At the current level of design the cost is estimated to lie somewhere between the two figures quoted below.

These cost estimates were then reviewed by experienced people in the construction industry, who have been involved with bidding for and constructing similar works. Comments from this review were included in a revised estimate. Tonkin & Taylor (T&T) internal, and external peer review was also carried out.

The cost estimate for the dam (water augmentation only) as of November 2009 is:

NZ$35.5 million (GST exclusive) for 2 culvert diversion

NZ$38.1 million (GST exclusive) for 3 culvert diversion

The bills of quantities associated with these estimates are included in Appendix C. It should be noted that the estimate for construction cost is for the dam area only, and does not include any of the following costs which may be extra to the overall development cost:

- Taxes
- Insurance
- Developer related costs
- Resource consenting
- Environmental mitigation
- Land purchase
- Financing
- Distribution or allocation management
- Operation and maintenance
- Environmental compliance
- Construction cost variations due to high demand
- Increases in costs of steel, fuel, or any other construction related material
- Other items not specifically identified in the bill of quantities.

The construction cost of the hydro-electric add-on has been estimated (to a low level of detail). At the recommended installed capacity of 1 MW, the total construction costs would be:

NZ$39.8 million (GST exclusive) for 2 culvert diversion,

NZ$42.4 million (GST exclusive) for 3 culvert diversion with the same assumptions and exclusions as outlined above.

The effect of reduced irrigation demand scenarios on construction cost has been estimated to allow consideration of sensitivity of cost to assumed demand. This analysis is presented in Appendix G.

1 Introduction

1.1 General

In 2007 Tonkin & Taylor Ltd and its sub-consultants completed a Phase 1 pre-feasibility evaluation of a number of options to provide water storage for long-term irrigation and community supplies in the Waimea Basin, Tasman District. The evaluation was undertaken on behalf of the Waimea Water Augmentation Committee (WWAC). The overall principle of the study was to identify and develop a water augmentation scheme to capture excess water for storage and release that water back into the Waimea River system during periods of high water demand and/or low natural water flows to augment those supplies, either directly or via recharging of the groundwater system.

The outcome of that Phase 1 study was to focus feasibility investigations on a water storage dam and reservoir site located in the upper Lee River catchment, a tributary of the Waimea River. In 2007 WWAC initiated Phase 2 of the study, to take the Lee investigation programme to a feasibility level.

This report presents the results of dam engineering investigations completed as part of the Phase 2 feasibility study. It is based on a potential dam on the Lee River in the Tasman District, at a site approximately 300 metres upstream of the confluence of Anslow Creek and the Lee River. The required storage capacity of the reservoir has been determined to be approximately 13 million m^3, with a normal top water level to RL 197m. The reservoir would extend approximately 4km upstream from the dam, and cover an area of approximately 65 hectares (based on normal top water level).

Figure 1-1 shows the location of the proposed dam, and the indicative reservoir extent.

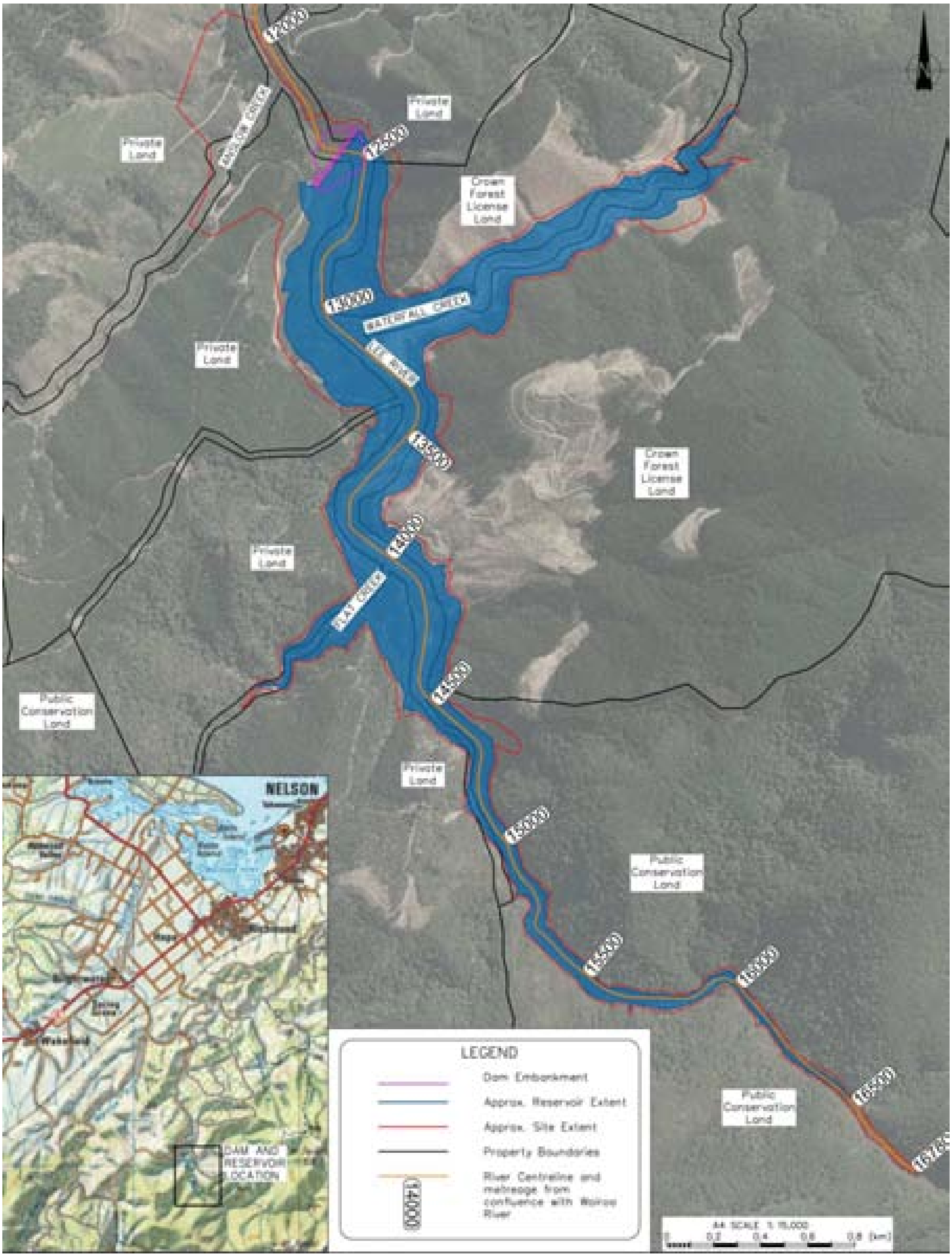

Figure 1-1: Location of proposed dam and reservoir

1.2 Report layout

This report is set out in four main parts. The dam site selection, design standards and inputs are outlined in sections 1 through to 9 and set the stage for evaluation of the various dam and spillway options.

The second part deals with evaluation of options at the selected dam site including embankment options, flood diversion and routing and an optimisation of spillway and dam crest parameters. This evaluation is summarised in section 10 with the majority of the assessment contained in Appendix A.

The third part of the report covers the arrangement and construction methodology of the selected dam and associated structures and is contained within sections 11 and 12. The arrangement is summarised and the engineering feasibility of the various components is discussed. The feasibility of the arrangement is largely divided between the embankment and the spillway arrangements followed by a description of the outlet works proposed.

Finally, sections 13 and 14 cover the capital construction cost estimates and provide conclusions and recommendations.

2 Previous Studies

There have been a number of previous studies that led to the feasibility study for a dam at the current proposed location on the Lee River. The Phase 1 study by T&T (2004-2007) for WWAC investigated the potential for water harvesting and storage in the upper parts of the Waimea catchment. This study considered:

- water availability
- storage site options
- environmental and economic analysis
- water allocation methods.

A preliminary scan of 18 storage site options was completed in December 2004, which was shortlisted to 3 by April 2005.

After considering the options, the Upper Lee Valley was selected as the preferred option for further investigation, and pre-feasibility level investigations continued based on a site approximately 1km downstream of the confluence of Anslow Creek and the Lee River (at Chainage[1] 11,010m). These investigations included site geology, geotechnical conditions, preliminary dam layout, and construction material sources. The current Phase 2 feasibility investigations commenced based on that general location.

Section 3 sets out the dam site selection process that were undertaken for Phase 2.

[1] A river referencing system has been set up for the project, based on the distance in metres upstream from the confluence of the Lee River with the Wairoa River.

3 Dam Site Selection

In the early stages of the current (Phase 2) study an assessment of the favoured site within the Lee Valley (at Chainage 11,010m) was carried out. This analysis is documented in the T&T "Optimisation of Dam Location and Type" report (December 2007). Twelve sites on the Lee River were selected for comparison purposes. The locations were between Chainage 10,200m and 12,400m and are shown on Figure 3-1.

Embankment, concrete faced rockfill, and roller compacted concrete dams were considered. The size of dams at each location was estimated based on storage-elevation curves, and the approximate cost for each type of dam estimated, to show the relativity between sites. Initial evaluation of geotechnical conditions at each site was also undertaken, based on visual assessment. The evaluation indicated that a dam located at Chainage 11, 010m offered the most economic solution.

Subsequent geotechnical investigations at that site revealed poor founding conditions on the right abutment of the dam, and potentially unstable slopes on the left bank of the reservoir. When viewed cumulatively, these issues had an adverse effect on potential cost and programme in relation to a dam at Chainage 11,010m. A decision was subsequently taken to investigate an alternative site located between Chainage 12,100m and 13,000m.

On the basis of preliminary engineering geological mapping and consideration of earthworks volumes, a site at Chainage 12,400m was selected for drilling investigations. The details of the geotechnical investigations are reported in the accompanying Technical Report "Geotechnical Investigations Report" by T&T.

The preliminary dam design reported in this report is based on a dam at approximately Chainage 12,430m.

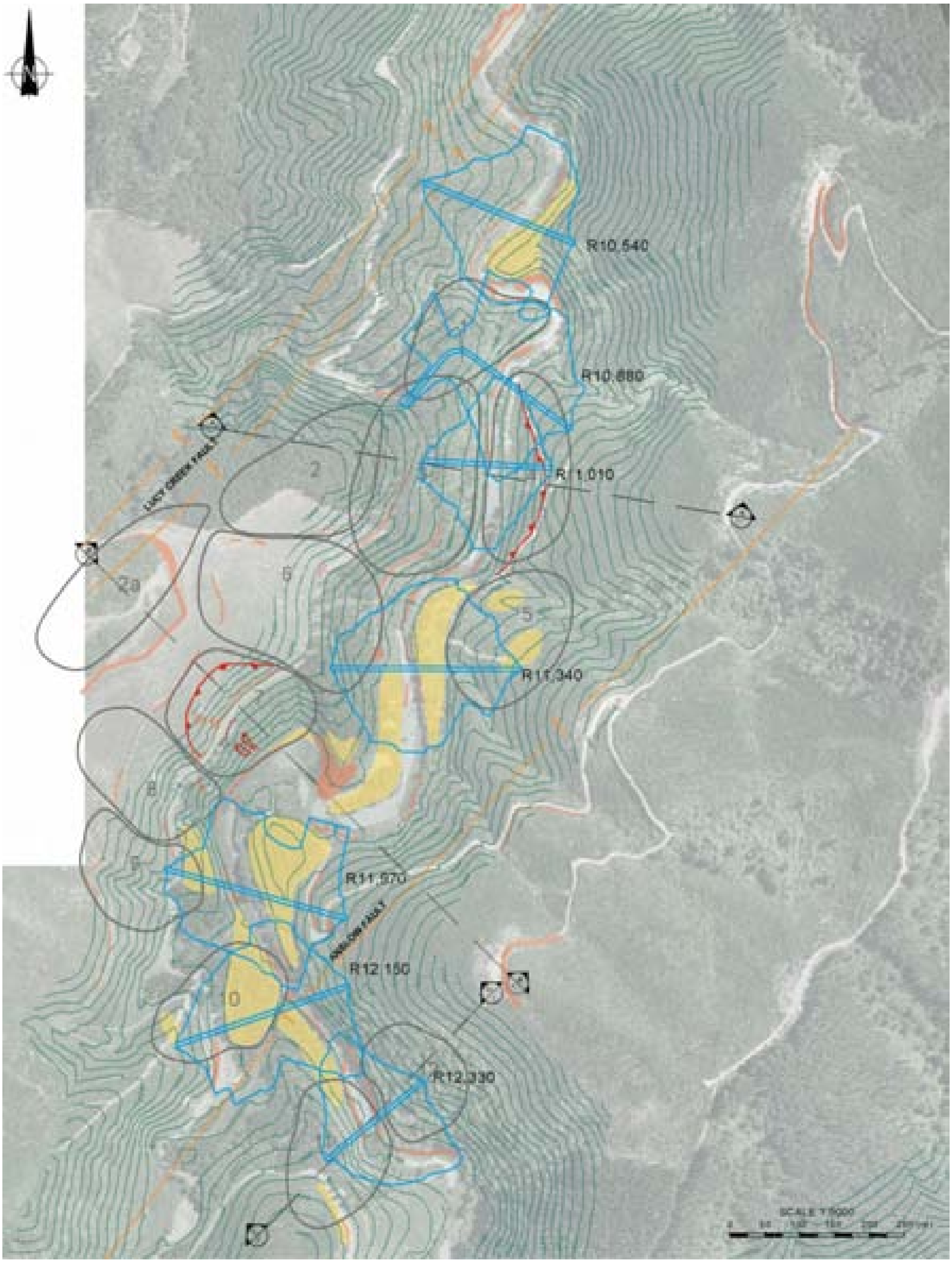

Figure 3-1: Locations considered in first stages of current (Phase 2) study

4 Site Topographical Characteristics

The topographical features of the selected dam site are shown on Figure 4-1. The Lee River runs in a north-easterly direction at the nominal upstream extent of the dam site, and then turns about 90 degrees to the left for a short distance, then continues in a north-westerly direction. The river bed is at about RL 150m in the region of the proposed dam site, which is also shown in outline form on Figure 4-1.

Anslow Creek is located to the left of Lee River, and flows at a generally higher elevation (and steeper gradient) until it joins the Lee River. About 300 m separates Lee River and Anslow Creek, with a ridge that rises to about RL 250 m. The end of the ridge that separates the two water courses is relatively rounded with flatter topography than the right bank of the Lee River.

Access tracks/roads leading up the valley are located on the left bank of the Lee, and traverse back up Anslow Creek to a crossing, then return down the valley to the left bank of the Lee. The area is generally forested with commercial exotic forest species and some pockets of indigenous vegetation.

Further information on the site characteristics is provided in the accompanying Technical Report “Geotechnical Investigations”.

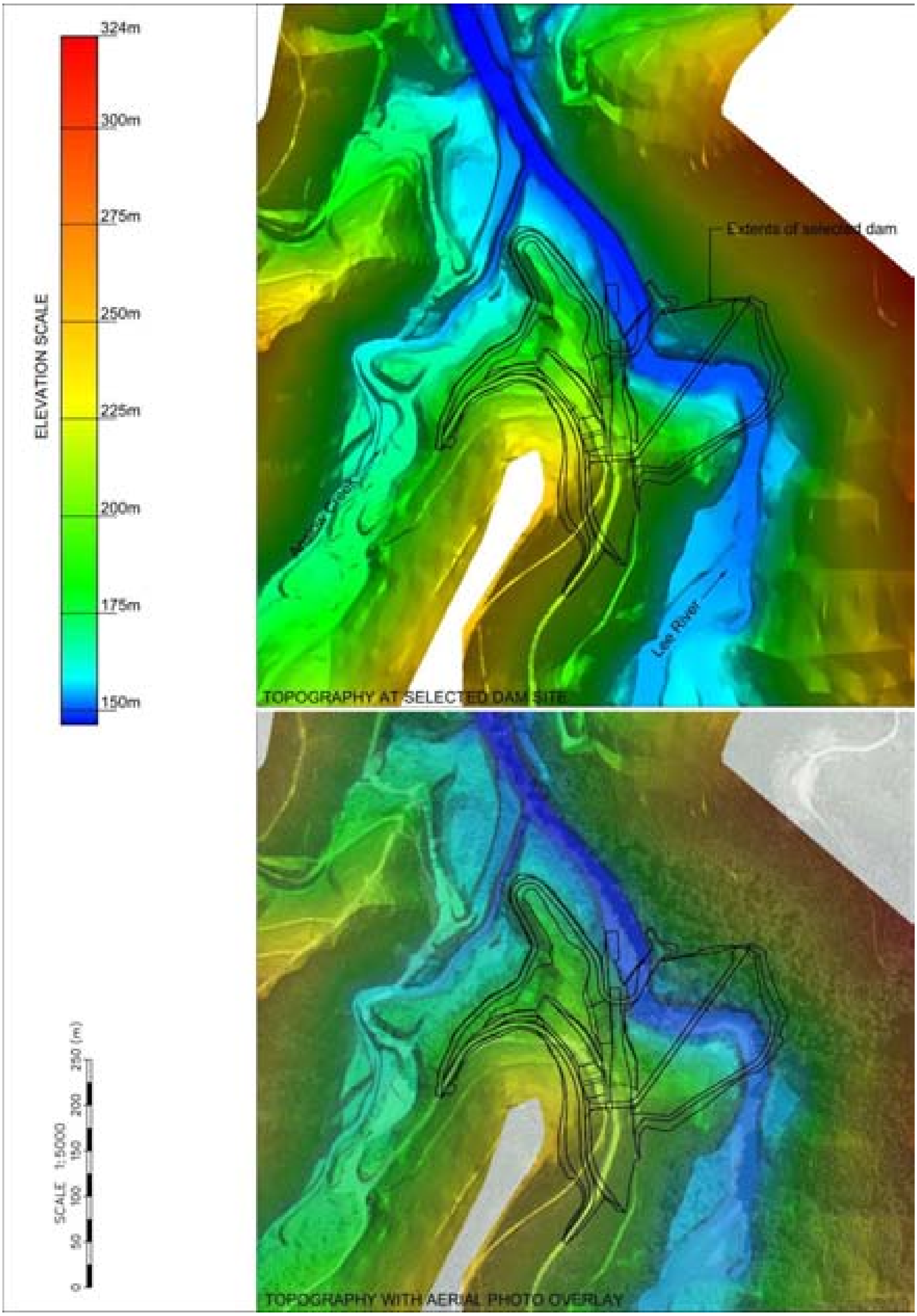

Figure 4-1: Topographical characteristics at selected site

5 Potential Impact Category and Design Standards

5.1 General

The standards adopted for dam design are in two main parts. One part specifies the extreme events (floods and earthquakes) that the dam must withstand, and the other specifies the factors of safety that the dam should display under the various loading cases.

Expected factors of safety are applicable to most large dams, and these are set out in the New Zealand Dam Safety Guidelines (NZSOLD, 2000).

The extreme events that the dam must withstand are dependent on the potential hazard that the dam poses in the event of an uncontrolled release (breach). For a dam which has relatively large consequences of failure, there is an expectation that it is able to withstand more extreme events prior to failure. This balance of consequence against likelihood of failure sets the overall risk profile for the dam.

Dams in New Zealand are categorised based on their Potential Impact Category (PIC). Three levels of PIC are set for large dams: Low, Medium and High. The New Zealand Dam Safety Guidelines provide guidance for selection of PIC based on the social, economic and environmental consequences of a hypothetical failure. The Building Act also now provides requirements for selection of PIC, but we understand these requirements are now under review and may be revised.

Selection of PIC leads to definition of the extreme events that a dam should withstand. Selection of PIC for the proposed Lee dam has been based on a dambreak assessment, and this is reported in the accompanying Technical Report “Dambreak Hazard Assessment”. The design standards for the dam have been selected on this basis, and are summarised in Section 5.3.

5.2 Conclusions from dambreak assessment

A dambreak assessment has been undertaken and is reported fully in Appendix E of this report. The assessment concluded that the PIC for the proposed dam would be High.

5.3 Standards adopted for dam design

The standards that have been adopted for the feasibility level design of the Lee Valley Dam are summarised in Table 5-1.

Table 5-1 – Standards adopted for Lee Dam feasibility design

Classification and Primary Design Standards				
Item	**Value**		**Source**	**Notes**
Potential Impact Classification (PIC)	High		Estimate	*Based on dambreak assessment*
Operational Basis Earthquake (OBE)	1:150 yr		NZSOLD	*Based on PIC*
Maximum Design Earthquake (MDE)	MCE		NZSOLD	*Based on PIC*
Operational Basis Flood (OBF)	1:200 yr		NZSOLD	*Industry custom/precedent*
Maximum Design Flood (MDF)	PMF		NZSOLD	*Based on PIC*
Construction Diversion Flood (CDF)	1:50 yr		None	*Refer discussion in report*
Minimum freeboard for 100 yr wave	0.5	m		*Industry custom*
Minimum freeboard at OBF+10 yr wave	0.5	m		*Industry custom*
Minimum freeboard at MDF+10 yr wave	0.0	m		*Industry custom*
Hydrology				
Item	**Value**		**Source**	**Notes**
Live storage volume	12,000,000	m^3	Demand studies	
Dead storage volume	1,000,000	m^3	Demand studies	
Total storage volume	13,000,000	m^3	Demand studies	
Peak inflow OBF	412	m^3/s	Hydrology studies	
Peak inflow MDF	1,094	m^3/s	Hydrology studies	
Peak inflow CDF	340	m^3/s	Hydrology studies	
Irrigation Outlet Requirements				
Item	**Value**		**Source**	**Notes**
Irrigation release flow at minWL	2.25	m^3/s	Demand studies	
Irrigation release flow at maxWL	2.25	m^3/s	Demand studies	
Flushing flow outlet requirement	5	m^3/s	Cawthron	
Water elevation range for flushing	All live storage		Cawthron	
Outlet requirements				Either to river or via hydro turbine

6 Geological and Geotechnical Conditions

A detailed description of the geological and geotechnical conditions in the region of the dam site is provided in the accompanying Technical Report "Geotechnical Investigation Report". This provides information on the investigations undertaken, and interpreted material properties. A brief summary of the geotechnical conditions expected at the site is provided here.

At the dam site bedrock consists of a sequence of Rai Formation greywacke sandstone and argillite beds generally dipping at between 30° to 60° towards the north-west. There is a progressive steepening of dip from upstream to downstream in the river exposures. No major fold axis has been identified and the reverse dip seen in the river exposures is not evident in outcrop at higher elevations. Locally, within the zone of south-east dipping rock, there are poorly formed chevron and kink folds and irregular quartz veins.

When broken down by weathering grade[2] the Unconfined Compressive Strength (UCS) is estimated as follows:

SW rock:	66 MPa parallel bedding	90 MPa perpendicular bedding
MW rock:	24 MPa parallel bedding	53 MPa perpendicular bedding

Rock mass defects have been mapped for the right and left abutment slopes. Rock mass defects include bedding, joints and sheared zones. Typical of greywacke rocks, there is a broad scatter of defect orientations. However, four roughly orthogonal and conjugate sets of defect orientations are recognised.

Sheared zones have been mapped, logged in core or inferred by surface lineaments. They form a generally orthogonal pattern of zones of weakness beneath the dam footprint. Sheared zones are mainly mapped parallel to bedding and dominant joint sets, although other orientations are also evident locally. The most common are bedding parallel sheared zones where argillaceous beds have been sheared and crushed between more competent sandstone beds. Bedding plane sheared zones vary from 20 mm thick to about 1 m wide incipient zones of shatter containing clay crushed seams. Persistent sheared zones are spaced at 10 to 50 m intervals.

The weathering process has altered the bedrock in two ways. Chemical weathering, primarily due to oxidation above the groundwater table, has leached, altered and redeposited minerals (notably iron), reduced the intact rock strength and altered the colour from blueish grey through to brown. The change in intact rock strength from UW to MW is minor, but there is a significantly lower intact strength in HW rock.

Weathering is more concentrated on defect surfaces. In the SW rock defects are often iron stained, and rarely contain silt, but are not noticeably weaker than UW defects. In MW rock, defect surfaces are discoloured and altered, are more open, and have regular infilling of cemented iron oxide and silt. Joint wall strength is lower than the UW to SW rock and joints are more closely spaced. This leads to higher permeability and lower rock mass strength in MW rock than SW or UW rock.

[2] UW: Unweathered; SW: Slightly Weathered; MW: Moderately Weathered; HW: Highly Weathered

Associated with the weathering process is a progressive dilation of the rock mass from UW through to HW. The greywacke is unweathered in river exposures. The depth of weathering varies around the site and in the drillholes.

Alluvial gravel is present at the potential dam site. Two low level terrace deposits exist on the right bank of the river. These contain gravelly sand, and sandy coarse gravel that is inferred to be 2 to 5 m thick. On the left abutment an elevated terrace gravel deposit overlies a rock bench at RL 170m. These gravels are slightly to moderately weathered. Generally 1 to 2 m of unweathered sand and gravel overlies rock in the active river bed. Locally, along the inferred location of a shear zone feature (mapped as SZ11), more than 3 m of gravel infills a 5 m wide eroded slot in the river bed. All gravels are unconsolidated (loose to medium dense).

On the left abutment, up to 5 m of gravelly SAND (solifluction) overlies the gravel deposit and rock bench at RL 170 m. Elsewhere on the left abutment soil consist of gravelly sand colluviums generally less than 1 m thick.

On the right abutment soils consist of unweathered, poorly graded gravel, within scree deposits that blanket steeper slopes (>35°) downslope of rock outcrops and infill shallow steeply plunging gullies to 4 m depth. Elsewhere the slope is mantled by generally less than 1 m of gravelly sand colluviums.

7 Hydrological Flood Conditions

7.1 2.33 Year to 10,000 Year ARI Floods

Flood inflow hydrographs are used as the primary input to flood routing models (e.g. HEC-ResSim). These flood routing models are used to model reservoir attenuation and outflow, and hence design the spillway provisions at a dam.

Synthetic inflow hydrographs for the dam site, which has a catchment area of 77.5 km^2, for average recurrence intervals (ARI) of 2.33, 5, 10, 20, 50, 100, 200, 1000 and 10,000 years have been computed, and are shown in Figure 7-1. The peak flow and 48 hour inflow volume for each ARI flood is noted in Table 7-1.

Table 7-1 Peak inflow and flood volume to Lee Dam reservoir

Average Recurrence Interval of Flood	Peak Inflow (m3/s)	48 Flow Volume (million m3)
2.33 years (mean annual)	168	10.3
5 years	216	13.8
10 years	255	16.6
20 years	292	19.2
50 years	339	22.7
100 years	375	25.0
200 years	412	27.9
1000 years	496	33.9
10000 years	616	42.4

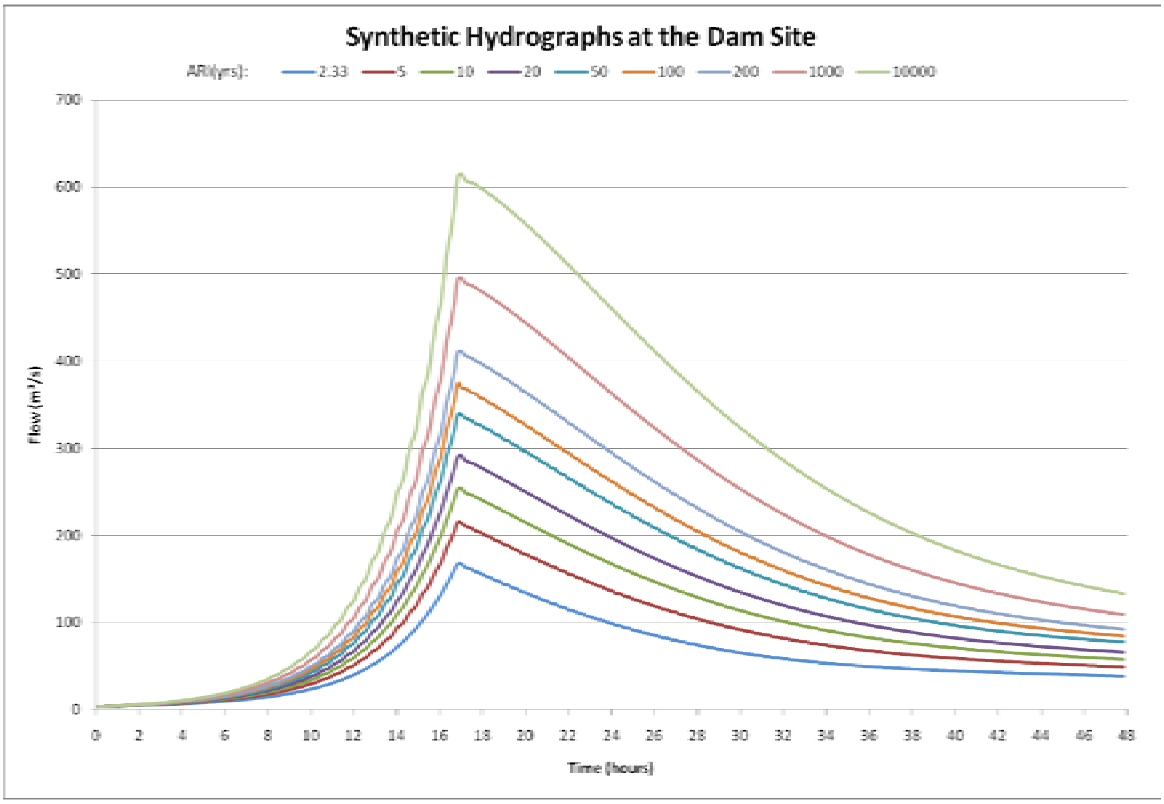

Figure 7-1: Synthetic inflow hydrographs for Lee Dam site (77.5 km^2)

These synthetic hydrographs have been derived from a statistical flood frequency method as explained below.

Repeated flood volume frequency analyses for various durations (averaging intervals) are required to derive relationships expressing flood flow as a function of duration for a range of ARI's. These relationships define the average flood flow, and hence inflow volume over a particular time period. For a particular ARI, the synthetic inflow hydrograph is constructed from nested flow volume versus duration pairs derived from these relationships. The time of peak (hours from the start of the storm) is determined from an analysis of historical flood hydrographs and selecting a typical value.

As this method relies on flood frequency analysis, it requires a reasonably long and reliable flow record to determine the required flood and volume frequency distributions. The Wairoa at Gorge/Irvine flow record spans over 52 years (1957 to 2009) and was considered representative and appropriate to use in developing the synthetic hydrographs at the dam site. There is a flow recording station near the dam site, Lee above Waterfall Creek, which was established in April 2007 specifically for this project. The 2 years of flow data available from this station demonstrate an excellent correlation between the flows in the Lee and Wairoa Rivers. See Figure 7-2 which overplots the two records; the Wairoa River flows (red) have been factored downwards by the approximate ratio of its mean flow to that of the Lee River (blue).

The average flood flow versus averaging interval relationships for the Wairoa River at Gorge/Irvines are shown in Figure 7-4.

The flood inflow hydrographs were initially developed for the Wairoa at Gorge/Irvines site and then scaled down to represent the corresponding flow hydrographs for the dam site. A scaling factor (0.239) was used, which is based on the ratio of the catchment area at the dam site (77.5 km²) to that at Wairoa at Irvines (464 km²) raised to the power 0.8 which is in accordance with the recommended procedure in the 1990 Hydrology Centre publication "Flood Frequency in New Zealand"(McKerchar and Pearson 1990).

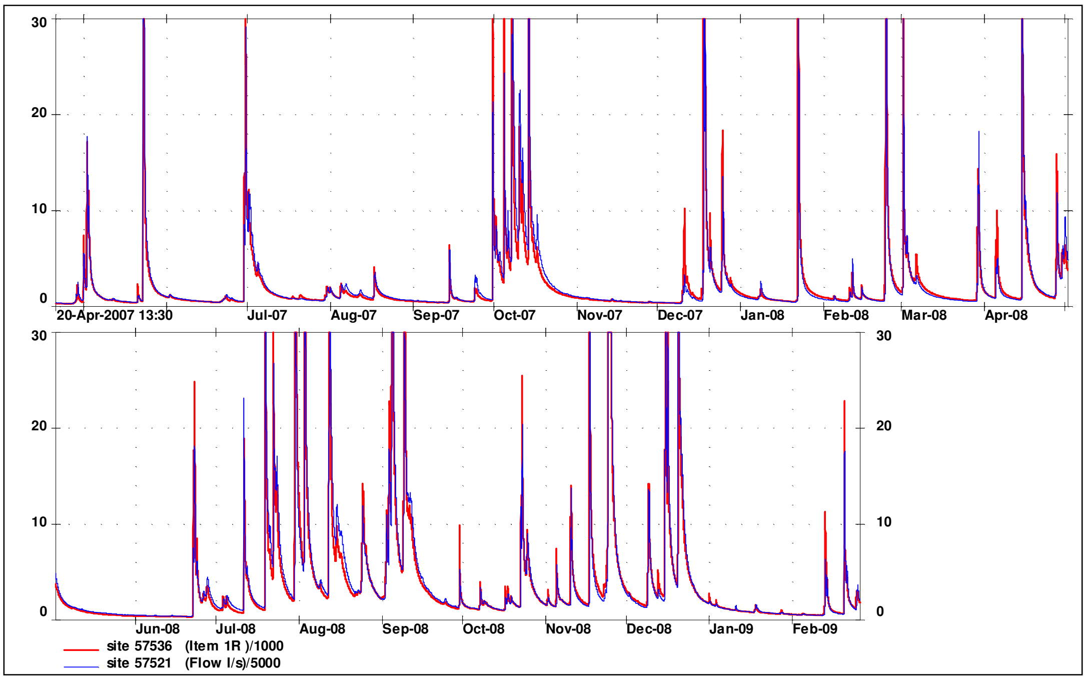

Figure 7-2: Lee above Waterfall Creek flows plotted against Wairoa at Irvines flows (factored downwards) from April 2007 to February 2009

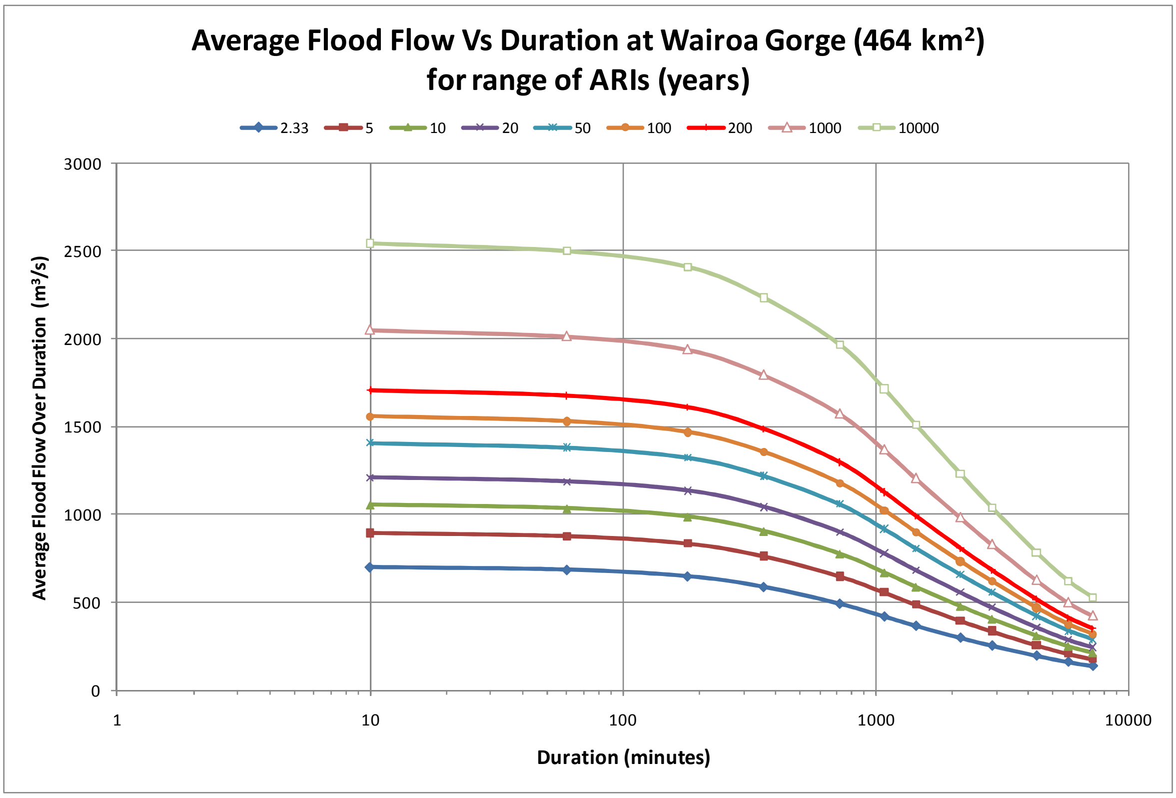

Figure 7-3: flood flow versus averaging interval for a range of ARIs

7.2 Rainfall to runoff modelling

A conventional catchment rainfall-runoff modelling approach has been applied for the Lee River dam site to compare and validate the flood inflow hydrographs derived from the flood frequency method described in the previous section. The catchment model, which has been calibrated using a series of recorded storm rainfall and flood hydrograph data for the Lee River, has also been used to generate a Probable Maximum Flood from the Probable Maximum Precipitation event. See Section 7.3.

HEC-HMS (Hydrologic Modelling System developed by the US Army Corp of Engineers) has been used to model the catchment response to storm rainfall and hence produce appropriate design hydrographs. The key parts to this process were (1) calibrating the model using the Lee (above Waterfall Creek) flow record and rainfall records available for the Lee River and wider catchment, (2) adjusting the calibrated parameters to represent the smaller, but similar adjacent catchment (Waterfall Creek) which also contributes to the flow at the dam site, and (3) combining the two flow records to represent the flow hydrograph at the dam site.

Three storms were used to calibrate the model viz. 23 May 2007, 22 January 2008 and 24 November 2008. The last calibration event (24 November 2008) was the largest flood event of the three and had an estimated return period of 14 years. Sample calibration results are shown in Figure 7-4 and Figure 7-5. The comparison demonstrates a reasonably good fit between the actual and predicted flow at the Lee above Waterfall Creek recording station. This shows that the calibration has been successful and that the calibrated model may be used to reliably compute the design flood hydrograph from a design rainstorm.

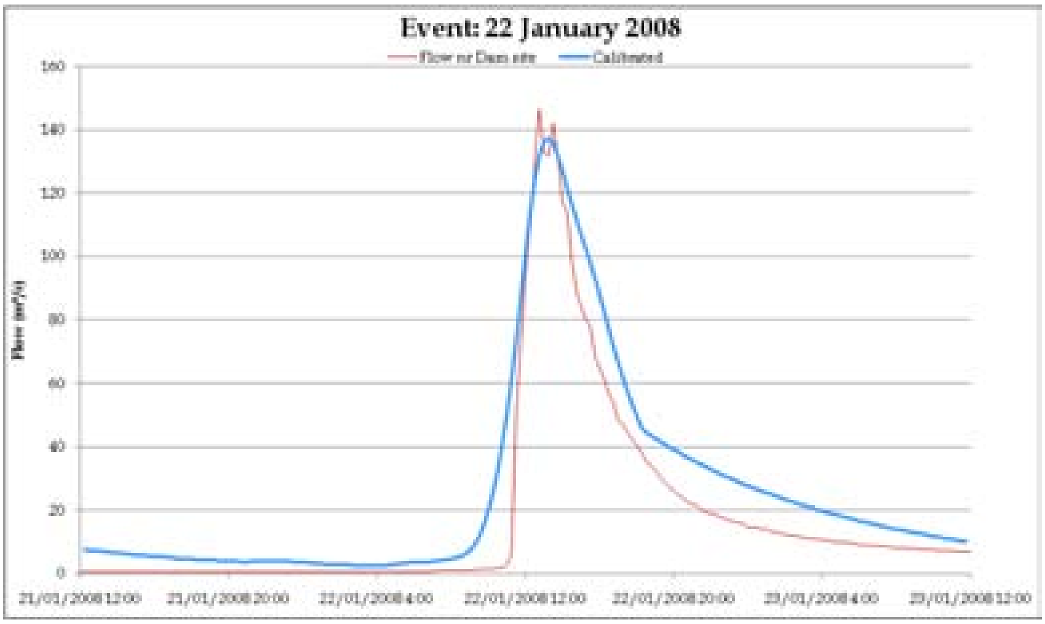

Figure 7-4: Calibration results for rainfall event on 22 January 2008 in the Lee River

The calibrated catchment model was also used to provide an independent check against the synthetic inflow hydrographs derived from flood frequency analysis described earlier. Figure 7-6 shows a comparison between the model hydrograph generated using the HEC-HMS catchment model for the critical storm event (48 hour) and the synthetic flow hydrograph from flood frequency analysis. The inflow hydrograph derived from the catchment model (which is considered the conventional approach) is comparable with the synthetic flow method in terms of both peak flow and flow volume. Note that the time to peak of the synthetic inflow hydrograph has been adjusted to provide a closer match to the shape of the HEC-HMS model hydrograph while retaining the same volume-duration characteristics.

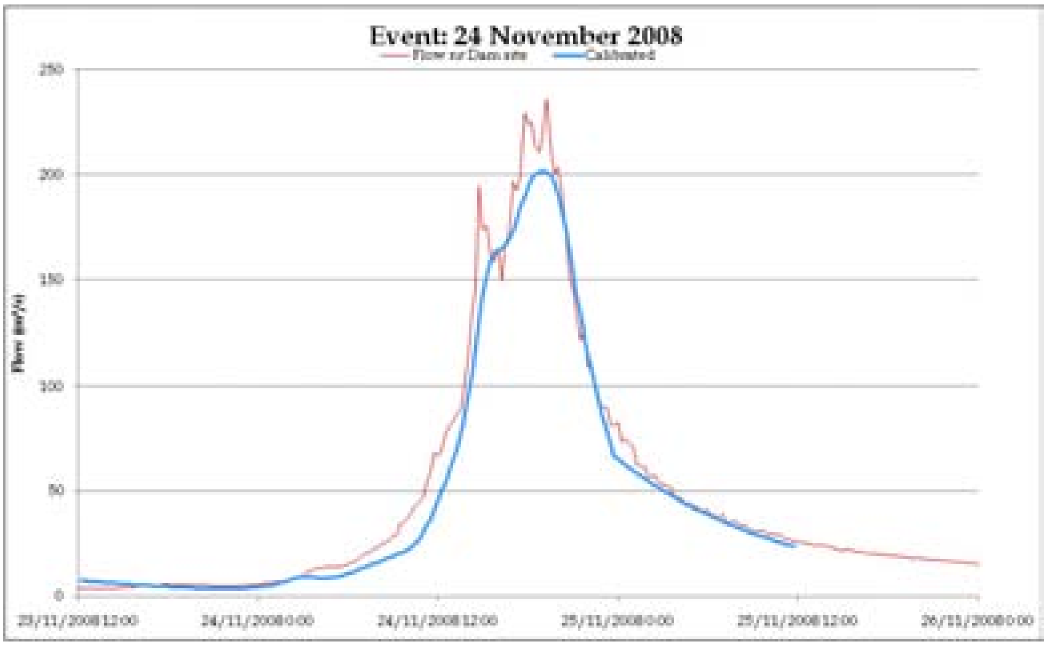

Figure 7-5: Calibration results for rainfall event on 24 November 2008 in the Lee River

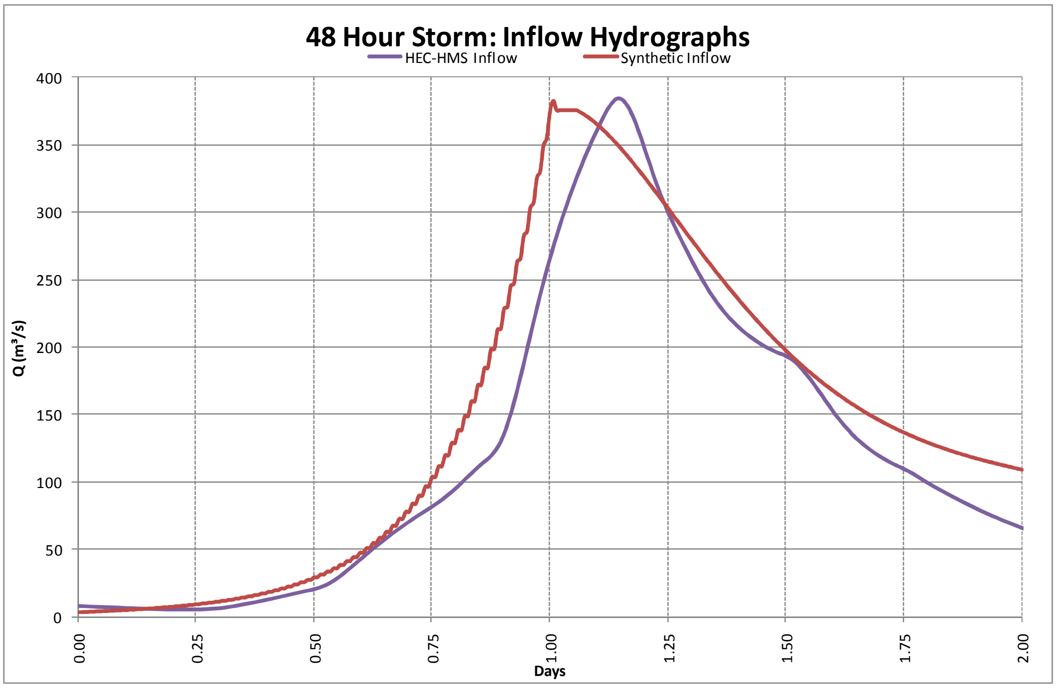

Figure 7-6: Comparison of the HEC-HMS model hydrograph generated using HEC-HMS (critical 48 hour storm) and the synthetic flow hydrograph from flood frequency analysis

7.3 Probable maximum flood

A Probable Maximum Flood (PMF) for the dam site (Figure 7-7) has been developed. This PMF has been computed from the Probable Maximum Precipitation (PMP) assessed for the catchment at the dam site using the 1995 NIWA approach, "A Guide to Maximum Precipitation in New Zealand" (Thompson and Tomlinson 1995). The calibrated catchment rainfall-runoff model described in the previous section was used to generate the PMF from the critical 24 hour duration PMP.

The peak PMF inflow at the dam site is estimated to be 1094 m^3/s and the 48 hour flood volume 57 million m^3.

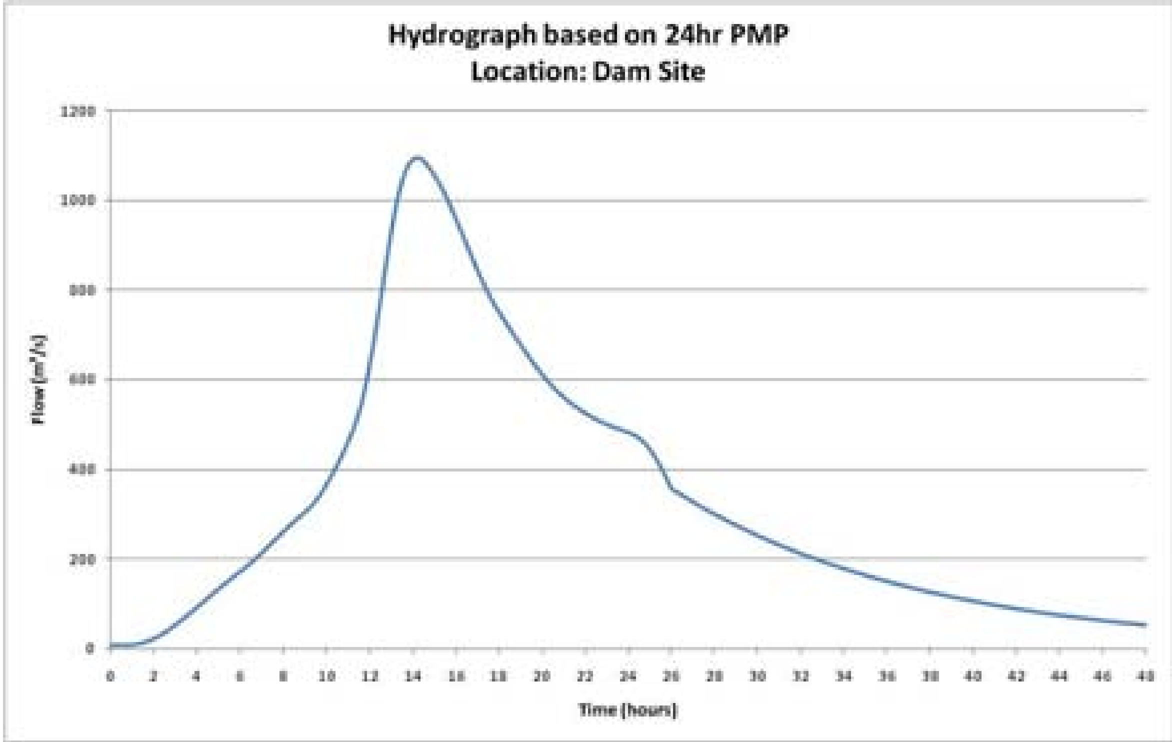

Figure 7-7:PMF Hydrograph for the Lee Dam site

7.4 Further Details

Further details of the derivation of the synthetic inflow hydrographs, the associated flood frequency analyses, the storm rainfall analysis, and the derivation of the PMP is found in the accompanying Hydrological Assessment.

8 Wave Environment

An assessment of the potential for waves generated in the reservoir is necessary to consider what their effect could be on the embankment, with regard to both erosion and to overtopping. This is especially important when the water level in the reservoir is elevated above normal levels during flood passage. It has implications for both dam safety and shoreline effects and therefore the extent of land affected.

Waves can be generated by wind action across the reservoir, or by seiche effects associated with landslides into the reservoir, or seismic action and reservoir response. The following sections provide estimates of the potential windspeeds, wind generated wave/run-up heights, and a discussion of waves associated with seiches.

8.1 Potential Wind Speeds

Estimations of extreme wind speeds were obtained from the New Zealand structural design actions code for wind loads (AS/NZS 2002) and converted to mean 1 hour wind speeds via empirical methods contained in the USACE Coastal Engineering Manual (2002).

While the maximum straight line fetch to the dam is 1027m from the south, the effective fetch is limited by the surrounding land and the irregular shoreline. An effective fetch of 439 m was calculated using the method developed by Saville et.al. (1962). The most significant fetch direction is from the south and the mean 1 hour wind speeds for return periods of 1 in 10, and 1 in 100 years in this direction are presented in Table 8-1. These return periods have been selected to match the design loading combinations, as discussed in Section 5.3.

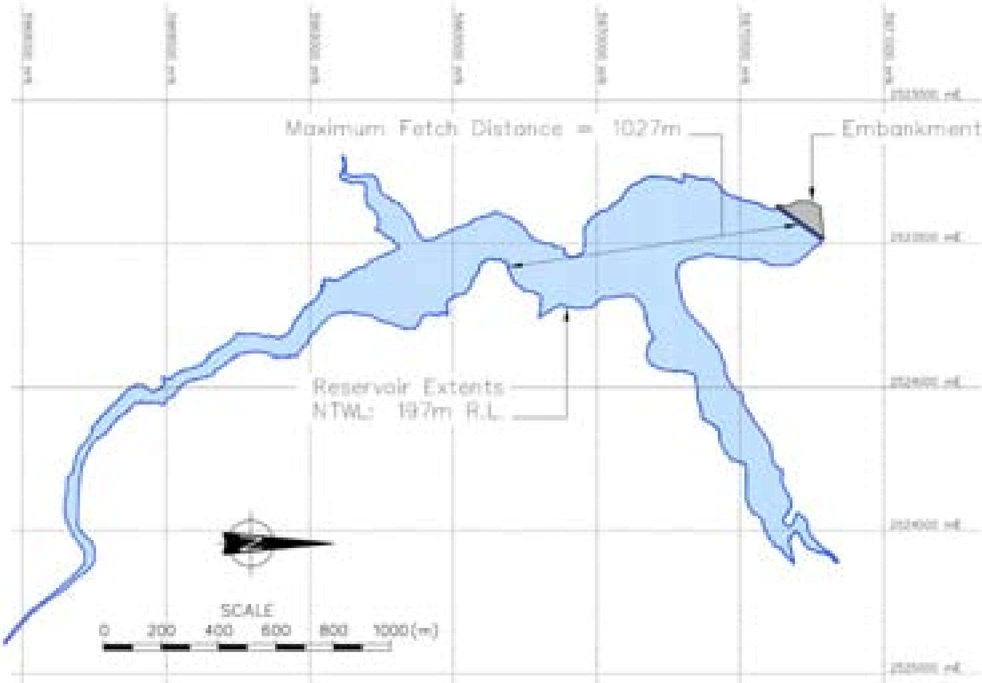

Figure 8-1: Most significant reservoir fetch

Table 8-1 - Design Wind Speeds

Return Period (years)	Mean Wind speed (m/s)
10	30.6
100	36.9

8.2 Wind generated waves

The wave climate was assessed using theory based on Young & Verhagen (1996). The extreme fully developed significant wave heights and hydrodynamics were calculated for the dam site assuming depth and fetch limited conditions and wind speeds as evaluated in Table 8-1.

The main processes that have potential to affect the dam face are wind generated waves and wave run-up. Rock armour is typically used to protect the face of an earth dam and can serve to absorb some of the wave energy. As the Lee Valley Dam has been designed with a concrete facing, it will absorb less wave energy resulting in more reflection and run-up than a rock armoured face. While rock facings are typically placed at slopes close to 1V:2H, the concrete face will (nominally) be built at 1V:1.5H and as a result, will experience higher run-up.

Run-up is defined as the height above the still water line that is exceeded by 2% of the incoming waves. Run-up was calculated using the methods developed by Delft Hydraulics and reported by van der Meer (1988) & (1992) and incorporated in the method used by the USACE (2002). The method was developed from long crested wave data impinging head on to an impermeable slope. The run-up is dependent on the significant wave height, wave properties and the slope of the dam. Significant wave heights (Hs), Peak Period (Tp) and wave run-up above still water level at the dam face for the significant wave height and the highest 2% and 0.1% of waves (Rs, $R_{2\%}$ and $R_{0.1\%}$) are presented in Table 8-2. Figure 8-2 provides an illustration of the wave climate.

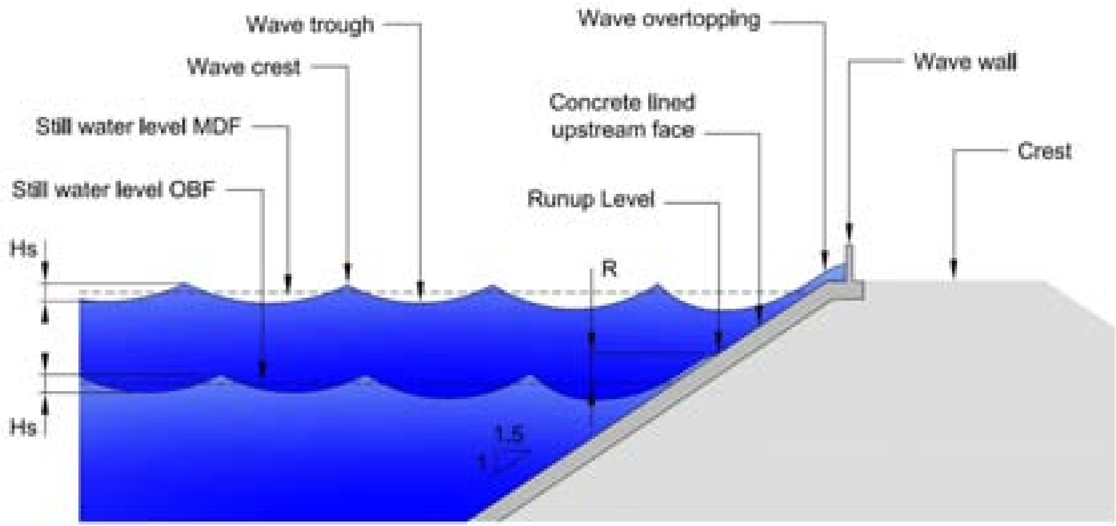

Figure 8-2 - Wave Climate

Table 8-2 - Design Wave Climate at Dam Face

Return Period (years)	Hs (m)	T_p(s)	R_S (m)	$R_{2\%}$ (m)	$R_{0.1\%}$ (m)
10	0.228	1.838	0.291	0.449	0.583
100	0.277	2.007	0.373	0.545	0.730

Wave walls can be employed to protect an embankment against wave overtopping and erosion and are an economic means of providing adequate freeboard where the alternative is to provide additional embankment height, although wave run-up is not particularly high in this case due to the relatively short fetch. The effect of a wave wall on the run-up and its effect in preventing overtopping were investigated.

With the inclusion of a vertical wall, the surf similarity parameter becomes very high and run-up equations are typically not applicable. Instead, empirical data is used to estimate the overtopping discharge per metre of wall in a given wave climate. The crest and wall configuration can then be determined for an allowable overtopping discharge.

The Lee Valley Dam has been assumed to have a still water level 0.5 m below the crest with a crest wall 1.0 m high set back 0.5 m from the upstream crest. The wave overtopping was then calculated using methods outlined in the USACE Coastal Engineering Manual (2002), Chapter 5.

The adopted tolerable discharge over the wall is 1x10-6 $m^3/s/m$. This was based on recommendations presented in "Wave overtopping of seawalls design and assessment manual" (R&D technical report W178 1999) for “No Damage” to buildings. Tolerable discharge rates are higher for “No Damage” to embankment seawalls or revetment seawalls. Overtopping limits have traditionally been specified in terms of mean discharge rates. The maximum individual event is expected to result in higher short term discharge rates.

The maximum discharge rates calculated for this configuration and wave climate ranged from 1.3×10^{-7} to 9.1×10^{-10} and well within adopted tolerable rates.

8.3 Seiche effects

The effects of seiche generated from earthquake shaking have not been specifically considered at this feasibility stage. However, they should be considered in detail at the detailed design stage. The freeboard allowance (NTWL to crest) is comparable with or higher than that for other dams in the region, which is expected to provide sufficient protection for feasibility stage costing.

No large landslides that could generate significant seiche effects have been identified in the reservoir area. This assessment should also be revisited at the detailed design stage.

9 Seismic Conditions

Current dam safety standards (NZSOLD 2000) require an assessment of the response of a dam and its associated structures to seismic events. The extent of the assessment is dependent on the Potential Impact Category (PIC) of the dam and its location.

To undertake any assessment of seismic effects, an appreciation of the site's seismicity is required. The Lee Valley Dam site is located roughly 20 km south of Nelson and 19 km from the Wairau Fault. The PIC of the Lee Valley Dam is assessed as High and, in accordance with the New Zealand Dam Safety Guidelines (NZSOLD 2000), the dam and any critical structures should be analysed for a Maximum Design Earthquake (MDE) of between 1 in 10,000 year annual exceedence probability (AEP) event, and the maximum credible earthquake (MCE). A site specific seismic assessment should be undertaken for this dam at the detailed design stage. For the current assessment the code AS/NZ 1170 provides a standard procedure for estimating the ground response based upon ground conditions and the AEP as seen in Figure 9-1.

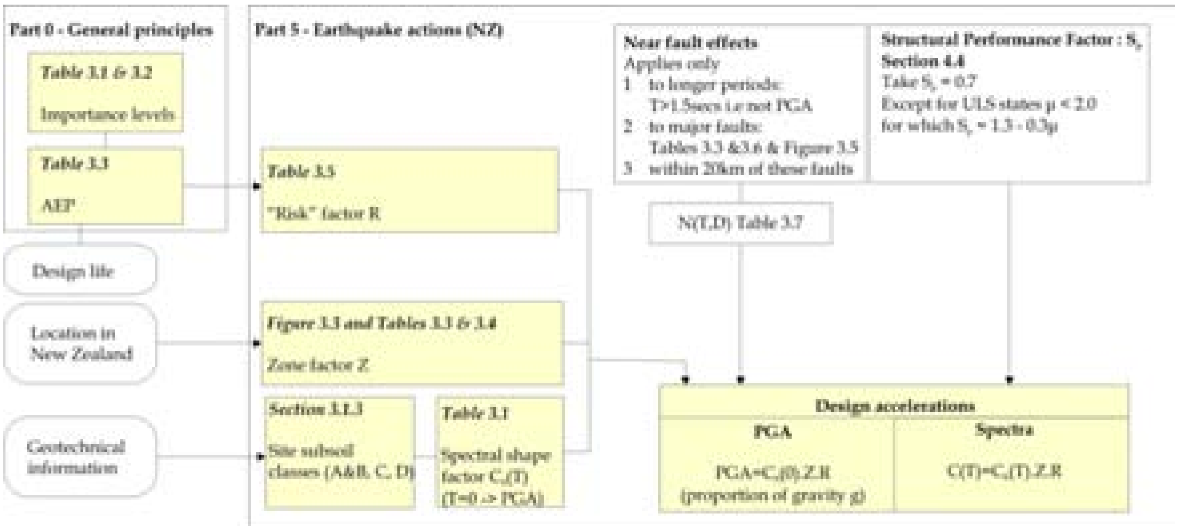

Figure 9-1: Procedure for assessment of seismic conditions AS/NZS 1170.1:2002

The four parameters required to determine peak ground acceleration (PGA) and the elastic site hazard spectrum from the code were as follows:

- Spectral shape factor, Ch(T) = 1, based on a subsoil class A (Rock).
- Hazard Factor, Z = 0.32, from Figure 3.4 of the code (AS/NZ 1170) (Figure 9-2).
- The return period factor, R, was taken to be 150 yrs for the operational base earthquake (OBE). An R factor is not given in the code for AEP events greater than 2500 yrs. However, there is a maximum required value of the ZR product (0.7) applicable to areas close to the Alpine Fault. Figure C3.3 of the NZS 1170.5 Supplement (1:2004), shows a comparison of R factors for various New Zealand locations and the code adopted R-factor values. Using this figure, the R factor for this site for a return period of 10,000 yrs is estimated to be > 2.2. Therefore the ZR product will be limited to 0.7.
- The near fault factor, N(T,D) = 1, for a period of 0 seconds, and a distance of 19 km to the nearest major fault.

Using the above factors, the PGA for the OBE and MDE were 0.187g and 0.7g respectively as presented in Table 9-1.

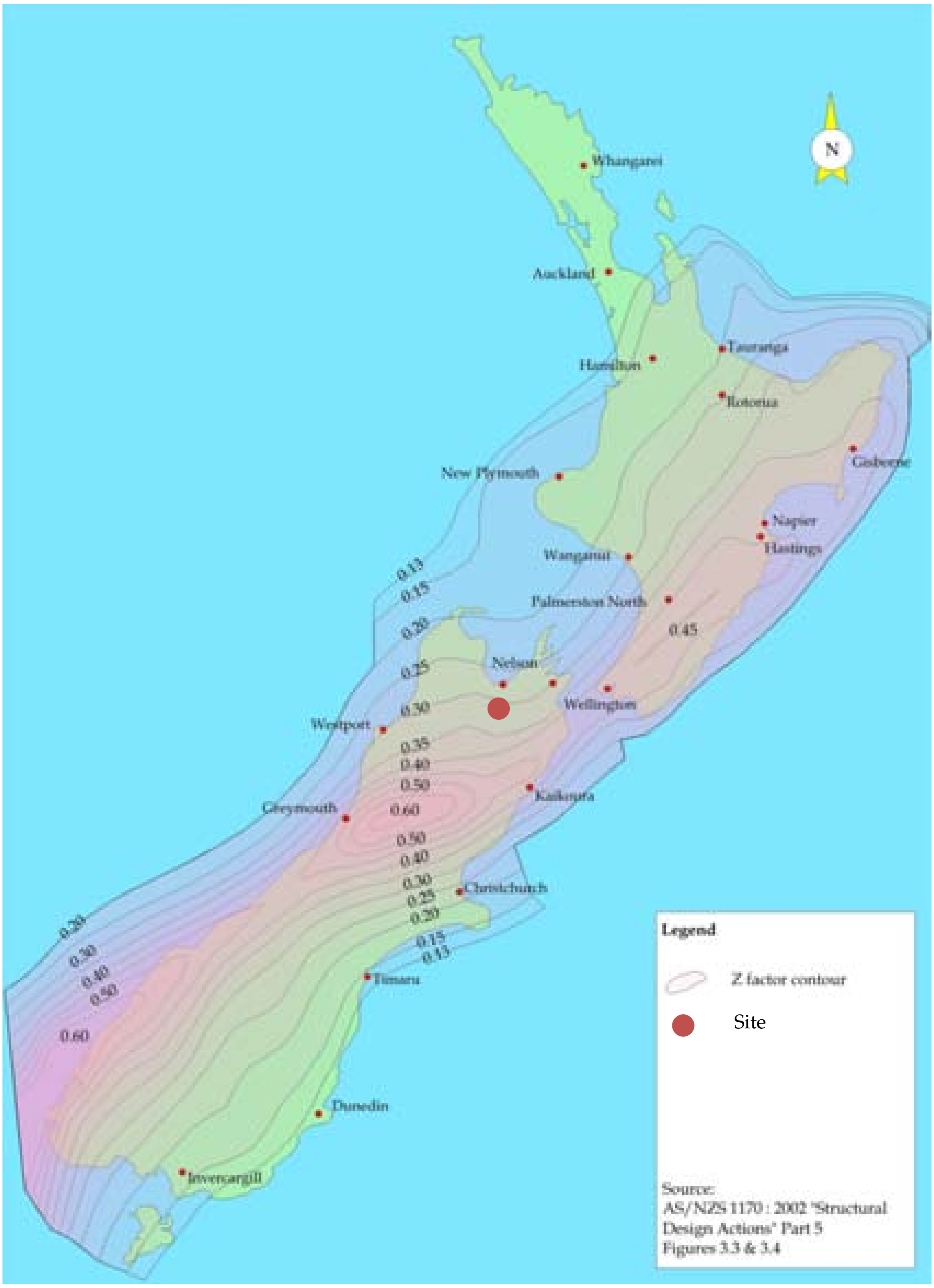

Figure 9-2: Seismic Hazard. AS/NZ 1170 Figures 3.3 & 3.4

Table 9-1: Peak ground accelerations

Earthquake	Return Period	PGA
OBE	150 Years	0.187g
MDE	10,000 Years	0.7g

Full elastic spectra were also developed using the code. AS/NZS 1170 assumes a 5% damped spectra however additional spectra were developed for damping ratios of 2%, 10%, 15% and 20% using the relationship developed by K.Kawashima, K. Aizawa and K.(1984). The full range of damped spectra for the site are plotted in Figure 9-3 and Figure 9-4.

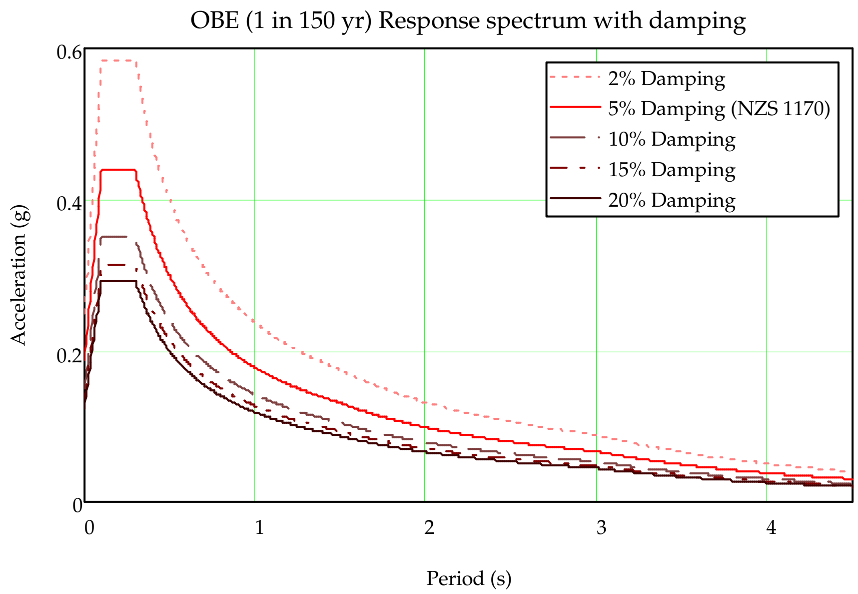

Figure 9-3:OBE (1 in 150yr) Response spectrum with damping

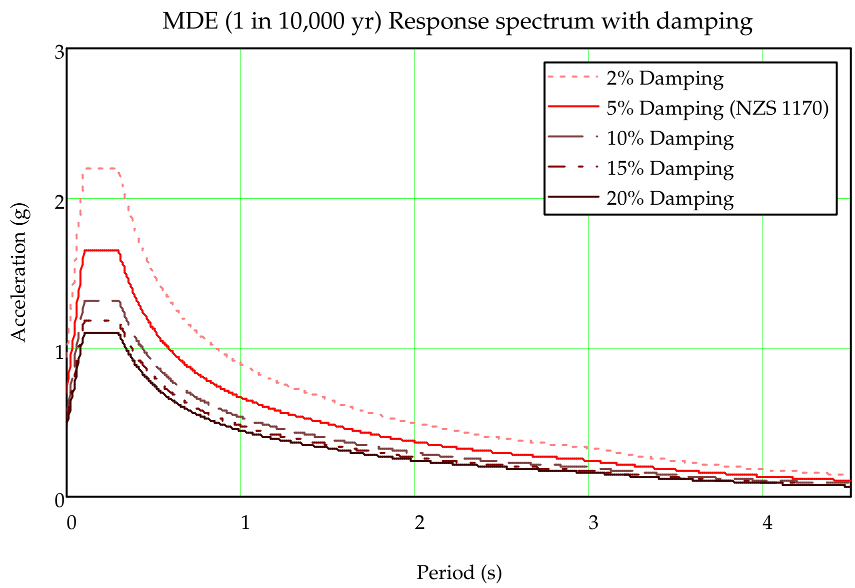

Figure 9-4: MDE (1 in 10,000yr) Response spectrum with damping

10 Dam Arrangement Options

10.1 General

The arrangement of a dam includes an almost infinite number of combinations of spillway type, embankment type, freeboard allowance, and outlet systems. For example, if a large freeboard is provided between the crest of the spillway and the crest of the embankment, more flood water can be temporarily stored in the reservoir and the flow capacity of the spillway can be reduced. This will result in a lower cost spillway, but a more expensive embankment. The material that is excavated from the spillway area may be used in embankment construction if the quality is suitable. Depending on the volumes required for embankment construction this may make spillway excavation very inexpensive, and lead to a large shallow auxiliary spillway being attractive. In addition, works which have been used for temporary flood diversion during construction may be able to form part of a permanent spillway system.

A change in each component of a dam clearly affects a lot of other components, and the optimum arrangement can be elusive. The limitations in available information at the feasibility design stage also need to be recognised, and the potential for necessary changes during the detailed design stage to address an issue which is currently not apparent needs to be appreciated.

We have undertaken a preliminary/scoping level design and cost assessment for a range of combinations of embankment height (available freeboard), spillway type and size, and rockfill material quality. This process has developed curves of approximate cost for embankment and spillway components and options, with a primary aim of selecting the embankment crest level, and the spillway type and size. While the cost curves developed do not include all the components of the dam, they do include costs which are specific to individual options allowing reasonable comparisons to be made. As such these curves should be considered a ranking process rather than development of absolute construction costs.

10.2 Options considered

Details of the options assessment are included in Appendix A. In summary, the following options were included as part of the assessment:

Embankment types:	Zoned earthfill
	Concrete faced rockfill
	Roller compacted concrete
Spillway types:	Combination primary and auxiliary spillway
	Ogee weir primary
	Labyrinth weir primary
	Bell-mouth primary with dropshaft

10.3 Spillway and embankment selection

Total cost index curves show an economic embankment height at RL 202 m for all options with the exception of the bell-mouth spillway. The bell-mouth spillway option indicates that a lower embankment (and larger diameter spillway crest) is more cost effective. The curves indicate that a bell-mouth spillway is overall the most cost effective option, but this is only marginally so for the case of good quality rock. The cost effectiveness of the bell-mouth stems from use of construction diversion culverts as the outlet system.

If good quality rock is assumed it appears that after the bell-mouth spillway, the ogee weir and chute for configuration A at RL 202 m is the next most economical option. For the poor rock case, the next best option is a labyrinth weir at RL 202 m.

This outcome is understandable as the ogee weir spillways generate the most cut and therefore an economical embankment when the rock is good, while the labyrinth weir is narrower and produces less cut to waste if the rock quality is poor.

A labyrinth weir option in poor rock will cost more than an ogee weir built in good rock. The difference in cost between both ogee and labyrinth in poor rock is small.

A bell-mouth spillway introduces significant disadvantages for passage of native fish, and likely significant operational difficulties associated with accessing the outlet works, as these may need to be shared with spillway passage. Due to its height, the outlet tower's resistance to seismic forces will have a large impact on the cost. The likely hydraulic interaction of the spillway with any outlet tower works would also disadvantage this option.

Based on the work presented in Appendix A and subsequent consideration, the following parameters have been adopted for preliminary design and costing:

- embankment crest at RL 202m.
- ogee weir (adjacent to embankment centre line) with chute primary spillway
- auxiliary spillway with fusible embankment 19.5 m wide adjacent to the ogee weir and discharging to Anslow Creek
- construction diversion consisting of 3, 2.5 m x 5 m square box culverts with separate upstream coffer dam with crest at RL 163 m
- outlet tower with outlet via steel pipe housed in diversion culvert.

An outline arrangement for the preferred option is provided in Figure App 13. Note this figure is simplified in that it does not include requirements for vehicle access or many of the details of the dam, but is included to show the development of the design. More detail is developed in Section 11.

11 Arrangement of Selected Dam

11.1 Summary and specifications

A summary of the arrangement and specifications for the selected dam and spillway are listed below in Table 11-1.

Table 11-1: Summary and Specifications

Embankment Characteristics		
Normal top water level (NTWL)	RL 197	m
Embankment type	Concrete faced rock fill (CFRD)	
Crest elevation	RL 201	m
Maximum flood water level	RL 201.58	m
Maximum dam height (from riverbed to dam crest)	52	m
Crest length	220	m
Wave wall height	1	m
Spillway Characteristics		
Total peak outflow OBF	372	m^3/s
Total peak outflow MDF	1036	m^3/s
Primary spillway type	Ogee Weir	
Primary spillway width	22.3	m
Peak outflow OBF for primary spillway component	372	m^3/s
Peak outflow MDF for primary spillway component	449	m^3/s
Auxiliary spillway type	Fuseable Embankment	
Auxiliary spillway width	19.5	m
Peak outflow MDF for auxiliary spillway component	606	m^3/s
Spillway Chute and Energy Dissipation Characteristics		
Chute length (plan)	105	m
Chute width, wide section	22.3	m
Chute width, narrow section	10	m
Chute minimum wall height	4	m
Dissipation type	Flip Bucket	
Flip bucket radius	25	m
Bucket lip level	RL 156.58	m

Outlet Characteristics	
Number of outlets	2
Outlet type	Sloping outlet conduits on upstream face with removable screens and gate control.
Outlet level - Upper	RL 185 m
Outlet level - Lower	RL 167 m
Control gate type	Radial
Control gate size	1 x 1 m
Conveyance conduit size (under embankment)	2.5 x 5 m
Number of conveyance conduits	2 (access via third)
Conveyance conduit downstream protection	Stoplogs

11.2 Embankment arrangement

11.2.1 General

This section discusses the embankment arrangement that has been adopted, and provides a basis for the preliminary design features. A Concrete Faced Rockfill Dam (CFRD) has been selected, which uses a concrete slab on the upstream face as a waterproofing element.

Linking the upstream face into the foundation is clearly a critical component of this type of dam. This is achieved by the plinth which is a concrete slab cast against the prepared foundation surface and tied to the foundation with grouted reinforcing bars. Grout is also injected into the foundation where necessary to reduce leakage to acceptable amounts.

The internal zoning of the dam is arranged to minimise settlements of the upstream face during first reservoir filling, and to manage leakage in the unlikely event that cracks form through the upstream concrete face. The zoning also makes most economical use of the materials which are available locally at the dam site, and preferably from excavations required for the spillway and other related activities.

The arrangements for the embankment are shown in the following figures (Appendix B) which should be read in conjunction with this section:

- Figure B-01 Dam General Arrangement
- Figure C-01 Embankment Cross Section
- Figure C-02 Embankment Outline Details.

The following sub-sections provide an assessment of these factors.

11.2.2 Foundation treatment

The dam foundation is formed by in-situ rock of various weathering grades, and overlay of soil-like materials placed through alluvial or colluvial action. The foundation materials are discussed in detail in the Geotechnical Report and have been simplified into four main units for the purposes of foundation treatment, which are:

- Class 1 insitu rock
- Class 2 insitu rock
- Class 3 insitu rock
- Overburden materials (soil).

The target depth for subexcavation varies across the footprint as different parts of the dam require different quality materials as a foundation. The dam plinth (as discussed in the next sub-section) requires the best foundation to minimise potential leakage. The size of the plinth is related to the foundation quality and for the feasibility design we have located the plinth in Class 1 rock (where possible with reasonable excavation) or Class 2 rock. This has required significant excavation in some areas, especially at the left abutment.

The rock underneath the plinth will also require grouting to minimise leakage. This is discussed further in the next sub-section.

The foundation under the body of the dam has a lower requirement for quality. The main objective in this area is to remove material which could result in additional settlement of the embankment, or form weak planes (shear surfaces) under the embankment. Allowance has been made to strip all overburden materials down to the top of rock under the body of the dam.

A contour plan showing the fully excavated profile for the dam (and spillway excavations) is shown on Figure B-02. Figure B-03 shows isopachs of excavation depth below existing ground level, and a summary of the volumes of the various materials forming the overall excavation.

11.2.3 Plinth arrangements

The plinth consists of a concrete cap or blanket upstream of the heel of the dam that forms a leakage resistant joint between the concrete upstream face, and the foundation rock. It includes the actual concrete cap and any grouting/remedial works in its region. The functions of the plinth are to:

1. Provide the main barrier to water flow through the foundation rock from the reservoir.

2. Prevent erosion of the foundation rock due to water flow and seepage gradients in the rock.

3. Provide a base for construction of the concrete face, and a waterproof connection of the face to the foundation.

Several papers provide some guidance on the design principles for the plinth. These appear to be largely based on precedent for what has and has not worked, in terms of limiting seepage through the foundation to acceptable levels. Some papers provide guidelines on the relationship between foundation rock quality, and the required width of plinth relative to reservoir head (hydraulic gradient).

Experience from HEC Tasmania design and construction of CFRD plinths is summarised as follows:

Rock Quality	Plinth Width	Grouting
Sound rock	0.05 x head	3 rows holes. Outside shallow consolidation grouting and inside deep curtain grouting.
Lesser quality	0.10 x head	

In addition, HEC gave the following recommendations:

- minimum plinth width of 3m
- dental treatment (concrete infill) of faults and highly fractured zones crossing the plinth
- in foundations with highly erodible joint infilling filters can be placed over the foundation downstream of the plinth

The "Guidelines for Design – High Concrete Face Rockfill Dam" (CFRD International Society 2008) provides the following guidelines for plinth design (note a minimum width of 3m is recommended):

Rock Rot Degree	Allowable Hydraulic Gradient
Fresh, weak weathering	>20
Moderate weathering	10 to 20
Intense weathering	5 to 10
Full weathering	3 to 5

A presentation at SANCOLD/US 2005 with outline design parameters for plinths recommended a minimum plinth width of 3m for dams greater than 25m high, and:

Rock Weathering	Allowable Hydraulic Gradient
Fresh	20
Slightly to moderately weathered	10
Moderately to highly weathered	5
Highly weathered	2

The few examples cited above give some background to the requirements for the plinth design. The primary consideration is the hydraulic gradient of water flowing through the foundation rock underneath the plinth, and its relationship to the quality of the rock on which the plinth is founded. The cited examples provide a relatively consistent relationship between acceptable hydraulic gradient and rock quality.

The relationship between plinth width and rock quality allows some optimisation in the balance between depth of subexcavation, and the width of the plinth. This is particularly important for the Lee Valley Dam where the depth to unweathered rock is significant, especially at higher elevations in the abutments, and removal would be extremely expensive. A greater plinth width, coupled with grouting, in these areas may offer a more economic solution.

Accordingly, the following approach has been adopted for feasibility level design of the plinth for the Lee Valley Dam (detailed design will investigate this area in more detail and may result in different subexcavation depths, and possibly a different optimum location for the plinth):

Rock Weathering	Allowable Hydraulic Gradient
Fresh (Class 1)	20
Slightly to moderately weathered (Class 2)	10
Moderately to highly weathered (Class 3)	5
Highly weathered	2

Figure A-02 shows an elevation of the upstream face of the dam. It indicates the levels of the various rock weathering grades, hydraulic head, and estimated plinth width based on the guidelines developed above.

Grouting of the rock is generally required underneath the plinth to reduce permeability and hence leakage. Three rows of grout holes are generally adopted, and this has been applied here. A central line of deep curtain grouting would be placed first, followed by one upstream and one downstream curtain grout lines.

Local imperfections will require treatment to avoid erosion of infilled joints in the region of the plinth, where relatively large hydraulic gradients are present. The joints would be cleaned out and replaced with slush concrete throughout the contact area. In very poor conditions a concrete slab and/or filter layers can be placed over the foundation downstream of the plinth (this additional treatment has not specifically been allowed for at this stage of assessment, but contingency allowance should cover such eventualities).

The plinth slab will be extended up the left abutment and will join into an apron upstream of the spillway. This connection will ensure a continuous barrier to seepage across the dam foundation.

A mass concrete starter dam has been included at the upstream toe of the embankment, which will also form the plinth and will have a perimetric joint at its crest to connect with the concrete upstream face. This feature has been included to assist with river/flood diversion during construction as discussed more fully in Section 12.

11.2.4 Internal zoning

The internal zoning of the dam serves a number of objectives, including:

- using the available materials to the most economic effect
- control settlement of the dam to amounts that will not cause distress of the upstream concrete face
- allow seepage flow through the dam body without the formation of a high phreatic surface, both in the case of normal operation and if cracking forms in the upstream face allowing larger leakage
- provide a bedding layer for formation of the upstream face
- provide stability against static and seismic loadings.

These objectives are sometimes conflicting and a compromise must be reached in developing the internal zoning.

The most upstream zone is the face slab and is constructed from reinforced concrete. More detail is provided on assessment of the upstream face in section 11.2.5.

A semi pervious zone (Zone 2A) is provided immediately downstream of the concrete slab. This is a processed rockfill or alluvium, grading from silt to cobble or gravel size. The zone provides uniform support for the face slab and acts as a semi-impervious layer to restrict flow through the dam in the event that cracking of the face slab or opening of joints occurs. A zone width of 1 m has been adopted for this evaluation.

Zone 2B (downstream of Zone 2A) is a selected fine rockfill which acts as a filter transition between Zone 2D and Zone 3A in the event of leakage through the dam. A zone width of 5 m has been adopted for this evaluation.

The two main rockfill zones are termed Zones 3A and 3B. Zone 3A is under the upstream face and is formed from higher quality (and hence lower compressibility) rockfill. This zone forms the main support for the rockfill and is critical in limiting settlements during the critical first filling stage.

Zone 3B is also rockfill, but can be formed from lower quality (higher compressibility) material. Ideally the zone should be highly permeable to assist with drainage, and the lower quality is generally achieved through less compaction effort. In this case lower quality rock has been used in this zone to optimise materials available on site. Zone 3A has been extended underneath Zone 3B to maintain drainage paths through the embankment.

Embankment settlement is a primary consideration in selecting the rockfill zoning, and materials that can be used in the zones. Settlement of the embankment has been evaluated for a number of material zoning cases and compared with historical CFRD performance. This aspect is discussed in section 11.2.6.

The zoning arrangements adopted for the embankment for this study are shown in Figure C-01. If the permeability of the rockfill materials proves to be lower than expected when initial trials are undertaken (possibly due to particle breakdown), another zone may need to be introduced to act as a high permeability blanket and drain, connecting the upstream shoulder to the downstream face. Screened alluvial gravels would be the best source for such a zone.

The transition zones (2A and 2B) are likely to be able to be sourced from screening of alluvial deposits upstream from the embankment in the base of the river valley.

Preliminary analysis of settlement and stability (sections 11.2.6 and 11.2.7) indicate that rockfill Zone 3A can be sourced from Class 1 rock excavations and rockfill Zone 3B can be sourced from Class 2 and Class 3 rock excavations. If the quality of these rock sources proves to be lower than currently expected when more detailed investigations are undertaken, Zone 3A can be sourced from alluvial gravel borrow in the river bed upstream from the dam, and Zone 3B can be sourced from Class 1 and Class 2 rock excavations. The approximate volume balances are shown in Table 11-2.

.

Table 11-2: Cut and fill balance

Total Cut and Borrow Volumes					
Cut/Borrow Type	**Class 1**	**Class 2**	**Class 3**	**Overburden**	**Alluvial Borrow**
Cut to Fill Volume (cu.m)	88,000	83,000	68,300		
Cut to Waste			110,000	122,000	
Borrow to Fill					166,800

Fill Volumes						
Embankment Zone	**Volume (cu.m)**	**Fill Type (cu.m)**				
		Class 1	**Class 2**	**Class 3**	**Overburden**	**Alluvial Borrow**
1A	2,100	-	-	-	-	2,100
1B	7,300	-	-	7,300	-	-
2A	7,200	-	-	-	-	7,200
2B	30,500	-	-	-	-	30,500
3A	215,000	88,000	-	-	-	127,000
3B	144,000	-	83,000	61,000	-	-
Fill Total	406,100	88,000	83,000	68,300	-	166,800

11.2.5 Upstream concrete face

Established design procedures have been adopted for sizing and detailing of the upstream concrete face. These procedures are from a range of papers and documents, largely summarised by Cooke and Sherard (March 1987).

A thickness of 0.25m has been adopted for the face. This has been found satisfactory for low to medium dams (75 to 100m) and is about the minimum that can be adopted and still provide sufficient cover to internal reinforcing steel.

The compressive strength of the concrete is of relatively low importance. Durability and impermeability are of higher importance. A 28 day compressive strength of 20 MPa is often adopted, with maximum aggregate size of 38mm, and pozzolan/air entraining to minimise permeability.

Reinforcing with steel/concrete ratio of 0.4% has been adopted, with 20mm diameter bars placed centrally and running horizontally and upstream/downstream.

Joints have been allowed at the perimeter of the upstream face intersection with the plinth (perimetric joint), and vertical joints at 15m intervals to allow slip forming of the upstream face. Outline details for these critical joints are shown in Figure C-02.

11.2.6 Settlement

Embankment settlement is one of the most important considerations with a CFRD. The general construction methodology for a CFRD (refer to section 12 for more detail) involves placement of rockfill first, followed by placement of the concrete upstream face.

This staging allows settlement of the rockfill during its construction to take place prior to placement of the concrete face. This reduces deformation and loading of the upstream face significantly.

The modulus of the rockfill in the embankment is critical in determining the settlement as the reservoir loading is applied during first filling. Modulus values for the available rock sources have been estimated and recommended in the Geotechnical Report based on the rock type and quality, and reference to historical measurements/correlations.

For this feasibility study a sensitivity analysis was undertaken to assess the upstream face deformation on first reservoir filling with a range of materials assumed in the embankment. Estimated deformations were then compared to historical deformation as measured on a number of CFRD's, and reported by Hunter and Fell (2002). The relevant raw slab deformations from this study have been normalised based on embankment height to allow comparisons with modelled estimations. The normalised historical deformation results are shown on Figure A-01.

An analysis was carried out using GeoStudio, with the embankment only modelled. The foundation was assumed relatively stiff (refer to Geotechnical Report which indicates foundation stiffness approximately an order of magnitude higher than the embankment material). The model was built up in stages so that embankment stresses and deformations could be developed with time. Deformations were reset to zero, and a load representing the reservoir applied to the upstream face. The face deformations were noted, plotted, and normalised by embankment height to allow comparison with historical values.

Eight cases of rockfill material were considered, as shown in Table 11-3. The Poisson's ratio adopted for the rockfill had a significant effect on results. Various authors indicate Poisson's ratio between 0.0 and 0.3 are appropriate for rockfill in a CFRD on reservoir first filling.

Table 11-3: Rockfill Cases and Poisson's Ratios

Case Name	Rockfill in Zone 3A	Rockfill in Zone 3B	Poisson's ratio
Case A	From Class 1 rock	From Class 1 rock	0.30
Case B	From Class 1 rock	From Class 1 rock	0.05
Case C	From Class 3 rock	From Class 3 rock	0.30
Case D	From Class 3 rock	From Class 3 rock	0.05
Case E	From alluvium	From alluvium	0.30
Case F	From alluvium	From alluvium	0.05
Case G	From Class 1 rock	From Class 3 rock	0.30
Case H	From Class 1 rock	From Class 3 rock	0.05

The model geometry, rockfill modulus values, and analysis results are shown on Figure A-01. The results indicate that expected face deformations for all of the cases considered (and listed in Table 11-3) are well within the range of deformations experienced by historical dams. This indicates that, based on the modulus values estimated for the rockfill materials from excavations, face deformations would be acceptable for any of the rockfill materials. This conclusion led to the development of embankment zoning discussed in section 11.2.4.

11.2.7 Stability

CFRDs traditionally do not have stability analyses carried out to assess their static or dynamic stability. The rationale for this is that provided the slopes are flatter than the angle of repose of the rockfill, the embankment is inherently stable. Consideration of CFRDs already constructed of heights considerably larger than Lee Valley Dam, and also in seismically active zones, indicates that the selected batter slopes (1.5:1) are conservative. These slopes should be reviewed during detailed design to confirm that they are optimum, where detailed consideration of embankment deformation, especially during seismic loading, should be undertaken. This will require detailed testing and knowledge of strength and shear modulus of the rockfill materials during seismic loading.

11.3 Construction diversion arrangement

Construction diversion arrangement is discussed in more detail in section 12 (Construction Methodology). However, as it has a significant bearing on permanent arrangements, an outline of the adopted arrangements as they affect permanent works is included here.

Construction diversion will be through two to three (the exact number/arrangement will need to be determined as part of detailed design) concrete culverts located underneath the embankment. Each culvert has internal dimensions of 2.5m width and 5.0m height. A concrete starter dam at the upstream toe will form the coffer dam for directing flow through the culverts.

When the embankment is at design height, the culverts will no longer be required for flood diversion. They will then be converted for use as discharge of irrigation flow, and person access to the gate/valve control chamber at the upstream toe. More detail is provided on these arrangements in Section 11.5.

11.4 Spillway arrangement

11.4.1 General

This section discusses the spillway arrangement and provides a basis for the preliminary design features.

The selected arrangement includes a primary ogee weir, steep chute and flip bucket, and an auxiliary channel with fuse plug weir discharging to Anslow Creek as shown in Figure B-01 and B-04.

Standards adopted for the Lee Dam feasibility design outline minimum freeboard requirements and are laid out in Table 5-1. Wave heights and flood rise were calculated and combined according to these standards for a range of dam crest levels and spillway configurations as summarised in Appendix A.

The components of the freeboard allowance for the selected spillway arrangement and crest height are summarised in Table 11-4.

Table 11-4: Wind, Wave and Flood Rise Freeboard Summary

Storage Requirement		
NTWL for 13,000,000 cubic meters	197.000	mRL

Wave Heights			
Wave Height	10yr AEP 0.1% Wind	0.423	m
	100yr AEP 0.1% Wind	0.514	m

Freeboard allowance cases above floodrise			
OBF	100yr Wave	0.514	m
	10yr Wave + 0.5m	0.923	m
MDF	10yr Wave	0.423	m

Flood Rise above NTWL*		
OBF (200yr AEP)	4.077	m
MDF (PMF)	4.577	m

Required Crest Height		
NTWL + OBF + 100yr Wave	201.591	mRL
NTWL + OBF + 10yr Wave + 0.5m	202.000	mRL
NTWL + MDF + 10yr Wave	202.000	mRL
Dam Crest	202.000	mRL

*Engineered to achieve Crest of 202 m RL

11.4.2 Routing

Hydraulic analysis of the selected weir configuration was carried out to confirm the estimates provided in Appendix A.

The flood conditions represented by the inflow hydrographs presented in Section 7 were applied to a reservoir modelled using the Hydraulic Engineering Corps reservoir simulation package (HEC-ResSim 3.0). This confirmed the previous reservoir levels and weir lengths.

Outputs from the HEC-ResSim routing are summarised in Figure 11-1 and Figure 11-2 which show routed reservoir levels and inflows and outflows. Spillway rating curves may be found in Figure 11-3 and Figure 11-4, showing the relationship between reservoir level and spillway discharge.

The routing model shows a dramatic increase in flow for a short period when the fuseable embankment operates. During detailed design, the spillway arrangement could be refined to control the maximum increase in flow in the event the fuseable embankment is retained as a spillway system. Multiple fusible sections separated by retaining walls could be adopted to reduce the peak flows.

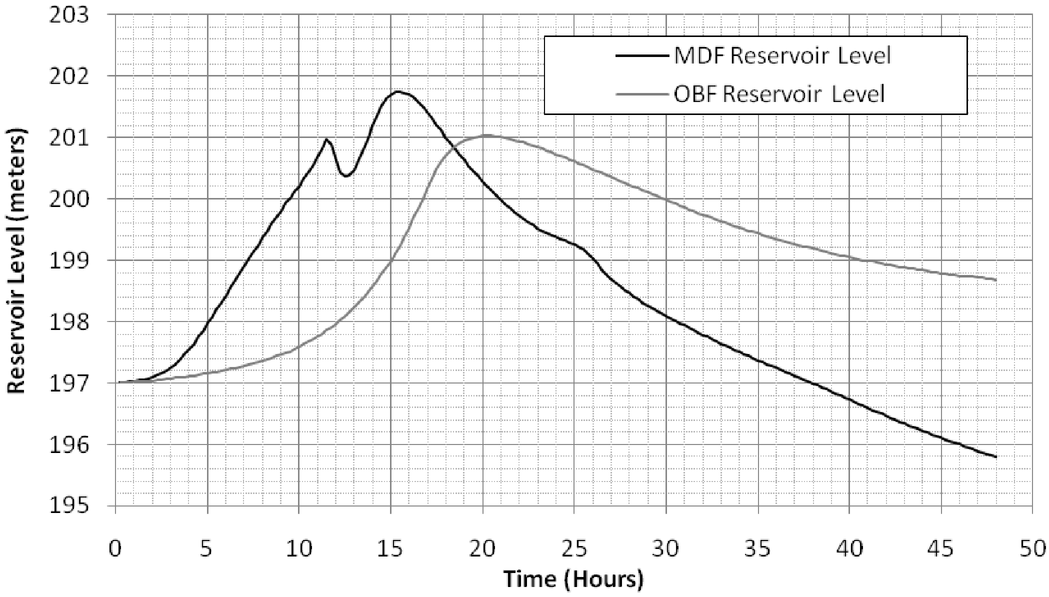

Figure 11-1: Reservoir levels during design flood events

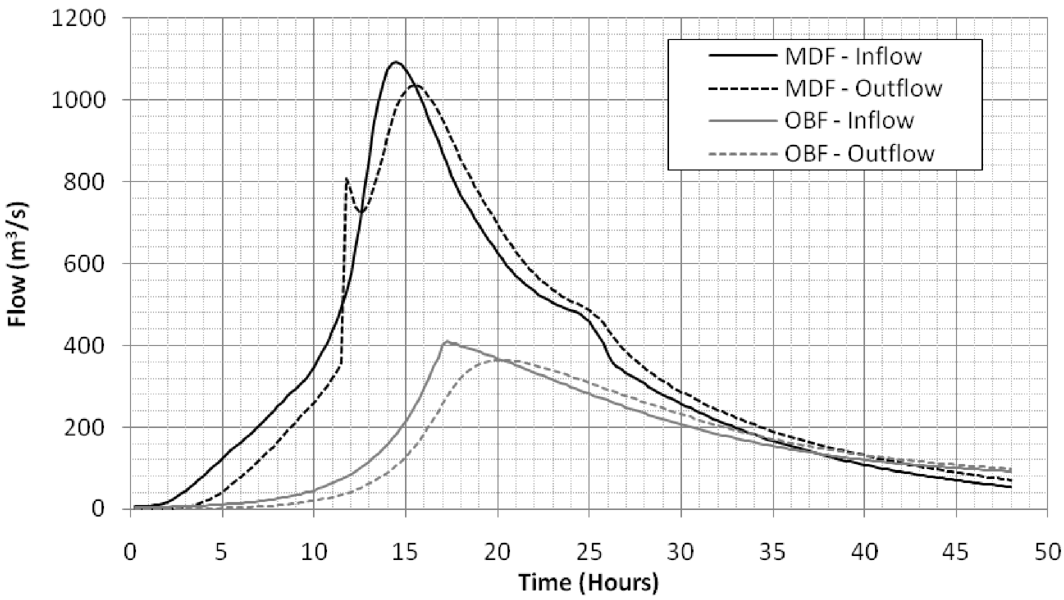

Figure 11-2:Inflows and routed outflows during design flood events

11.4.3 Approach channel

The approach channel is 46.8 m wide and includes a large radius bend followed by 45 m straight approach to the primary weir. The channel is trapezoidal in shape and the majority will be cut into rock with 1V:0.5H side slopes. The channel floor has a flat longitudinal slope allowing drainage under low reservoir conditions.

Maximum flow velocity expected during the OBF and MDF will be 1.1 m/s and 2.8 m/s respectively resulting in negligible head losses for the OBF flows and less than 50 mm during MDF flows.

General guidelines based on the results of studies and existing installations (Khatsuria 2005) were followed in the preliminary design of the approach channel as follows:

- approach velocity for design discharge generally less than 3 m/s although up to 6 m/s has been allowed in the past (Beas dam, India)
- a straight approach results in nearly uniform approach distribution of flow across the spillway, however curved layouts are inevitable to avoid large excavations
- a minimum straight length equal to 1 or 1.5 times the spillway width is recommended
- the ratio, radius of curvature to depth of flow (R/y) should be as large as possible but no less than 3
- a converging channel has been found to be effective in equalizing flow distribution.

11.4.4 Primary weir

An ogee primary weir was selected as evaluated in Appendix A. The arrangements adopted for this feasibility study represent one option that appears economic. Further studies during detailed design may conclude that refinement of the weir, chute, or flip bucket arrangements are warranted. This could result in changed vertical and horizontal alignment, and varied dimensions. Such changes should be allowed for in any resource consent applications.

The weir crest is at RL 197 m, has an effective length of 20.5 m and an approach depth of 3 m (weir crest to approach channel invert). During design flow (OBF), the operating head will be 4.06m. A spillway coefficient of 2.22 (Belvins 1984) was used at the design head to give an operational design flow of 372m^3/s.

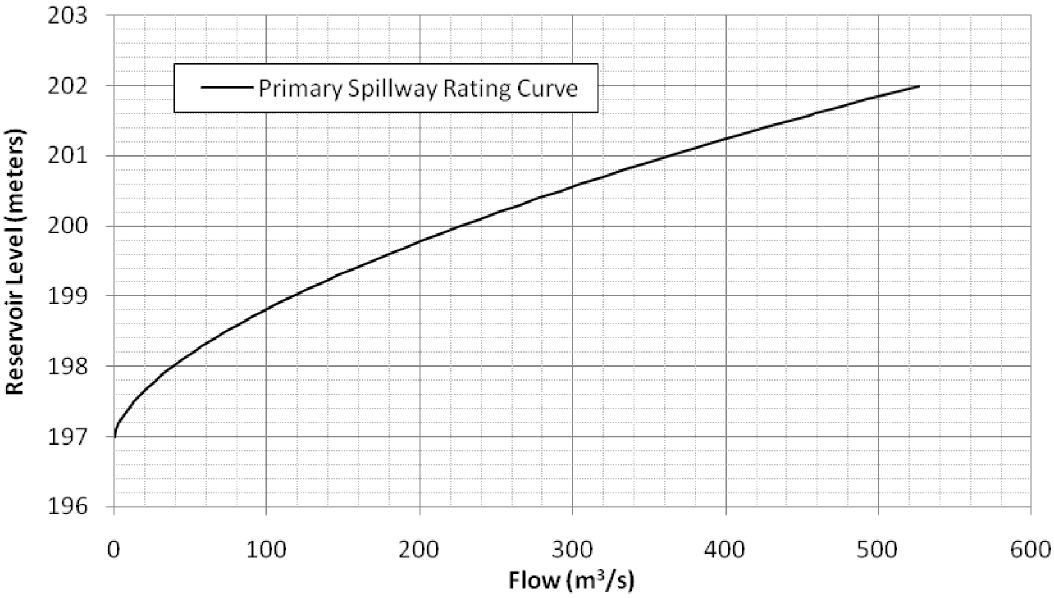

Figure 11-3: Primary ogee weir spillway rating curve

To access the dam crest, a bridge across the spillway is necessary. To reduce the span length and cost of the bridge a 1 m wide pier was included mid spillway. The spillway flow is affected by the spillway abutments and central pier and additional spillway width has been included to compensate.

The relationship presented by Khatsuria (2005) was used to allow for a single rounded bridge pier and rounded abutments by introducing additional spillway width. The additional spillway width required was 1.8 m resulting in a final spillway width of 22.3 m.

Potential head losses in the approach channel were not accounted for in the spillway design. However, this head loss is not expected to influence the final design of spillways. Head losses during OBF flows are negligible (less than 10 mm) and during MDF flows are less than 50 mm, depending on the lining and resulting roughness of the channel.

The spillway bridge deck is set to RL 203 m which allows for a bridge depth of 1.423 m to the MDF flood rise of RL 201.577 m without allowing for drawdown over the crest.

The weir profile was specified using the curves given in "Design of Small Dams" (USBR 1987). The downstream side of the weir continued the ogee profile until RL 195 m at which point a circular curve with a radius of 2.5 m smoothly transitions the flow into the chute.

Supercritical flow is maintained once flow passes the crest as the downstream chute has adequate slope (1:14.5) to ensure this. Flows over the downstream portion of the weir were modelled using the Hydraulic Engineering Corps river analysis system software (HEC-RAS 4.0) and is discussed further in Section 11.4.6.

11.4.5 Auxiliary weir and channel

The auxiliary spillway channel and fusable embankment as seen in Figure B-04, was selected as outlined in Appendix A. Routing in conjunction with the ogee weir required an auxiliary spillway width of 19.5 m.

The spillway flow was modelled using HEC-ResSim as a gated rectangular broad crested weir to confirm the previous calculations in Section A4.4.4.

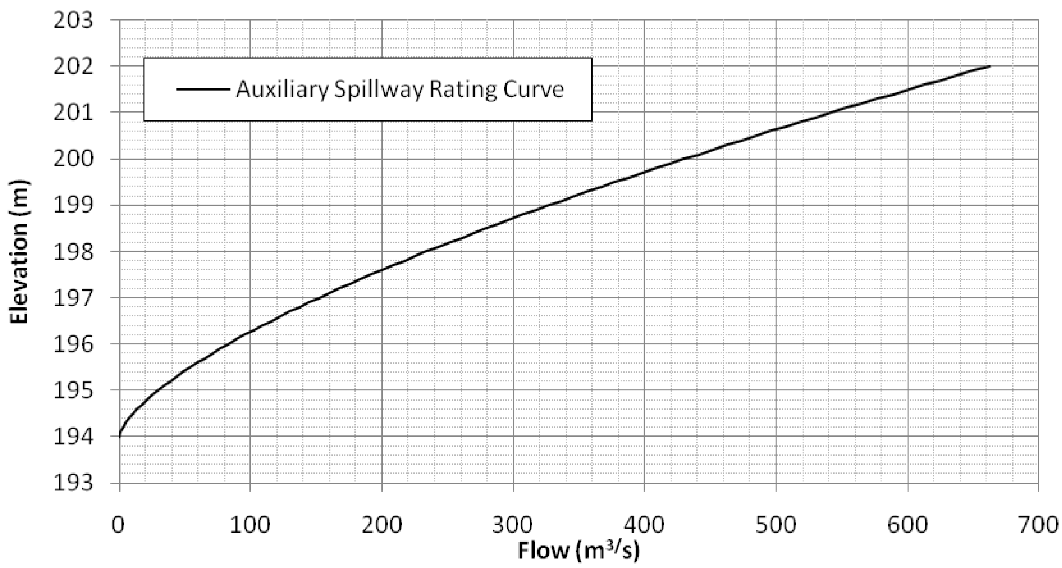

Figure 11-4: Auxiliary broad crested weir spillway rating curve

The fusable embankment will be built into the auxillary channel with a crest level of RL 201.1 m. Side slopes have currently been set at 1V:2H, however detailed design of the embankment and fuse mechanism has not been undertaken.

The fuse embankment is intended to operate once the water level increases beyond RL 201.01 m and will be designed fuse in a controlled manner, typically specified as a rate of increase in flow.

Controlled fusing can be achieved in a number of ways. However, it is common to divide the embankment into cells that fuse at different levels thus staging the increase in flow. This should be investigated during the detailed design of the embankment.

11.4.6 Primary chute

The primary spillway chute design has resulted in a rectangular chute with a contraction and vertical curve. The geometry of the chute was designed to minimise the amount of cut required while interfering with the embankment construction as little as possible. The chute is founded on rock where possible so as to reduce the amount of mass concrete foundation work required. Chute walls are 4 m high.

More detailed design studies may indicate that removal or reduction of vertical curvature may be warranted to avoid complication and simplify construction. Such changes are possible and should be considered as part of any resource consent applications.

The initial chute section has a mild slope of 1V:14.5H and maintains the full spillway width for 30 m. Contractions or expansions are usually to be avoided within one spillway width of the weir to allow the flow to normalise. Piers and abutments can generate waves with heights 25% of the flow depth and chute walls have been designed to allow for this when calculating freeboard.

At the 30 m point, the chute begins a 40 m long contraction reducing the spillway width from 22.3 m to 10 m. The chute is contracted to avoid excessive cut as it steepens and is cut into the side of the ridge. Contracting the flow increases the depth of flow for the same slope and a Hydraulic Engineering Corps river analysis system (HEC-RAS 4.0) model was developed to ensure that the flow remained supercritical. Figure 11-5 shows the gradually varying supercritical flow profile over the length of the chute.

Contractions can introduce complex cross-waves to the supercritical flow in a chute. These flow effects were not considered at this stage but would need to be addressed during detailed design with an in-depth analysis and numerical or physical modelling.

The chute enters a circular vertical curve 10 m into the contraction. The curve transitions the flow from a 1V:14.5H slope to a 1V:1.67H slope. This change in vertical geometry serves to speed up the flow as it is contracted, reducing the depth and ensuring flow does not go above critical depth. There is also some indication that accelerating the flow through a contraction may reduce the size and effect of the cross-waves generated. Vertical convex curves can encourage flow separation from the chute. This was checked using the method provided in Khatsuria(2005), and a 30 m radius curve was adopted to satisfy this condition and conform to the terrain. The curve was tangential to the sections either side with a length of 15.95 m.

The final section of chute has a slope of 1V:1.67H (60%). Final flow conditions at the end of this section as calculated using HEC-RAS are as follows:

Velocity:	25.91 m/s
Depth:	2.32 m
Froude Number:	6.27

HEC-RAS includes an allowance for the aeration of the flow as it travels down the chute and a freeboard allowance of 1m has been provided above this level to ensure that unforeseen flow conditions such as irregular cross-waves do not overtop the chute.

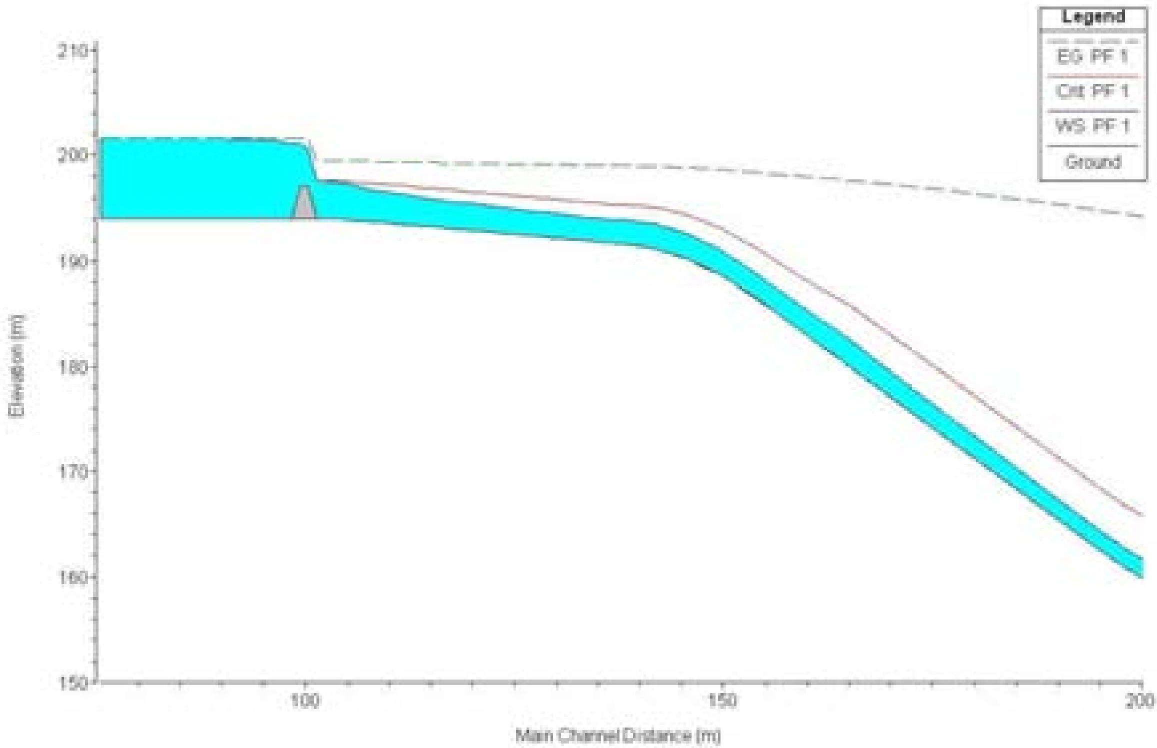

Figure 11-5: Primary spillway – HEC-RAS model flow profile

The chute floor and walls will need to be designed in detail. However, at this stage the floor and walls have been assumed to have an average thickness of 0.3 m based on experience in similar dam projects such as Nelson's Maitai dam. Wall details such as jointing and under-drainage will require special attention to prevent damage during spillway flows resulting from cavitation or the development of pressure in joints or under the floor slabs.

Construction of the steep chute section floor may require slip forming similar to that required for the upstream face of the dam.

11.4.7 Flip bucket

The dissipation of energy at the termination of the chute will be achieved with a flip or trajectory bucket and plunge pool. The flip bucket terminates the chute in a large radius curve that throws the water in an arc downstream and is often referred to as a "ski jump". Energy is dissipated as the flow jet breaks up in the air and as it enters the plunge pool downstream.

Design parameters have been established over time with both model and prototype studies such as Varshney and Bajaj (1970). Smooth circular curves are generally preferred with radii related to the flow characteristics entering the bucket.

A bucket radius of 25 m was selected following the relationship determined by Varshney and Bajaj (1970) which was based on studies of existing installations. The radius is based on the MDF unit flow of 45 m^3/s per meter width of spillway.

The bucket lip or exit angle determines the throw distance and angle of the flow entering the water, which in turn has a large effect on the scour depth in the plunge pool. In practice, angles typically vary from 20^0 - 40^0 and an angle of 30^0 was adopted for the preliminary design. The lip is set at 156.575 m RL or 1 m above the MDF tail-water level.

The bucket design parameters result in an overall bucket length of 26.4 m with a depth from lip to invert of 3.35 m. Bucket side walls were assumed to be 0.3 m wide as per the side walls of the chute. Wall thickness will need to be confirmed as a result of more detailed study of the pressures developed in the bucket invert and sidewalls.

The minimum depth of concrete at the invert is 1.75 m. A detailed analysis of the bucket forces and foundations, including model studies and drilling will need to be done during detailed design to ensure there is adequate foundation strength and mass in the bucket.

Hydraulic modelling is recommended to confirm the bucket performance over the full range of flows at the detailed design stage.

The area immediately downstream of the bucket will require concrete lining and protection as it is subject to frequent flows lower than the design that do not become airborne or are thrown short of the plunge pool.

11.4.8 Plunge pool

Energy dissipation with a flip bucket and plunge pool is economic due to the fact that erosion occurs at a distance from the toe of the dam.

A throw distance of 59 m during MDF flow was calculated using the projectile equation suggested by WES -HDC 112-8. Maximum scour depth was checked to ensure this distance was adequate by determining the size and extent of the plunge pool required.

Ultimate scour is predicted to be 9 m below current bed level using formulae suggested by Yildiz and Uzucek (1996) and assuming MDF tail-water conditions. Yildiz and Uzucek use a modified Veronose formula based on prototype studies and the recognition that the impinging angle of the jet to the tail water is significant for chute and flip bucket spillways. It is also noted that given a long enough period, the particle size is not important. Bedrock and larger particles are removed or broken up and ultimate scour is similar for either case.

The calculated ultimate scour depth could be reduced by altering the geometry of the flip bucket to 'fan' the flow. A bucket flare angle of only 5 degrees would reduce the unit discharge over 59 m to 22 m^2/s and result in an ultimate scour depth of only 5 m. This would also result in the plunge pool width increasing from 10 m to 20 m wide.

Plunge pools are often excavated and lined with rock sized to resist movement resulting from flow through the pool and the turbulence of the discharge. Excavating the plunge pool rather than allowing scour to develop may give greater control over the extent of scour and allow for a significant drop in tailwater under low flow conditions. This drop in tailwater could be utilized as additional head if electricity generation were to be installed.

11.5 Outlet works arrangement

The objective of the reservoir outlet works is to extract water from the desired elevation within the reservoir and discharge it downstream of the dam at the desired flow rate. Separate analyses (also reported separately) have established requirements for extraction levels and flow rates, and these have been used as design parameters for the outlet works. The design parameters distilled to relatively few inputs, summarised as follows:

top extraction elevation	RL 185 m
bottom extraction elevation	RL 167 m (also dead storage elevation)
maximum flow rate	5 cumecs (for flushing)
reservoir range for max flow	full range.

Screening will be needed for both the base (water augmentation only) and any hydro option cases. For the base case screen size is 100 mm and is intended to exclude debris which could jam the outlet system.

The intake system adopted for this feasibility assessment consists of twin rectangular conduits laid on the concrete upstream face of the embankment. This arrangement avoids the need for an intake tower and is expected to offer considerable savings. More detailed analysis and design may require return to an intake tower arrangement so this option should not be excluded for any resource consent applications.

The arrangement is shown on Figure E-01. An intake box is located at the base of the reservoir and at the upstream end of the diversion culverts. Holes are located in the roof of the box leading to twin rectangular reinforced concrete conduits on the upstream face of the embankment. These will be connected to the concrete face of the embankment with shear keys, but have sufficient mass to resist flotation while empty without relying on a tension connection to the concrete face.

Rails are cast into the embankment concrete face above the inlet elevations, which allow placement of interchangeable stoplogs and intake bell-mouth/screen system lowered via cable from the dam crest. This system will allow the conduits to be isolated when necessary for gate maintenance, and allow periodic maintenance and cleaning of screens (via extracting to the surface and replacement).

Radial gates are located in the intake box, which discharge to the culverts. Access to the gates is via the third culvert, which also includes a crane rail for placement/removal of the gates as required.

Stoplogs are provided at the downstream end of the culverts to protect the gates against extreme flood levels.

11.5.1 Hydroelectric add-on

The above parameters are applicable for a release system without hydroelectric addition (which is the base case for this feasibility report). There is an option to include hydroelectric generation on the flow release. The economic feasibility of this option is considered separately but hydraulic arrangements necessary for hydro are included here. If hydroelectric generation is included the outlet works design parameters will vary slightly from above. Three flow options have been considered, with maximum flow rates of:

4 cumecs

6 cumecs

8 cumecs.

Finer screens (20 mm aperture) are needed for the hydro option to exclude fish (as described in the accompanying Technical Report "Aquatic Ecology– Mitigation and Management Options Associated with Storage in the Proposed Lee Reservoir", with an approach velocity of 0.3 m/s. The biological design parameters have been developed separately and reference should be made to the specific assessment for more detail.For the hydro option, some changes are necessary from the base arrangement and are shown on Figure G-01. The system upstream of the intake box is essentially unchanged (except the conduit size will increase for larger flow options). Bell-mouth intakes to a pipe will be installed in place of the base case radial gates, leading to a butterfly valve on each pipe, followed by a bifurcation, and single penstock leading through one of the culverts to a power station located at the downstream toe of the dam.

11.6 Preliminary hydropower optimisation

11.6.1 General

The proposed dam and reservoir, while principally for water storage and flow release during low flow and/or high demand periods, also lends itself to hydropower generation. Hydro-electric power may be generated at the base of the dam by adding a set of turbines and associated generating equipment at the outlet conduit. Other required works comprise a power house to house the equipment, transmission, switchgear and various minor works. Outline arrangements for the recommended installed capacity are shown on Drawing G-01.

In essence, the dam and reservoir may be considered the headworks of the hydro-electric power scheme. However, it should be noted that the release pattern from the dam will be governed solely by downstream consumptive demands and in-stream requirements and not by electricity demand. That is, hydropower generation will be a by-product of dam operations to meet downstream water demands. Further, for the current assessment, the power station is assumed to operate as a constant flow as opposed to a daily peaking station. This means that the flow released from the dam is assumed to remain constant throughout each 24 hour period, but the flow may vary from one day to the next.

A preliminary optimisation has been completed to determine the apparent optimum size of generating plant. There are three main considerations in the optimisation:

1. Expected generation output (i.e. hydro-energy)
2. Cost of hydropower plant and other associated works
3. Value of hydro-energy.

The first two aspects are inter-related but in a complex way. That is, the expected generation increases with increasing plant size but the marginal increases in output are ever diminishing (compared with plant size increments). Clearly, the larger the plant, the more expensive, but the cost increase is not linear because of economies of scale i.e. cost per unit drops with increasing size. However, there may be step increases in certain items such as in the transmission line as plant size is increased. The value of the generation output is directly proportional to the volume of output.

11.6.2 Hydro-energy output

Power generated is proportional to the product of the generation flow and head, the latter of which is given by the water level difference between the reservoir and the river downstream. The height of the dam and thus the maximum generating head is more or less fixed. The energy output is thus dependent on the size of the installed plant, specifically the maximum design generation flow and, to a lesser extent, the buffering storage provided within the reservoir. When the reservoir is full or nearly full, this buffering storage allows partial capture of small floods and freshes (which would otherwise be spilled) for generation. The buffering storage is separate and additive to the live storage required for meeting water demands downstream in the design drought.

Figure 11-6 shows the relationship between expected annual energy output in GWh p.a. and the generation flow capacity in m^3/s for a range of buffering storages. Note that on the horizontal axis, the generation flow capacity may be converted to power output by using the simple conversion 0.40 MW per m^3/s.

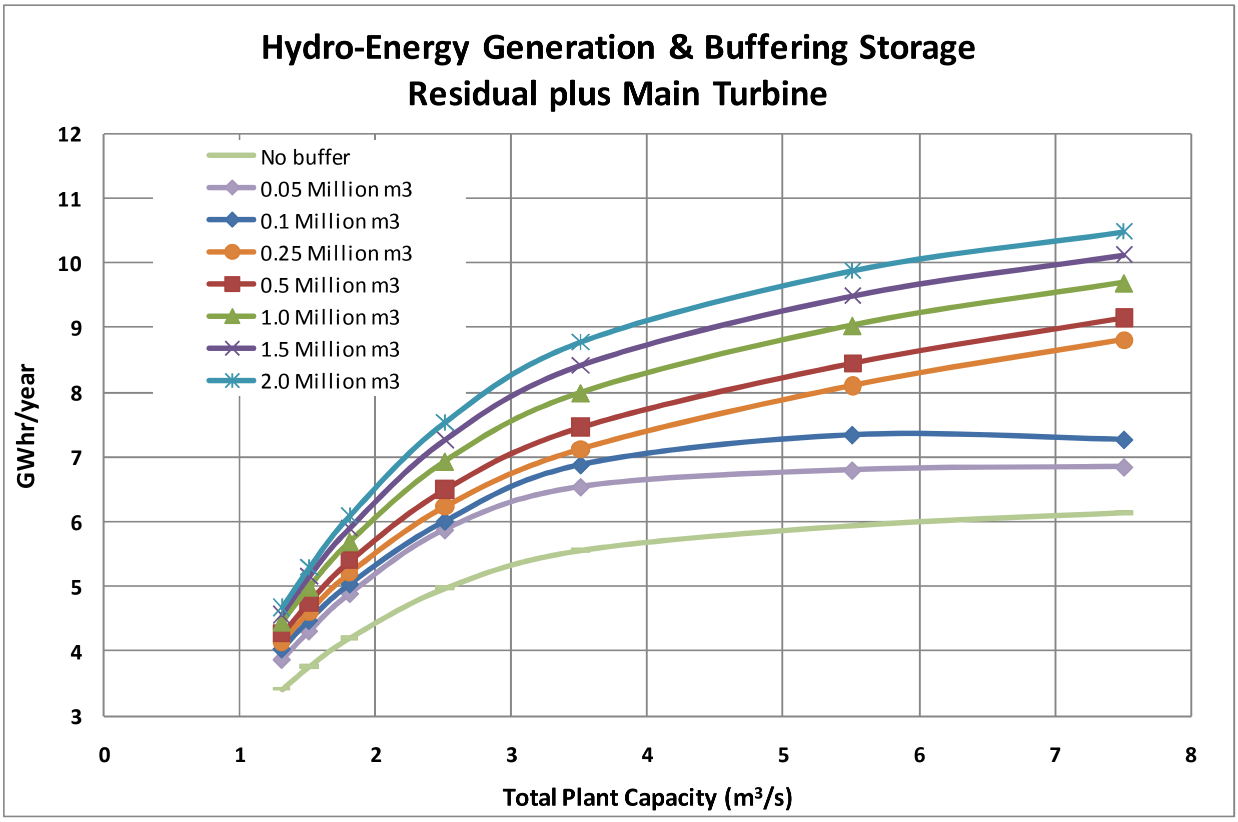

Figure 11-6: Expected hydro-energy output versus generation flow capacity and buffering storage

Other assumptions implicit in the hydro-energy calculation include:

- tailwater level nominally at RL 150 m, thus, at a full supply level of RL 197 m, the gross head available prior to hydraulic headlosses is 47 m;
- generating set comprising a residual flow turbine with 0.51 m^3/s generation capacity plus a main turbine (generation capacities ranging from zero to 7 m^3/s

have been modelled for the main turbine); the total flow capacity is the sum of the capacities of the residual flow turbine (0.51 m^3/s)and the main turbine;

- a total head loss 1.5 m at peak generation for both the residual flow turbine and the main turbine;
- generator efficiency of 0.97; the computed energy output is at generator terminals and exclude transmission line losses;
- minimum generating head of 33.1 m and 34.2 m for the residual flow and main turbines;
- minimum generation flow equal to 40% of the maximum generation flow for each turbine.

As noted earlier, generation is assumed to be completely incidental to dam release operations to meet downstream demands, except when the storage is greater than 13 million m^3, in which case the generating plant is assumed to operate at capacity to draw the reservoir down to 13 million m^3. In this way, the minimum storage reached during a drought is not any lower than without hydro.

Buffering Storage

The family of curves shown in Figure 11-6 corresponds with different amounts of buffering (or regulating) storage over and above the base 13 million m^3 gross storage for meeting downstream water demands. These range from zero (no buffer) to 2.0 million m^3; for example, the 1.0 million m^3 curve assumes a total storage of 14 million m^3, of which 1 million m^3 is assumed available for power generation operation. A significant proportion of the increase in energy output with increasing buffering storage arises from the slightly higher maximum water level and thus generation head available. However, there is a cost to this; for example adding 1.0 million m^3 of storage requires the dam to be raised by about 1.5 m.

Assessments show a very steep drop off in incremental energy output with increasing buffering storage. Applying approximate costs for dam raising (for providing buffering storage) indicate that only a relatively small buffering storage is warranted, depending on the generation capacity. There is no advantage, in net present value terms, in providing more than 250,000 m^3 of buffering storage for a total generation capacity up to 5 m^3/s.

At the nominated full supply level of RL 197 m (see section 12), the gross storage is about 13.42 million m^3. So there is 420,000 m^3 of additional storage compared with design requirements (13 million m^3 gross), which is greater than the maximum buffering storage requirement of 250,000 m^3. It should be noted that the nominated full supply level has been rounded up from the precise level corresponding with a storage volume of 13 million m^3 viz. RL 196.36 m. If a normal top water level of RL 197.0 m were maintained, then for a 250,000 m^3 buffering storage, hydro generation would result in the reservoir level typically fluctuating in a 0.39 m range between RL 196.61 m and RL 197.0 m, when it would otherwise be full and spilling.

Section 6.5 of the accompanying "Water Resources Investigations" report provides a description of the operating regime with hydropower generation added on.

Transmission

The proposed dam site is approximately 2.4 km beyond the end of the existing overhead 11 kV line at the Lee Cement works and an extension will be required to/from the dam site. Approximate costs for transmission out of the Lee Valley have been advised by Network Tasman Ltd (pers. comm. Murray Hendrickson) and are as follows:

- 11kV - generation up to 350 kW - $330,000
- 22kV - generation up to 1200 kW - $1.85 million
- 33kV - generation beyond 1200 kW - $2.32 million.

These estimates are construction estimates only and do not include consenting or easement purchase costs, which would be minor for the 11 kV option, but more substantial for the 22 kV and 33 kV options.

Approximate Costs of Hydro Add-On Components

Figure 11-7 shows how the approximate costs of the hydro add-on components vary with increasing plant capacity. Separate curves are provided for the civil works and generating plant and for the transmission options as described earlier. Refer to section 13 for commentary on the assumptions for this cost estimate.

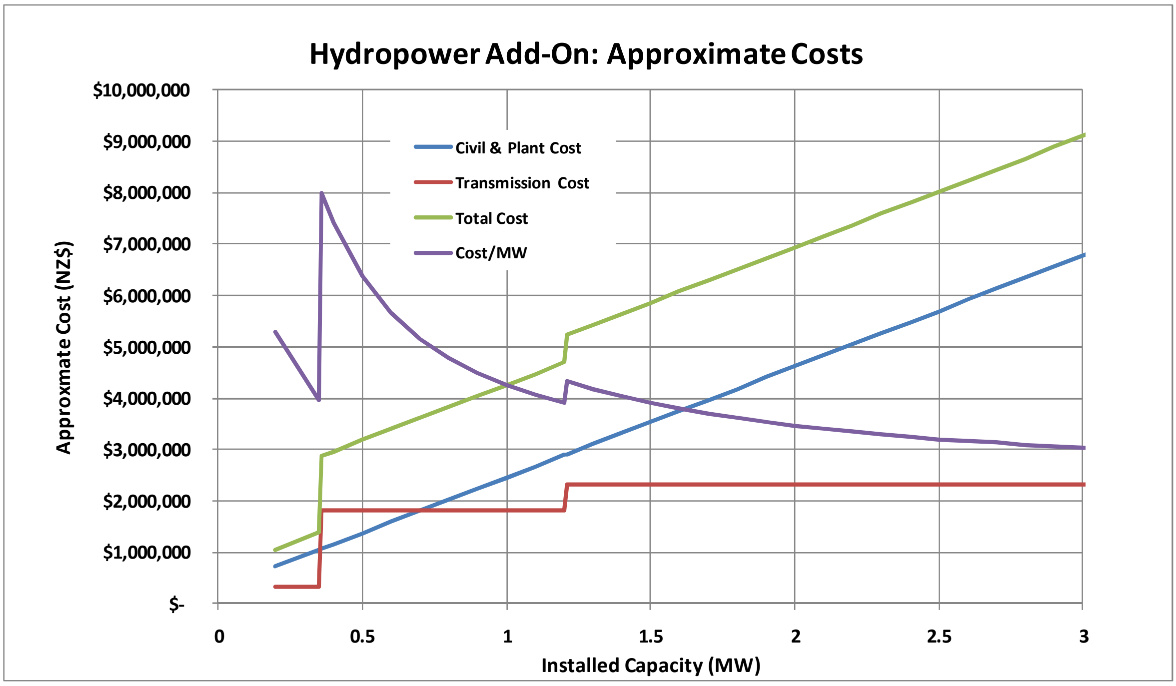

Figure 11-7: Approximate costs of hydropower add-on components

Approximate Value of Hydro-Energy Generation

Determining the value of the hydro-energy generated in the future involves making many assumptions, particularly as to the price path of wholesale electricity and discount rate. For the purpose of this preliminary optimisation, an indicative net present value (NPV) for hydro generation of roughly about $1.0 million per GWhr p.a. has been used, together with an upper estimate of up to $1.24 million per GWhr p.a.

Apparent Optimum Design Flow and Plant Size

An apparent optimum plant size may be identified by comparing the net present value of the expected hydro generation over the economic life of the project against the capital cost of the hydro add-on components for a range of plant capacities. Table 11-5 provides this comparison for generation NPVs of $1.04 million per GWh p.a. and $1.24 million per GWh p.a..

For both NPVs, the maximum benefit to cost ratio occurs when the plant is sized for passing just the residual flow (0.51 m^3/s, 0.204 MW). There is a secondary optimum at a plant capacity of 2.51 m^3/s (0.995 MW), which would comprise the residual flow turbine (0.51 m^3/s) and a main turbine with a 2.0 m^3/s generation capacity. However, the maximum potential return in monetary terms is indicated by the column "NPV less Add-On Cost". This clearly points to the larger planting case (i.e. 0.995 MW, 2.51 m^3/s) as the more attractive option.

It should be noted that further refinement in subsequent study phases may indicate a marginally higher optimum planting, say up to the limit for a 22 kV line upgrade option (viz. 1.2 MW).

Table 11-5 Preliminary optimisation of hydro plant size

NPV = **1.04** **$ million per GWh p.a.**

At 0.25 million m^3 regulating storage

Plant Size (MW)	Total design flow (m^3/s)	Add-On Cost ($ million)	Generation Output (GWh p.a.)	Generation NPV ($ million)	Ratio NPV/Cost	NPV less Add-On Cost ($ million)
0.204	**0.51**	**1.07**	**1.75**	**1.82**	**1.70**	**0.75**
0.519	1.31	3.22	4.15	4.32	1.34	1.09
0.599	1.51	3.40	4.61	4.79	1.41	1.40
0.717	1.81	3.65	5.21	5.42	1.48	1.77
0.995	**2.51**	**4.25**	**6.23**	**6.48**	**1.52**	**2.23**
1.391	3.51	5.62	7.12	7.40	1.32	1.78
2.183	5.51	7.33	8.1	8.42	1.15	1.09
2.976	7.51	9.05	8.81	9.16	1.01	0.12

NPV = **1.24** **$ million per GWh p.a.**

At 0.25 million m^3 regulating storage

Plant Size (MW)	Total design flow (m^3/s)	Add-On Cost ($ million)	Generation Output (GWh p.a.)	Generation NPV ($ million)	Ratio NPV/Cost	NPV less Add-On Cost ($ million)
0.204	**0.51**	**1.07**	**1.75**	**2.17**	**2.03**	**1.10**
0.519	1.31	3.22	4.15	5.15	1.60	1.92
0.599	1.51	3.40	4.61	5.72	1.68	2.32
0.717	1.81	3.65	5.21	6.46	1.77	2.81
0.995	**2.51**	**4.25**	**6.23**	**7.73**	**1.82**	**3.47**
1.391	3.51	5.62	7.12	8.83	1.57	3.21
2.183	5.51	7.33	8.1	10.04	1.37	2.71
2.976	7.51	9.05	8.81	10.92	1.21	1.88

Recommended Development

Based on the preceding considerations, an apparent optimum planting configuration has been identified. This configuration, which is the preferred hydropower add-on for this feasibility study, comprises the following elements:

- a residual flow turbine and generator with a flow capacity of 0.51 m^3/s (matching the dam residual flow) and power output of 0.20 MW; plus
- a main turbine and generator with a flow capacity of 2.0 m^3/s and power output of 0.79 MW;
- upgrading of the Lee Valley transmission line to 22 kV;
- retention of a normal top water level of RL 197.0 m for the reservoir; and
- provision of an operational storage volume of 250,000 m^3 for hydropower regulation, which corresponds with a 0.39 m operating range between RL 196.61 m and RL 197.0 m.

11.7 Fish passage

Upstream fish passage facility has been incorporated in the dam, following interactive development of design parameters and arrangements with Cawthron. A small pump will be installed at the dam crest to release a small flow (a couple of litres per second) down a 'naturalised' channel on the downstream face of the dam, and a release channel on the upstream face of the dam.

Drawing B-05 shows the indicative plan arrangements of the channel. Note that the channel alignment could easily be located with upstream intake at the right abutment of the dam if desired. The downstream end of the channel (at the dam toe) transfers into a fish –friendly culvert to cross the access bench, and pipe for location of the fish entry as close to the water release area as possible.

12 Outline Construction Methodology

12.1 General

This section outlines the anticipated construction methodology for the dam, including requirements for construction facilities, borrow areas, dump areas, and the like. Careful control of river diversion is essential for the safety of the dam during construction, and the hydraulic characteristics of the diversion stages are discussed in section 12.2. The anticipated site layout is discussed and presented in section 12.3. The construction process is a continuum, but has been broken into several nominal stages which are discussed in sections 12.4 to 12.11. Anticipated timing of the construction process is discussed in section 12.12.

This section of the report, while prepared by T&T, has been reviewed and commented on by sub-consultants active in the construction industry with construction firms.

12.2 River diversion hydraulics and stages

The hydraulics of the river diversion and dam during construction has been developed by routing flood flows through the diversion culverts. Cases of two and three culverts have been assessed to show the balance between cost and risk during construction, and to assist in developing a cost effective diversion strategy that maintains safety during construction. The flows and depths calculated in this process were used to evaluate the potential flows due to hypothetical breach of the incomplete works due to flooding and to ensure the construction design has an acceptable level of risk.

12.2.1 Diversion culvert routing

Firstly, a stage discharge relationship was developed for a single box culvert using the weir and orifice control equations provided in the FHWA Hydraulic Design Series Number 5 (2005). The culvert inlet hydraulics were analysed for a wingwall entry and 5 to 1 tapered inlet. The stage discharge relationship for combinations of two culverts and three culverts is given in the following Figure 12-1.

12.2.2 Breach flows

Using the storage elevation relationship shown in Figure 12-2 flood routing was undertaken to determine the ponding depth required for a particular flood to pass through the diversion culverts. The ponding depths resulting from this routing exercise are shown in Figure 12-3. Figure 12-4 shows two peak breach flow curves for two and three culvert options. These curves assume that the concrete starter dam does not fail in the event of an embankment breach.

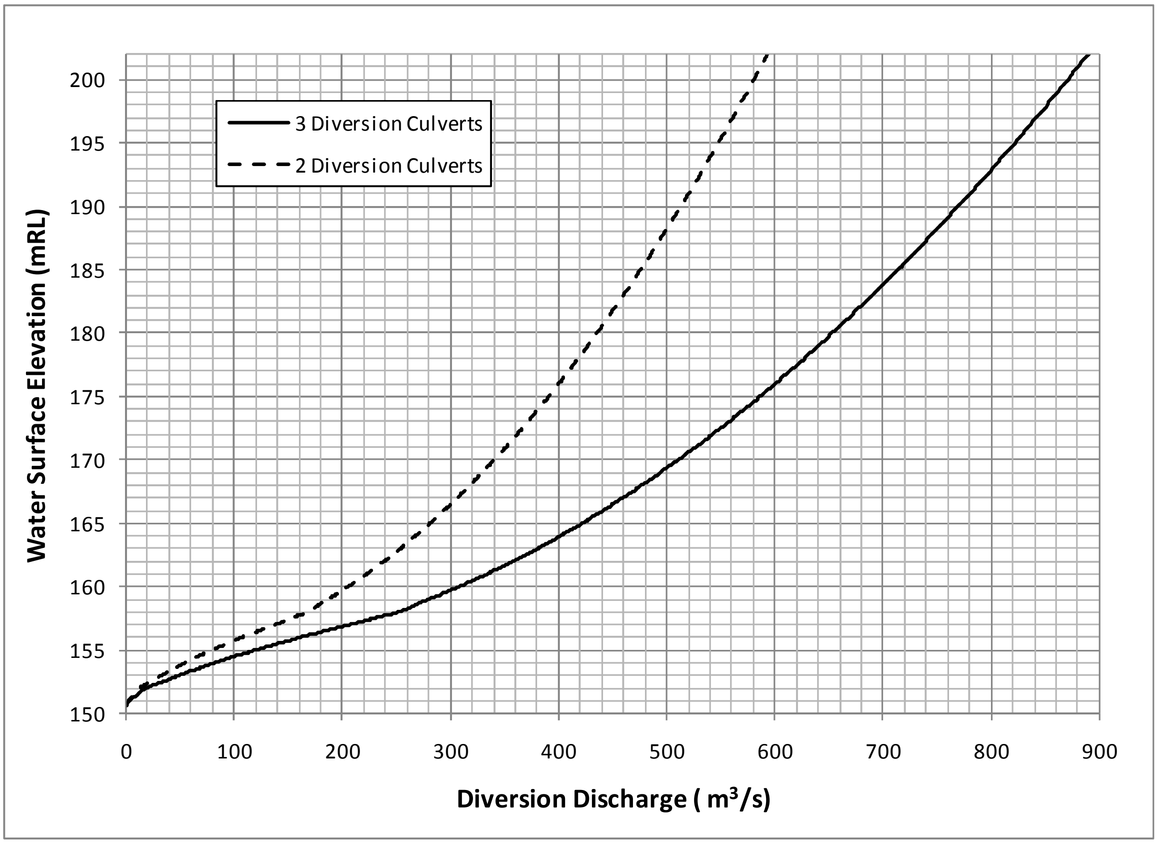

Figure 12-1: River Diversion Stage Discharge Curves

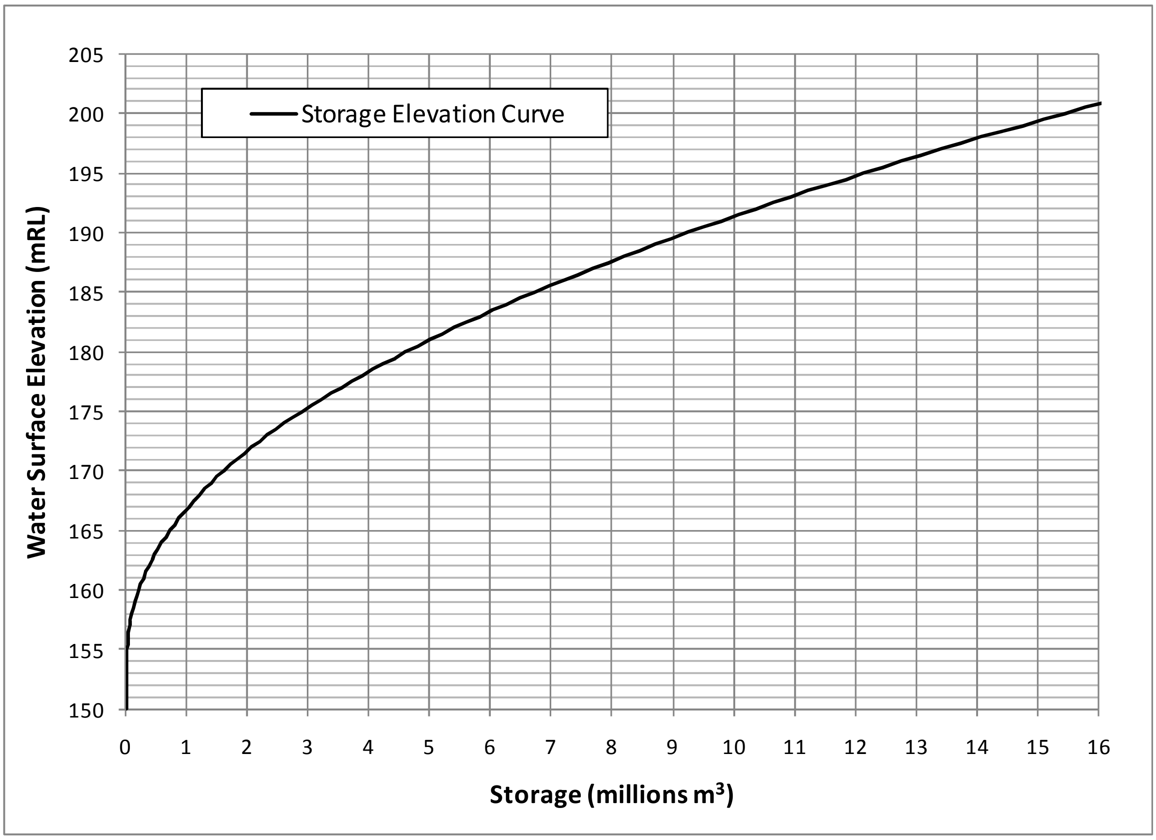

Figure 12-2: River Diversion Storage Elevation Curve

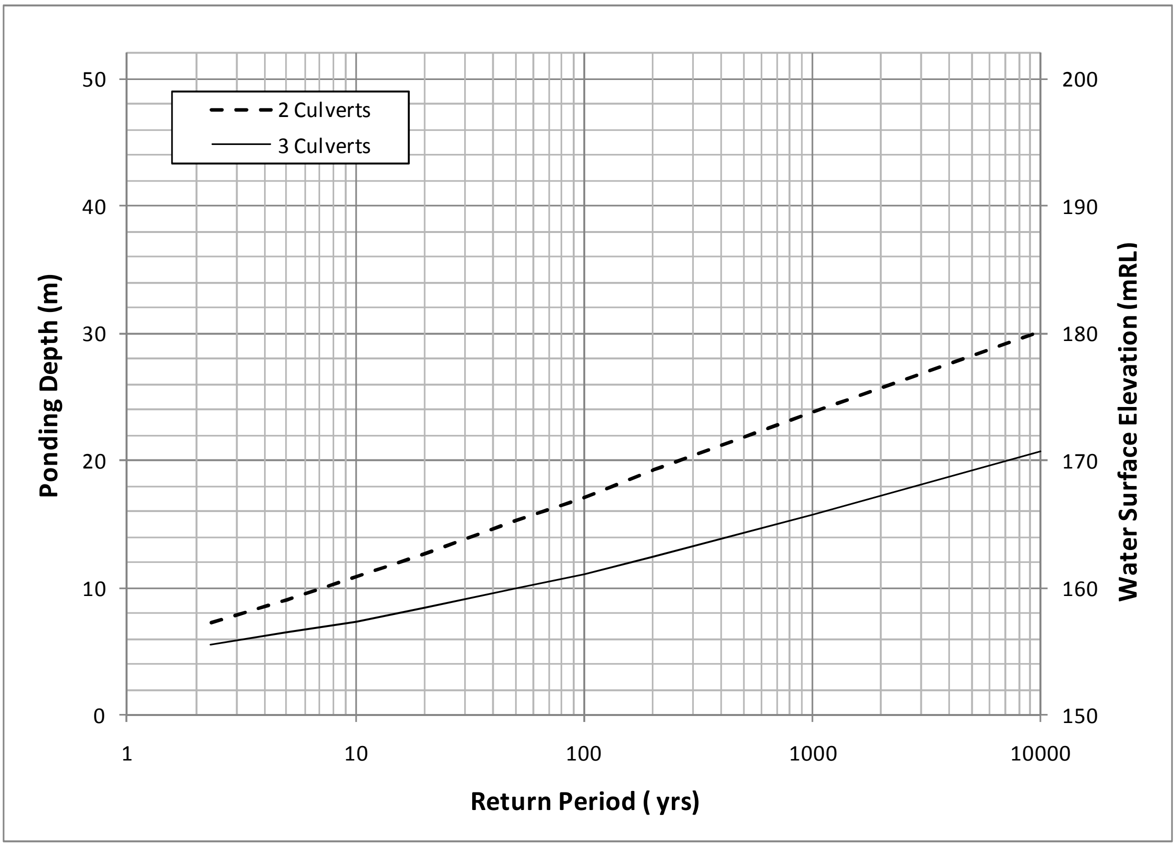

Figure 12-3: Ponding depths for flood flows routed through diversion culverts

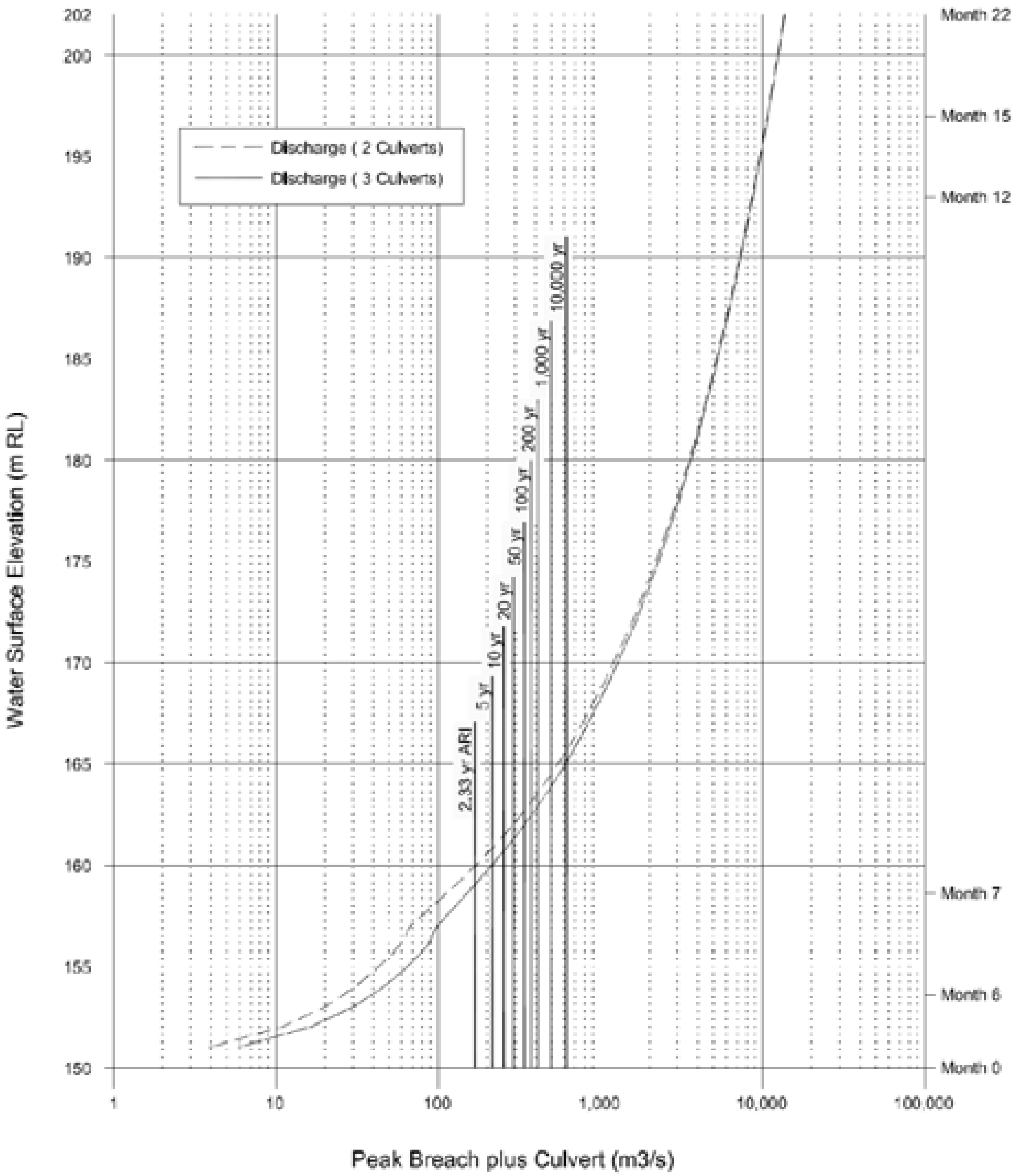

Figure 12-4: River Diversion Breach Peak Discharge Elevation Curve showing Staging and Return Period Floods

12.2.3 Staging and capacity

Given the capacity of the diversions after routing the flows, the capacity of the construction works to pass floods was plotted with respect to the stage of works (stage descriptions are provided in following sections). This capacity was then used to determine the recurrence interval of the storm each stage is capable of diverting without damage. Figure 12-5 shows a section across the river facing downstream and the diversion staging. The following four plots (Figure 12-6 to Figure 12-9) show available diversion capacity, recurrence interval of diversion capacity flood, potential peak breach flow if overtopped, and available storage upstream of the embankment, at the various stages of construction. These have also been consolidated into Figure 12-4.

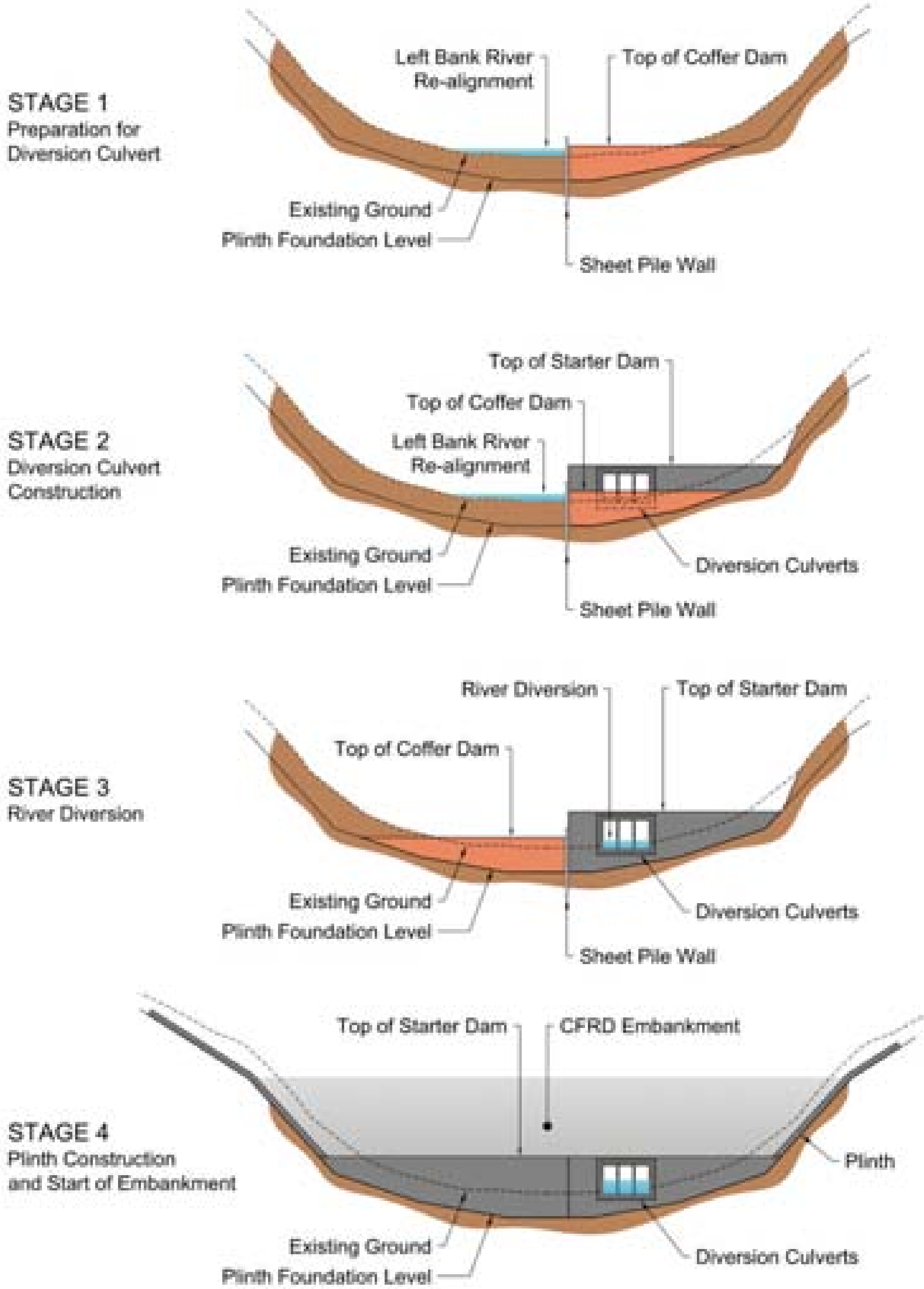

Figure 12-5: Diversion staging

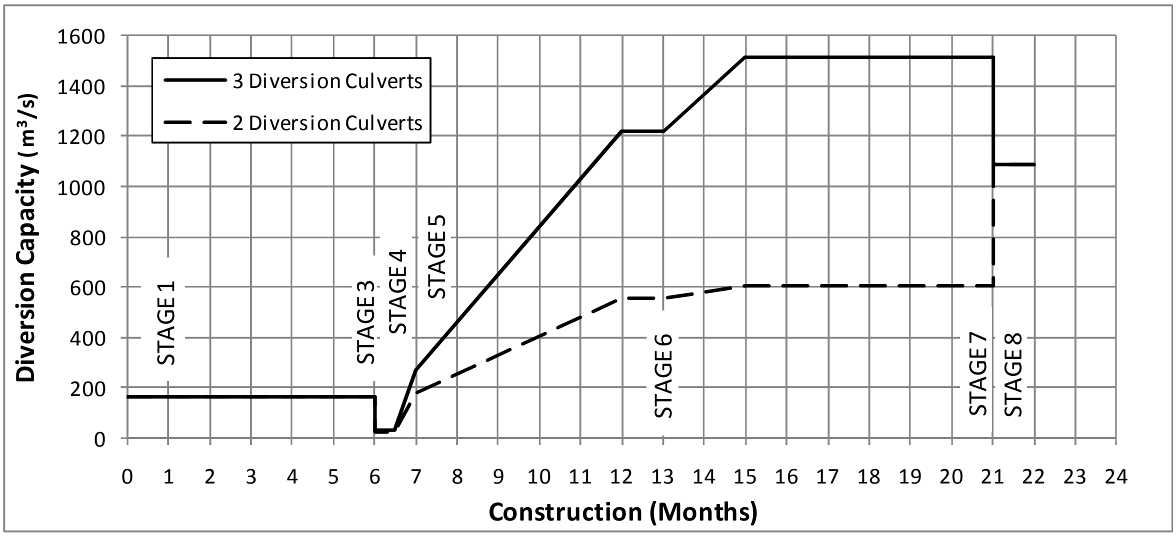

Figure 12-6: Construction stage and diversion capacity

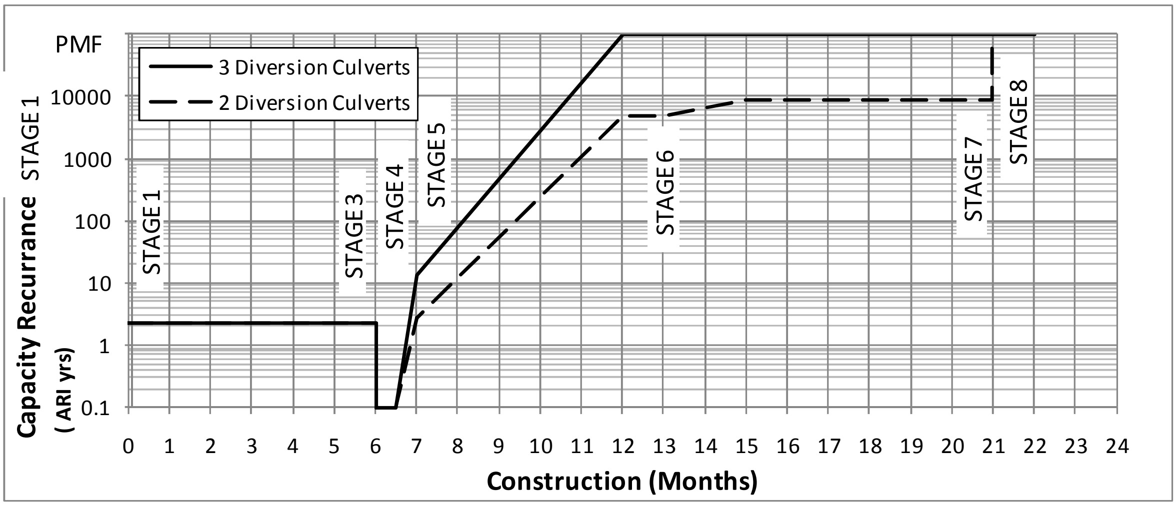

Figure 12-7: Construction stage and recurrence interval of diversion capacity

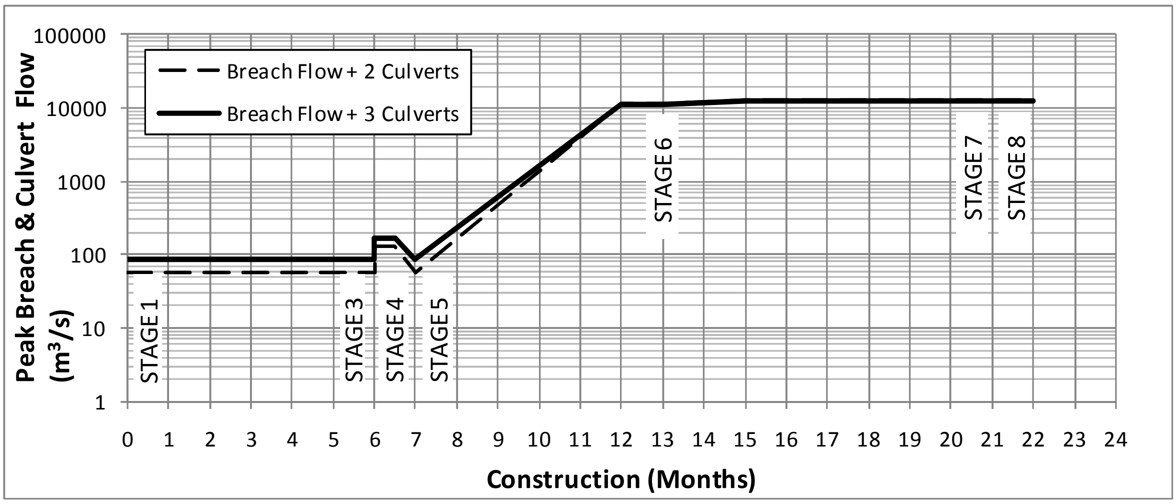

Figure 12-8: Construction stage and peak breach flows

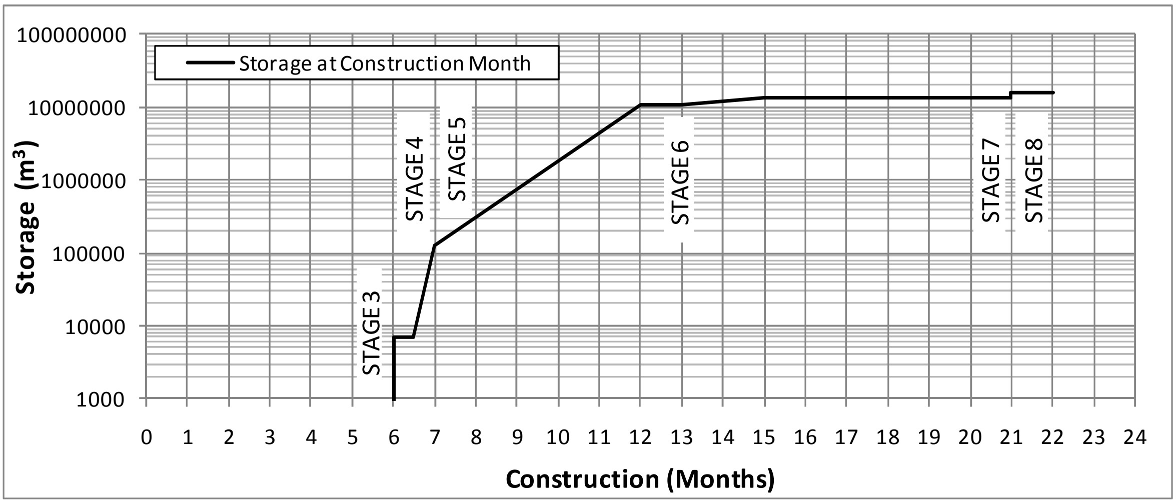

Figure 12-9: Variation in potential storage with construction stage

12.3 Site layout and facilities

An area downstream of the dam site on the left bank has been identified as suitable for site offices, workshops, concrete batching, and the like. The area upstream of the dam in the terrace elevated slightly above the river bed will likely form a good area for material processing for the embankment, and for aggregates. These areas are shown diagrammatically in Figure F-01.

12.4 Stage 1 – Preparation for diversion culvert

Equipment and staff will be mobilised to site and the site facilities (offices, labs, secure areas, etc) will be established.

Work at the dam site will start with river diversion. This will be achieved through culverts at the base of the dam. Low level works in the river bed will involve construction of a mass concrete starter dam which will form the plinth at the upstream toe of the main dam. The construction of this deep plinth will be undertaken in two stages to maintain river flows while works are being undertaken beneath current river bed level.

Preparation for the diversion culvert, located on the right bank of the existing river, will start early as this will undoubtedly be a critical path activity. The upstream end of the concrete culvert is located within the concrete starter dam referred to above so the construction of these two elements will be integral, and will need careful staging to maintain control over the river. The river will be pushed to the left of the current river channel (some localised left back excavation may be necessary) and a sheet pile wall and temporary coffer dams established to allow excavation for, and construction of the starter dam section. The bench for the culvert will be excavated into the right river bank and excavated material will be taken for direct placement in the coffer dams (right bank area).

A sketch of the activities during this stage is shown in Figure 12-10.

12.5 Stage 2 – Diversion culvert construction

With the bench prepared and the short section of starter dam constructed, the reinforced concrete (permanent) culvert under the embankment will be constructed. The areas of the embankment footprint accessible above water level will be stripped to founding level, and the excavated material taken to downstream dumping areas. Sediment control measures will be developed in parallel with the stripping. Some material processing of gravels may also be started during this stage and stockpiled for later use in the embankment.

A sketch of these activities is shown in Figure 12-11.

12.6 Stage 3 – River diversion

Once the permanent culvert is complete, the coffer dams will be removed and rebuilt in the left bank area, diverting the river through the culverts. Once this is complete the left bank section of the starter dam will be constructed to full height. A sketch of activities during this stage is shown in Figure 12-12.

12.7 Stage 4 – Plinth construction, start of embankment

With the river bed in the region of the dam dry, the remainder of the foundation can be stripped, and excavation/construction of the plinth can begin. Excavation of the spillway can start in parallel, with direct placement to the downstream shoulder of the embankment. This will also protect the embankment area from downstream water levels during flood events. The rockfill on the downstream shoulder may be reinforced with steel to improve resistance to excessive construction flood events. During this stage, some HDPE liner may be placed on the upstream face, if needed to assist with reducing throughflow during flood events. A sketch of these activities is show in Figure 12-13.

12.8 Stage 5 – Embankment completion and spillway construction

As excavation from the spillway cut proceeds, the embankment will increase in height (main rockfill zones) and construction of the concrete spillway will gradually progress as excavation allows. A sketch of these activities is show in Figure 12-13.

12.9 Stage 6 – Filter and embankment upstream face placement, completion of spillway

With the bulk excavations and embankment placement complete, upstream filter zones (2A and 2B) can be placed on the upstream face of the embankment, followed by slip-forming of the upstream concrete face. Completion of the concrete in the spillway structure will occur in parallel. A sketch of these activities is show in Figure 12-15.

12.10 Stage 7 – Intake conduits and diversion plugging

Once the upstream face is at least partially complete, the upstream face intake conduits, and some works for the intake gate structures (in the diversion culverts) can take place. This will involve swapping flow back and forward between culverts to allow gate construction. This will be immediately followed by diversion plugging, removal of the upstream steel culverts and coffer dam (once spillway structures are complete) and filling of the reservoir can begin. Passage of residual flow during filling will be maintained through the irrigation outlets, and possibly through a sacrificial low level valve which will later be plugged (when the reservoir is high enough to flow out the lower irrigation outlet). A sketch of these activities is show in Figure 12-16.

12.11 Stage 8 – Fish passage, tidy up, commissioning

The final activities will be installation of fish passage structures, and general tidy-up of the project area. Commissioning activities will include monitoring dam performance during filling, and operational testing of gates, screens, and control systems.

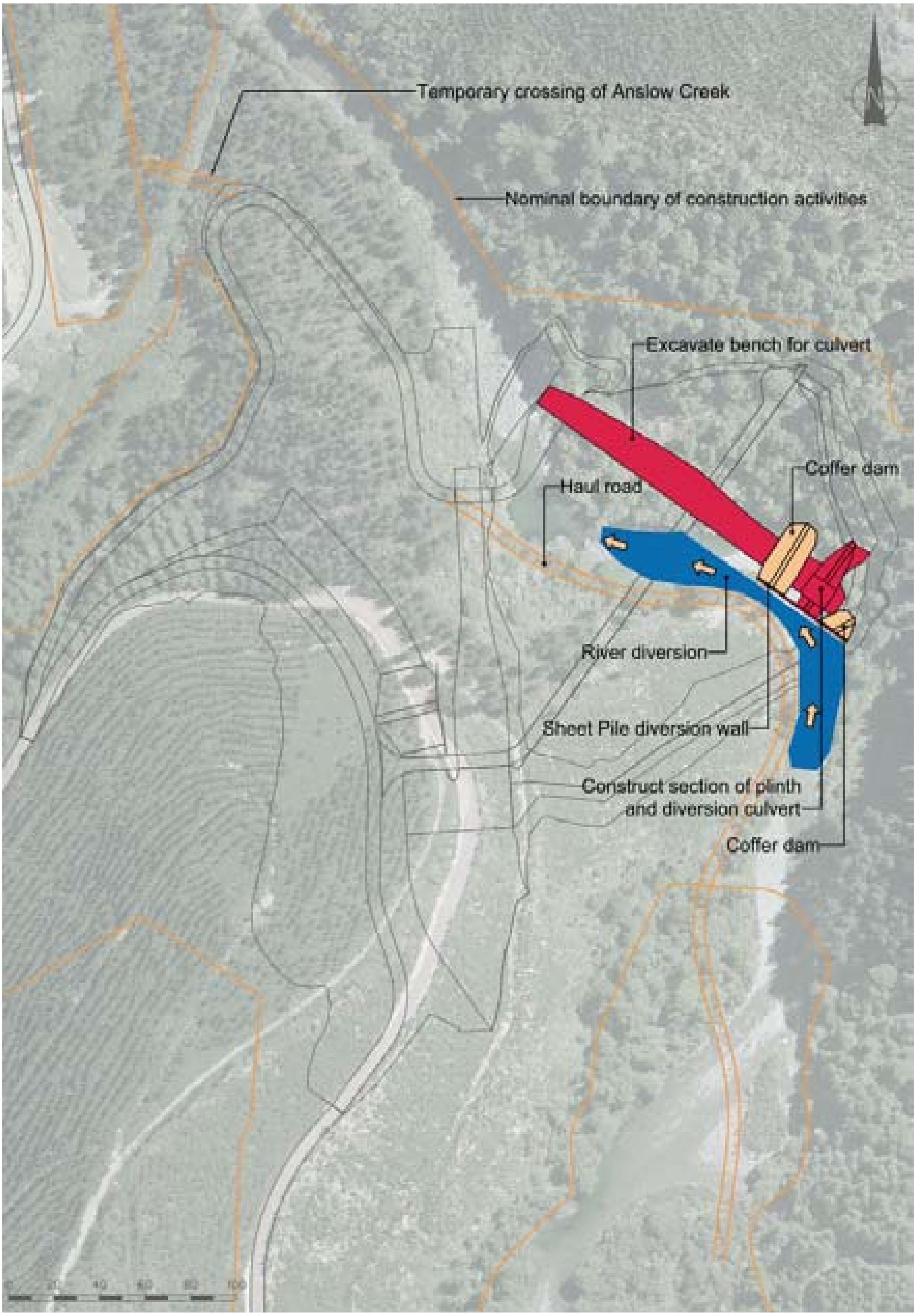

Figure 12-10: Sketch of Stage 1 of construction

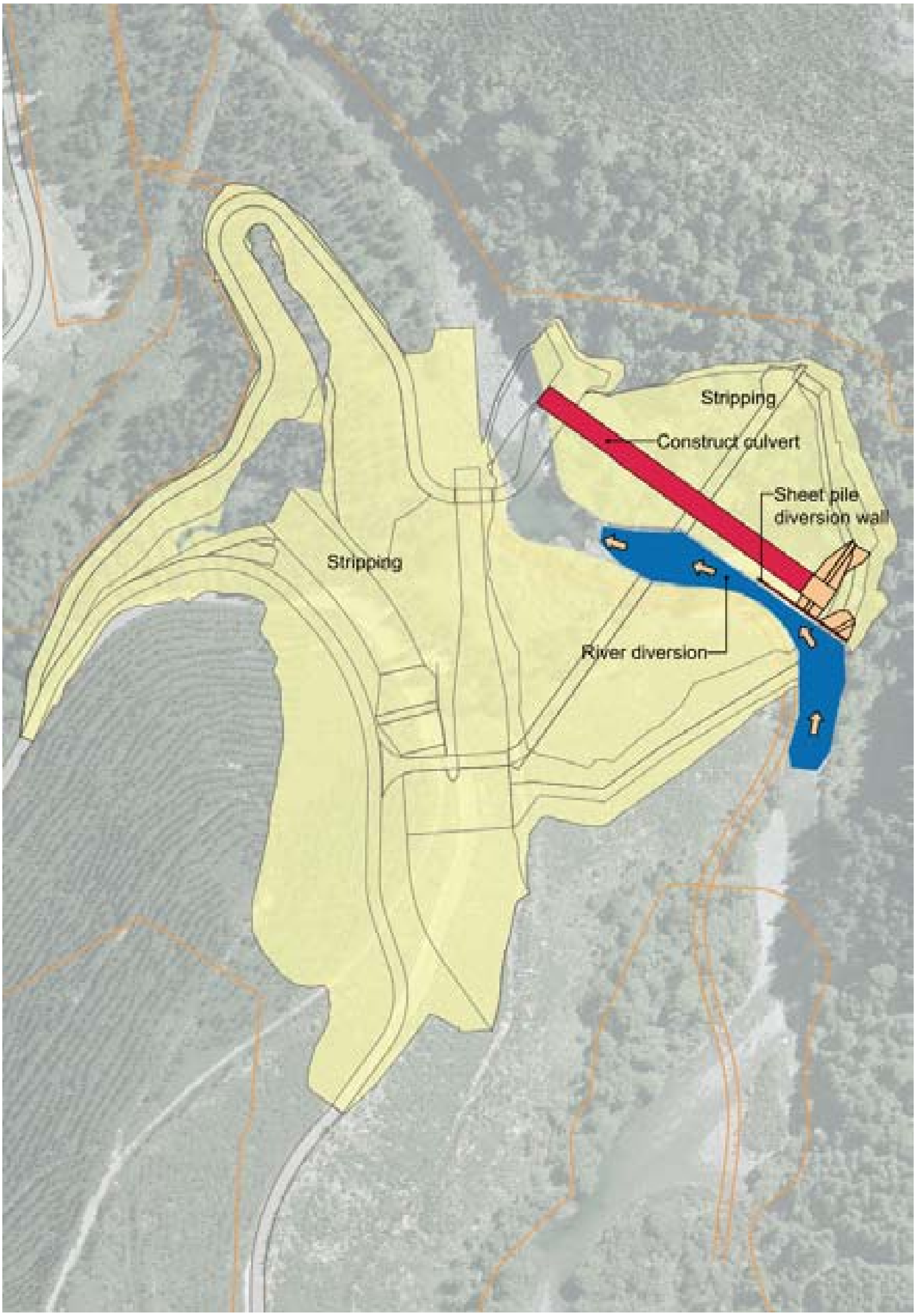

Figure 12-11: Sketch of Stage 2 of construction

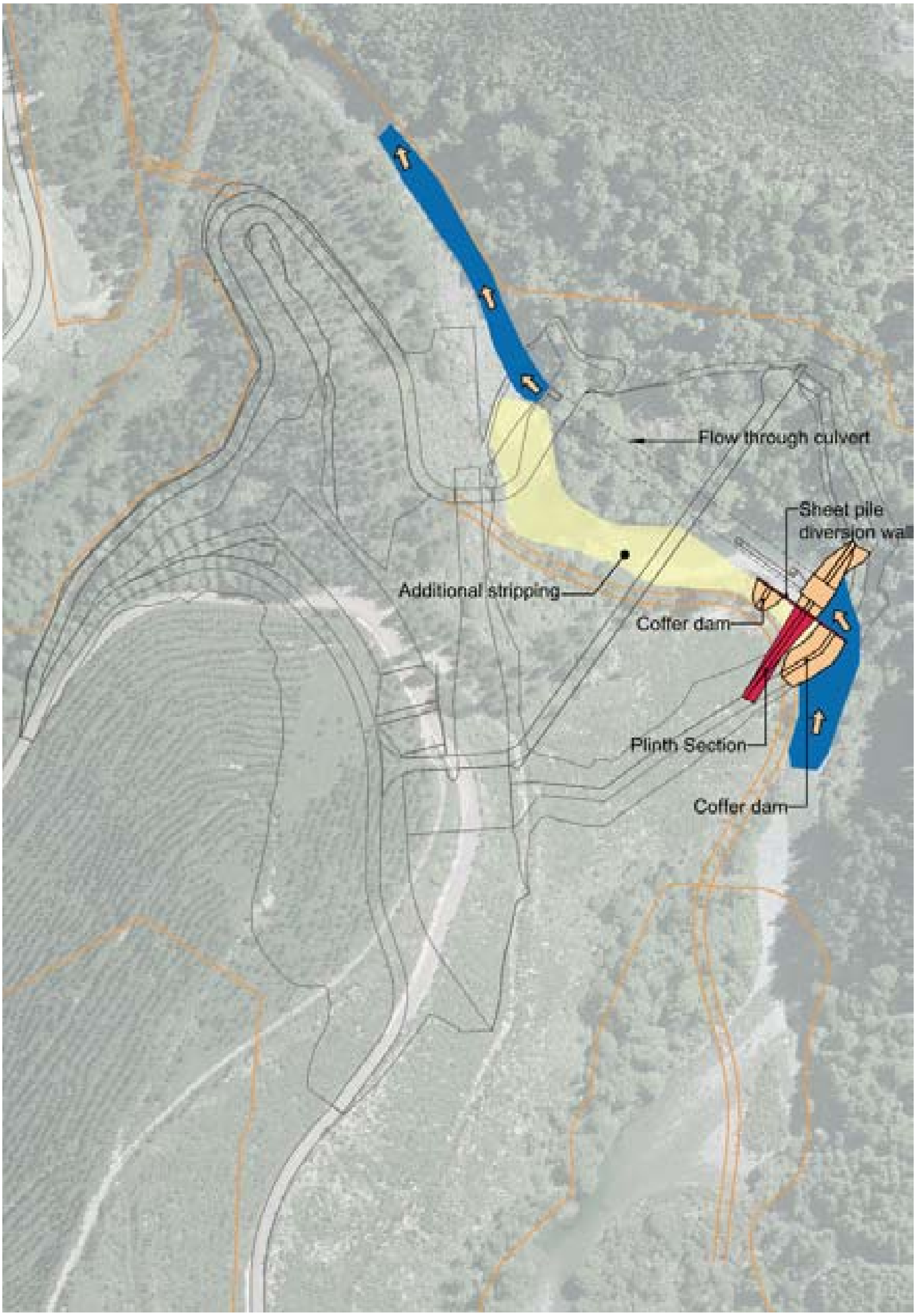

Figure 12-12: Sketch of Stage 3 of construction

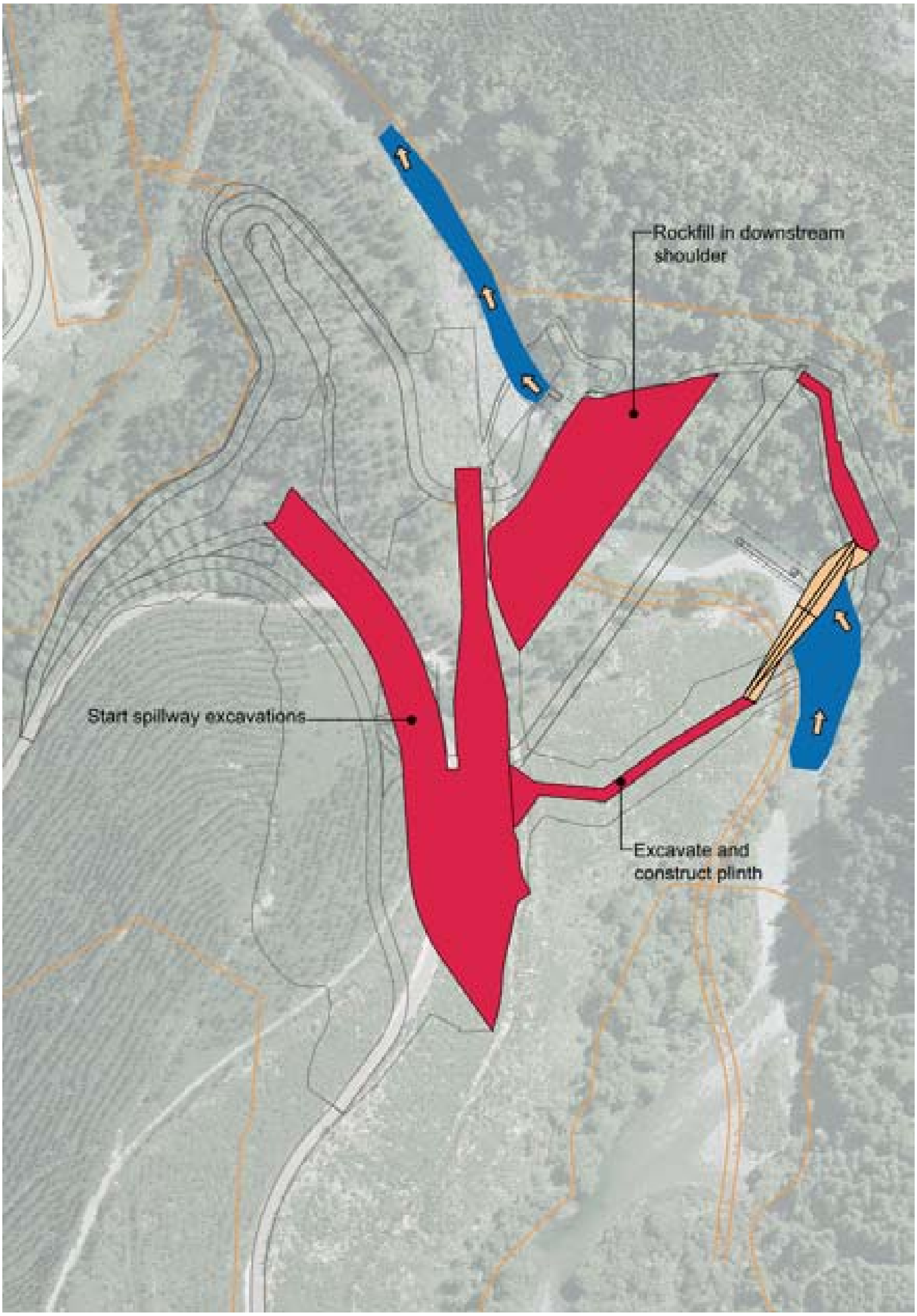

Figure 12-13: Sketch of Stage 4 of construction

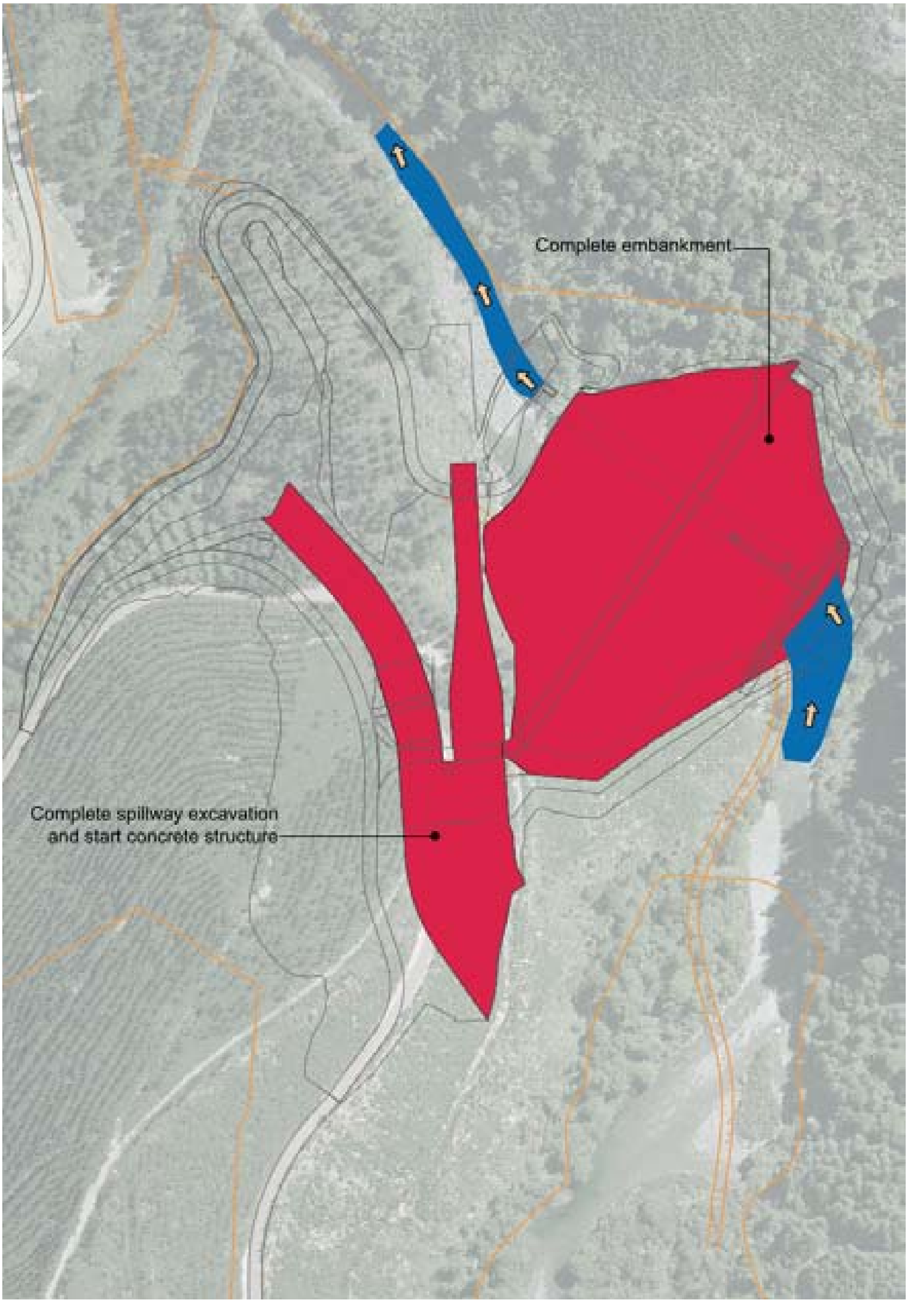

Figure 12-14: Sketch of Stage 5 of construction

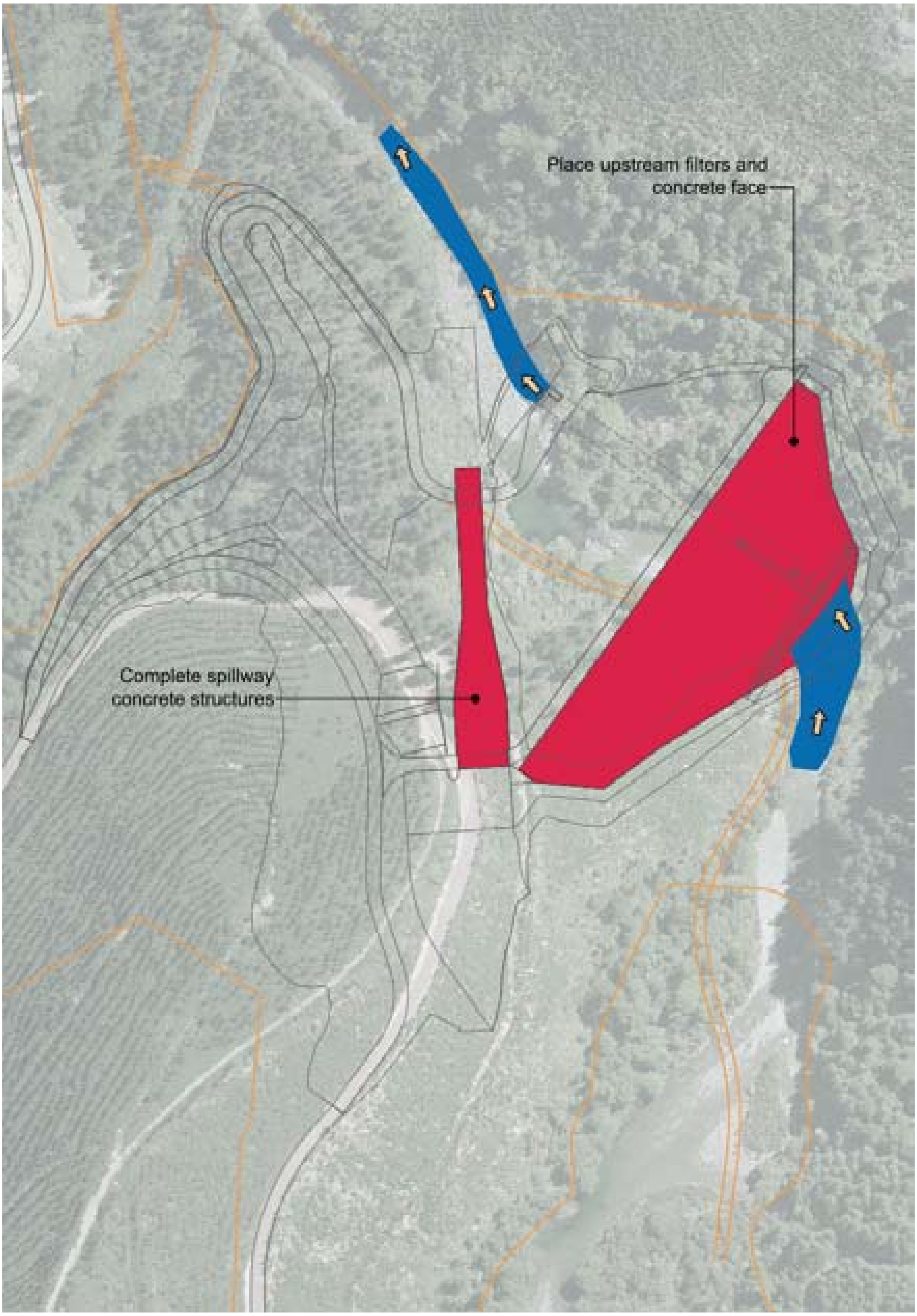

Figure 12-15: Sketch of Stage 6 of construction

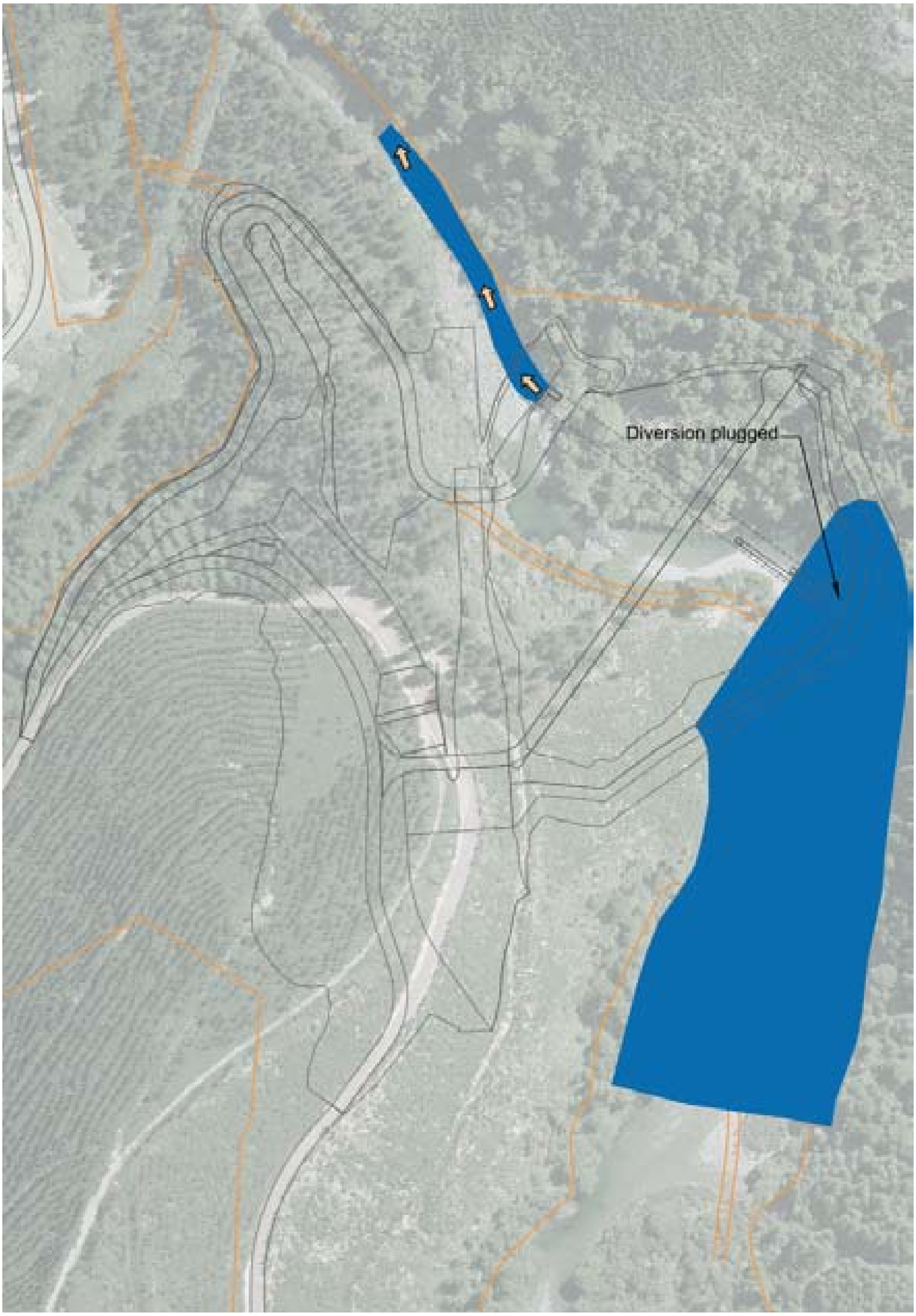

Figure 12-16: Sketch of Stage 7 of construction

12.12 Construction programme

An outline summary development programme for the dam is shown in Figure 12-17. More detail is provided in the programme in Appendix D.

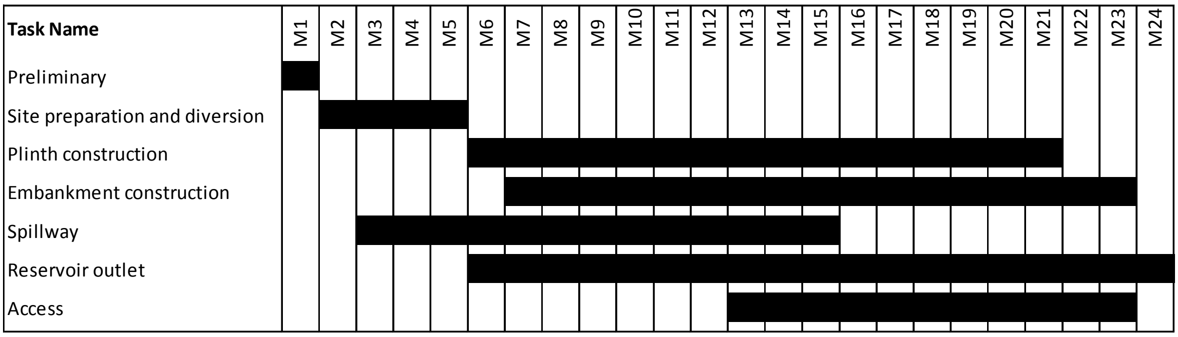

Figure 12-17: Summary outline construction programme

The overall construction period is expected to be of the order of two years.

13 Capital Construction Cost Estimate

Estimates of the capital cost for construction of the dam, and for potential hydro-electric generator add-on have been carried out. The estimation process for the dam itself has been more robust and detailed than that for the hydro add-on, which was based on historical guide prices for electrical-mechanical plant and for station arrangements. No specific investigation into the best type of plant has been carried out, and if hydro is considered to be attractive, an early re-assessment of costs should be carried out with an approach to plant suppliers.

The methodology adopted for cost estimation was developed to provide as reliable an estimate at the feasibility stage as possible. Following design of the structure (presented in earlier sections), quantities were estimated for all items and a bill of quantities developed. Likely rates were selected for items, based on previous tender and construction experience, and a base cost estimate built up. Percentages were allowed for contingency (20%), contractor's preliminary and general (15%), and design (10%). The total cost was arrived at with the following procedure:

1. The 'base cost' (BC) was estimated from estimated quantities and rates. This is the estimated amount that would be on the bill of quantities for physical construction items.
2. A 20% contingency was added to the base cost. This allows for unknowns that may be encountered during construction. Application of this brings the cost to 1.2xBC.
3. The contractor for the works needs to allow for management and overhead related costs (both on-site and off-site) and these are estimated at 15% of the base cost and contingency, i.e. P&G = (BCx1.2)x0.15.
4. Design and supervision of the works is estimated at 10% of the base cost and contingency, i.e. DES=(BCx1.2)x0.10.

The total cost is therefore made up from:

Base Cost	BC
Contingency	0.2 x BC
P&G	0.15 x (BC + 0.2 x BC)
Design	0.10 x (BC + 0.2 x BC)
Total	1.5 x BC

Costs were estimated for cases of two and three diversion culverts (as discussed earlier in the report). A significant portion of cost is attributed to diversion during construction as shown by this assessment. The actual requirement for diversion will need to be developed during detailed design and construction methodology development as part of a risk assessment, including contractor inputs. At the current level of design the cost is estimated to lie somewhere between the two figures quoted below.

These cost estimates were then reviewed by experienced people in the construction industry, who have been involved with bidding for and constructing similar works (Chris Hollingum, Earthworks and Civil Marlborough Ltd and Garth Townsend, The Breen Construction Company Ltd). Comments from this review were included in a revised estimate. Reviews were also undertaken internally by T&T, and by an independent external peer reviewer (Dr Trevor Matuschka, Engineering Geology Ltd).

The cost estimate for the dam (water augmentation only) as of November 2009 is:

NZ$35.5 million (GST exclusive) for 2 culvert diversion

NZ$38.1 million (GST exclusive) for 3 culvert diversion

The bills of quantities associated with these estimates are included in Appendix C. It should be noted that the estimate for construction cost is for the dam area only, and does not include any of the following costs which may be extra to the overall development cost:

- Taxes
- Insurance
- Developer related costs
- Resource consenting
- Environmental mitigation
- Land purchase
- Financing
- Distribution or allocation management
- Operation and maintenance
- Environmental compliance
- Construction cost variations due to high demand
- Increases in costs of steel, fuel, or any other construction related material
- Other items not specifically identified in the bill of quantities

The construction cost of the hydro-electric add-on has been estimated (to a low level of detail). Cost has been estimated for a range of installed capacity by interpolating between three turbine flow capacities.

The cost of transmission upgrade (as provided by Network Tasman Ltd) plays a significant part in the construction cost, and results in two steps in the cost profile. The cost of electrical and mechanical plant has been estimated on a per MW basis at this stage, and no enquiries have been made to plant suppliers. If the inclusion of hydro-electric generation in the scheme is attractive, confirmation of estimated prices should be undertaken as an early activity. The plot in Figure 11-7 show the variations in capital cost of including hydro for a range of installed capacity. At the recommended installed capacity of 1 MW, the total construction costs would be:

NZ$39.8 million (GST exclusive) for 2 culvert diversion

NZ$42.4 million (GST exclusive) for 3 culvert diversion

with the same assumptions and exclusions as outlined above.

The effect of reduced irrigation demand scenarios on construction cost has been estimated to allow consideration of sensitivity of cost to assumed demand. This analysis is presented in Appendix G.

14 Conclusions and Recommendations

This report (and the other associated technical reports including geotechnical and water resources) describes and summarises the work undertaken to assess the engineering feasibility of a water augmentation dam on the Lee River. Various sites and dam types have been considered, leading to the recommended solution of a concrete faced rockfill dam (CFRD) immediately upstream of Anslow Creek on the Lee River.

Geotechnical investigations have shown a lack of suitable nearby core material for a central core rockfill dam, leading to the recommendation of a CFRD. The selected dam site does not have the best storage/elevation characteristics in the study reach but it provides the most favourable geological conditions, both in terms of dam foundation and reservoir stability.

Our primary conclusion is that a CFRD at the selected location is technically feasible, based on the level of geotechnical investigation undertaken. We have undertaken a capital construction cost estimate for the dam, and subjected it to review by sub-consultants in the construction industry, and to external peer review. Conclusions regarding the economic feasibility of the overall scheme are provided in other reports.

The dam provides an opportunity to add hydro-power to the water augmentation outlet. Preliminary design and economic analysis indicates this addition has a positive return on investment and an installation involving two turbines (0.2 MW and 0.79 MW installed capacity) is recommended as optimum. This depends on some buffer storage availability.

Depending on the overall financial viability of the scheme, we recommend that the next (detailed design) stage of the scheme should include the following activities:

- more detailed assessment of hydro-power add-on including seeking costs for supply of plant from manufacturers;
- confirmation of normal top water level for dam, including any allowance for hydropower optimisation, and peaking;
- further geotechnical investigation and testing, including full valley width trenching at plinth, and testing of proposed rockfill materials;
- risk assessment of construction process, especially including river diversion, with optimisation of diversion strategy;
- assessment of the contractual delivery method for the dam (traditional, alliance, design-build, etc); and
- depending on the above, detailed design, documentation and tendering.

15 Applicability

This report has been prepared for the benefit of Waimea Water Augmentation Committee with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

TONKIN & TAYLOR LTD

Environmental and Engineering Consultants

Report prepared by: Authorised for Tonkin & Taylor by:

Robin Dawson

Senior Water Resources Engineer

John Chesterton

Water Resources Engineer

Report reviewed by:

Alan Pickens

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Appendix A: Options Assessment and Preliminary Option drawings

A1 General

The arrangement of a dam includes an almost infinite number of combinations of spillway type, embankment type, freeboard allowance, and outlet systems. For example if a large freeboard is provided between the crest of the spillway and the crest of the embankment, more flood water can be temporarily stored in the reservoir and the flow capacity of the spillway can be reduced. This will result in a lower cost spillway, but a more expensive embankment. The material that is excavated from the spillway area may be used in embankment construction if the quality is suitable. Depending on the volumes required for embankment construction this may make spillway excavation very inexpensive, and lead to a large shallow auxiliary spillway being attractive. In addition, works which have been used for temporary flood diversion during construction may be able to form part of a permanent spillway system.

A change in each component of a dam clearly affects a lot of other components, and the optimum arrangement can be elusive. The limitations in available information at the feasibility design stage also need to be recognised, and the potential for necessary changes during the detailed design stage to address an issue which is currently not apparent needs to be appreciated.

We have undertaken a preliminary/scoping level design and cost assessment for a range of combinations of embankment height (available freeboard), spillway type and size, and rockfill material quality. This process has developed curves of approximate cost for embankment and spillway components and options, with a primary aim of selecting the embankment crest level, and the spillway type and size. While the cost curves developed do not include all the components of the dam, they do include costs which are specific to individual options allowing reasonable comparisons to be made. As such these curves should be considered a ranking process rather than development of absolute construction costs.

The selected arrangement is then subjected to a higher level of preliminary design and costing, as discussed in section 11.

A2 Embankment Options

The embankment forms the primary barrier to the stored reservoir and needs to remain stable and serviceable under the various loads of water pressure, seeping water, flood rise, reservoir waves, earthquake loading, and other foreseeable loads.

Earlier stages of this assessment considered the relative merits of three generic options for the embankment component, which were:

- zoned earthfill embankment
- concrete faced rockfill dam (CFRD)
- roller compacted concrete (RCC) dam.

A brief recap of the dam types considered in the previous stage is provided in the following sub-sections, along with the primary conclusions and decisions for progressing to the current assessment stage.

A2.1 Zoned Earthfill Embankment

A zoned earthfill embankment includes a central core of low permeability material as the primary water retention element. Developing this type of dam economically relies on the availability of suitable borrow materials within reasonable haul distance. Filters are provided downstream of the core to provide protection against internal erosion through the core. The upstream and downstream shoulders are constructed from rockfill. Note that additional transition zones may be required on the upstream side of the core depending on the size of the rockfill used. The cross section shape indicated in Figure App 1 was adopted for comparative costing of sites and dam type options. The percentage of each material type (in terms of cross sectional area) is also indicated, which has been used as a basis for quantity evaluation.

The additional cost of spillways was based on preliminary spillway design and costing carried out during the Phase 1 assessment for the Lee River site (with allowances for the variations in dam height) which will allow general site and type comparisons.

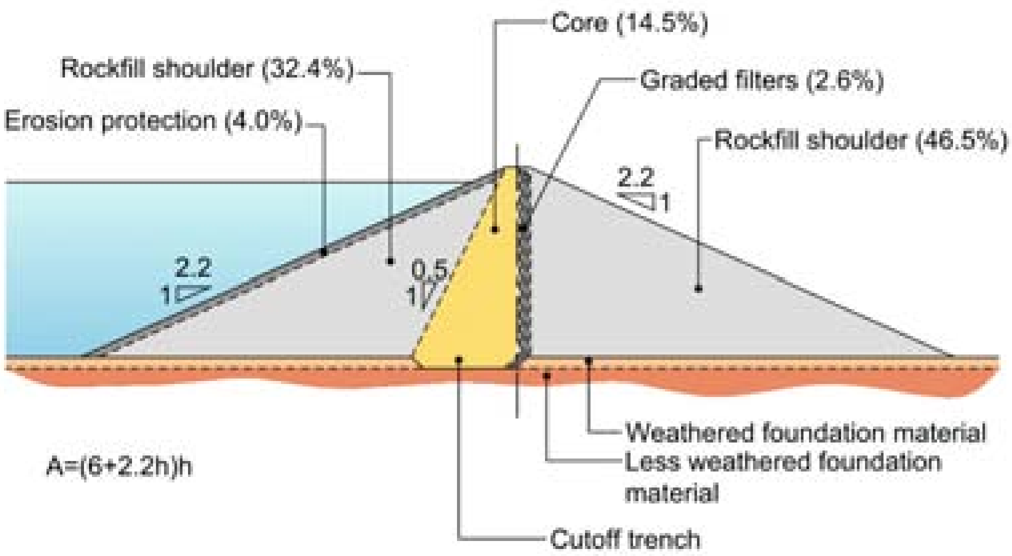

Figure App 1: Outline cross section for zoned earthfill embankment dam

A2.2 Concrete Faced Rockfill Dam (CFRD)

The CFRD includes a concrete upstream face as the primary water retention element. The use of high strength rockfill and near complete exclusion of pore water pressures in the body of the dam allows steeper embankment slopes to be adopted reducing overall material quantities, but the concrete upstream facing can be an expensive item, requiring extensive jointing and water-stopping. Transition zones are included downstream of the concrete slab that support the slab, and act to control leakage in the event of cracking on the face or opening up of joints. The outline design adopted for the current assessment is shown in Figure App 2.

The face slab is constructed from reinforced concrete, generally between 0.25 and 0.6 m thick (0.5 m thick was adopted for cost comparison evaluation). The face slab incorporates vertical, horizontal and perimetric joints to accommodate deformation which occurs during construction and when the water load is applied.

The face slab is connected into the rock foundation via the plinth, which is a concrete slab cast against and bolted to a prepared rock surface at the upstream toe of the embankment.

Zone 2D is a processed rockfill or alluvium, grading from silt to cobble or gravel size. The zone provides uniform support for the face slab and acts as a semi-impervious layer to restrict flow through the dam in the event that cracking of the face slab or opening of joints occurs. A zone width of 4 m was adopted for this evaluation.

Zone 2E is a selected fine rockfill which acts as a filter transition between Zone 2D and Zone 3A in the event of leakage through the dam.

Zone 3A is quarry run, free draining rockfill placed in layers about 1 m thick. This zone provides the main support for the face slab and is compacted to a high modulus to limit settlement of the face slab.

Zone 3B is a coarse, quarry run, free draining rockfill placed in layers about 1.5 to 2.0 m thick. Larger rock may be pushed to the downstream face. This zone is less affected by the water load than Zone 3A, so a lower modulus is acceptable. The thicker layers allow placement of larger rock.

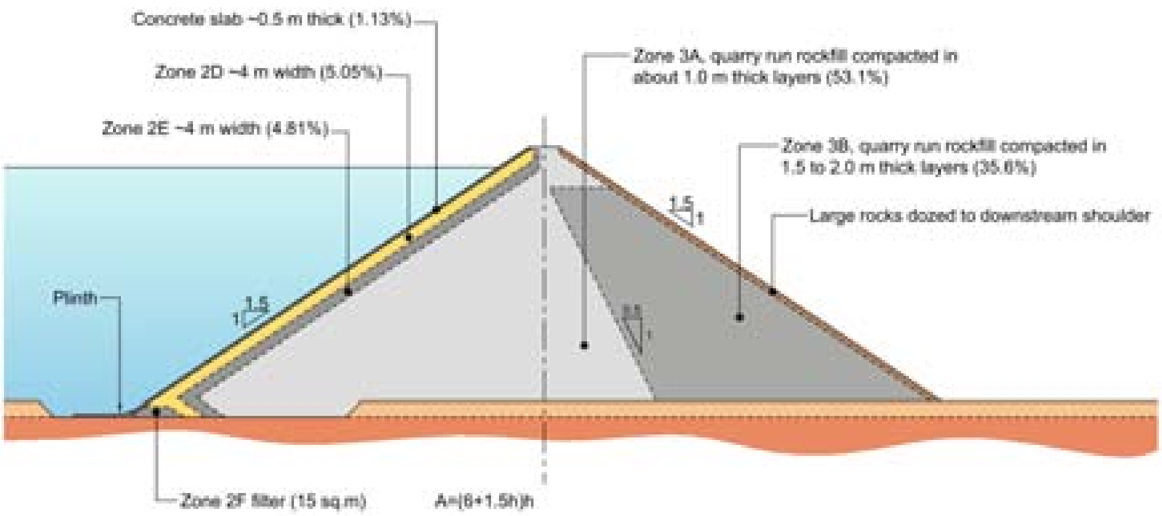

Figure App 2: Outline cross section for concrete faced rockfill dam

It should be noted that the figures above for an embankment dam and CFRD assume that the foundation is sound. If deeper permeable material is found during detailed design investigation to exist in the foundation, significant additional seepage cutoff works may be necessary. In the case of the plinth for the CFRD, a very competent foundation is required to minimise deformation and potential for leakage. Foundation treatment for all dam types has at this stage been assumed as subexcavation to competent ground, and no foundation grouting has been allowed for.

A2.3 Roller compacted concrete dam

Concrete gravity dams are formed from mass concrete and rely upon the weight of the structure to provide sufficient resistance to hydraulic and seismic loads in terms of base sliding or overturning. Traditionally mass gravity dams have been constructed from conventional concrete, at considerable cost. Concrete can also be placed using earthmoving techniques (roller compacted concrete) offering significant savings over conventional concrete. By far the majority of concrete gravity dams either currently or recently constructed adopt the roller compaction method due to the significant improvement in economy. Only roller compacted dams were considered during this evaluation due to their considerable economic benefits over conventional concrete.

Roller Compacted Concrete (RCC) describes concrete which combines the economical and rapid placing techniques used for fill dams, with the strength and durability of concrete. RCC has a no-slump consistency in its un-hardened state and is transported, placed, and compacted using earthworks construction equipment. The properties of hardened RCC are similar to those of traditionally placed concrete.

The concrete dam proportions shown in Figure App 3 were adopted for the earlier assessment of material quantities for an RCC dam. A high cementicious content approach has been adopted, and central spill and stilling basin have been allowed for spillway passage. Conventional concrete is used in the region of the spillway and stilling basin.

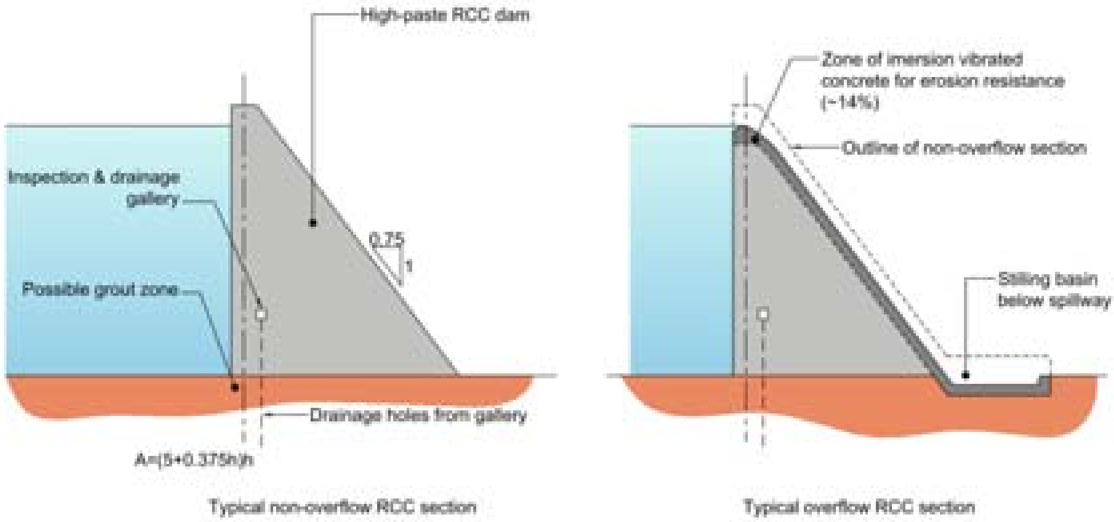

Figure App 3: Outline cross section for RCC gravity dam.

A2.4 Proposed embankment type

Based on initial costings RCC dams did not appear to be competitive, and showed a high risk to construction cost increases due to the rate adopted for RCC. Zoned earth fill embankments appeared competitive in price provided a suitable core material could be located within reasonable distance. Investigation to date has not revealed any nearby material that is suitable, with the nearest material very distant (refer to the Geotechnical Report for discussion of available materials). The expected cost of transport made the embankment dam less favourable. There would also be significant environmental effects from the large number of vehicle movements and durations that would be necessary.

The CFRD was cost competitive with embankments, and the materials in the region appeared suitable for use as rockfill. Accordingly, the CFRD was selected as the favoured embankment type, with zoned embankment as a second option. The least competitive option is the RCC, primarily due to its higher cost. It remains the ultimate fallback, however.

The assessment presented in this report is focussed on the CFRD type embankment, although comment on other types being included for consenting are provided. It is likely that the final decision on embankment type cannot be made until the detailed investigation and design stage.

As part of the process of developing the preferred arrangement of embankment and spillway, preliminary design and cost estimation has been carried out for three embankment crest elevations; RL 200m, RL 202m, RL 204m.

A3 Construction flood diversion options

During the construction of the dam, the Lee River will need to be diverted. The steepness of the terrain and height of the dam narrow the options to closed conveyance methods such as tunnels or culverts, as opposed to an open channel diversion. A tunnel option was evaluated early on as being prohibitively expensive. Therefore a culvert diversion option has been selected as the most practical and cost effective.

A3.1 Outline design criteria

As for the main embankment, the diversion design criteria are determined by weighing the probability and consequence of failure against an acceptable level of risk. The consequence of failure increases as the embankment construction progresses while the probability of failure is reduced.

ICOLD Bulletin 48a (1986) comments that a return period flood of 50yrs or more be used for earth fill dams during construction as they may be completely destroyed if overtopped. However, armouring in the downstream slope of a CFRD (rockfill) embankment can enable the construction to withstand some overtopping flow and allow a much reduced low level diversion capacity.

Following ICOLD recommendations, for the current design stage, the 50 yr flood was selected as the design storm for diversion. Assuming a construction design period of 1 year, the probability of a 50yr flood being exceeded would be 2%.

A maximum head water level of 15 m was assumed for outline design of the diversion.

Note that these assumptions were made during the options assessment stage. Refer to Section 12 for a more detailed assessment of flood protection.

A detailed risk assessment will be undertaken during detailed design. This will evaluate the increasing potential impact category combined with the commercial risk of the construction as it progresses to determine the appropriate diversion flow requirements. This will allow further opportunities for optimisation and the inclusion of multiple diversion components. For instance, an initial diversion could be constructed to withstand a 5yr flood given the lower consequence of failure, until the dam was robust enough to withstand a 50yr flood. The culvert diversion would increase in capacity with increasing head and storage and additional diversions such as abutment spillways or some allowable overtopping of the dam construction could be provided.

A3.2 Diversion alignment

The location of the diversion was selected to ensure that the majority of the diversion culvert would be able to be constructed beside the river, possibly requiring changes to the alignment of the lee river. As the diversion is also intended for use as a conveyance culvert for the outlet pipework and flushing flows, the location of the intake and outlet works were also taken into account.

An alignment on the right bank was selected as a left bank alignment could interfere with the spillway construction. Also, with a spillway on the left bank, the power station would need to be located on the right bank.

The alignment of the diversion underneath the main embankment was situated to allow its construction in the dry. The alignment underneath the embankment would be built as an integral part of the dam and culverts would either remain to convey outlet flows or be plugged with concrete.

To construct the plinth and lower upstream face of the CFRD, a temporary coffer dam may be needed upstream of the main embankment. An additional section of diversion culvert would be incorporated to take flow through an upstream coffer dam and would potentially be removed or abandoned together with the coffer dam after or during construction. Alternatively, a concrete cofferdam could be incorporated into the construction of the plinth and may be investigated during the detailed design phase.

A downstream cofferdam may also be required to prevent flood rise in the downstream channel entering the construction site. This coffer dam would be incorporated into the downstream toe of the embankment.

The total length of the diversion alignment selected for the outline design is approximately 248 m. The alignment will match the river bed levels at its inlet and outlet of 151.5 mRL and 149 mRL respectively, giving the diversion an overall grade of 1%.

A3.3 Culvert size and cofferdam height

Rectangular box culverts were used for the design of the outlet channel as they may be possible to precast in sections close to the site reducing cost of fabrication and transportation when compared with circular culverts. The culverts were designed with a height to width ratio of 2:1 to accommodate the difference between in vertical and horizontal loading.

The diversion culverts were sized to pass the 50yr flood with a maximum coffer dam height of 15 m. A stage discharge relationship was developed for a single box culvert using the weir and orifice control equations provided in the FHWA Hydraulic Design Series Number 5 (2005).

The 50yr synthetic hydrograph developed in Section 7 was used together with the stage discharge relationship for a dam situated at chainage. 12600 m to route the flow through the culvert and determine the maximum stage.

Limiting the maximum practical headwater level to 15 m and varying the number and size of culverts resulted in the selection of three, 2.5 m wide by 5 m high culverts to give a maximum routed flow depth of 12.4m at the inlet end. The maximum flow through each culvert during the 50 yr flow event would be 103 m^3/s giving a maximum culvert velocity of 8.24 m/s.

Given a ponding depth of 12.4 m at the inlet to the diversion and adopted outline design criteria, a coffer dam would be built with a minimum crest elevation of 164 m.

The height and configuration of the coffer dam and culvert size may need to be revised once a more detailed risk assessment is undertaken and during the detailed design of the diversion and outlet works.

A4Spillway Options

The objective of the spillway system is to safely pass the design storms from the reservoir to the river downstream of the embankment. The spillway must be proportioned to pass the design flood with a rise in reservoir level that matches the freeboard available between the spillway crest and the embankment crest. Flow discharged from the spillway outlet has significant energy (velocity) which must be dissipated prior to release back to the river to minimise river scour to acceptable depths.

A variety of spillway options have been considered for the Lee Valley Dam to arrive at an apparent optimum configuration. To assess these options, the following aspects were considered:

- spillway purpose and design criteria
- spillway types
- reservoir routing and rise for OBF and MDF
- spillway location and configurations
- energy dissipation
- spillway construction costs
- impact on embankment construction costs.

A4.1 Spillway design criteria

The purpose of the spillway is to provide an economical means of passing flood flows, balancing the flow capacity with the flood rise and embankment height above NTWL. Although peak flood flows are expected to be reduced, this is not a controlling requirement.

The adopted standards for the Lee Valley Dam are listed in section 5.3. Considering the design standards adopted, and the expected wind generated wave heights discussed in Section 8.2, the flood rise during OBF should come no higher than 0.92 m below the embankment crest, and for the MDF no higher than 0.42 m below the crest.

A4.2 Spillway design philosophy

Spillways can form an extremely expensive component of a dam structure and careful consideration of their type and arrangement is necessary to minimise overall costs while providing the necessary flood passage requirements for safety. It is common practice in dam design to provide a combination of primary and auxiliary (or emergency) spillway.

The primary spillway will manage flood flows up to the OBF, which are expected to occur a few times within the life of the structure. The aim is to ensure flows are passed without any significant damage to the structure, and without the need for other than routine maintenance. Primary spillways are generally formed from concrete (or other non-erosive material) and include energy dissipation that limits the energy of the discharged flow to acceptable levels. Primary spillways can also include gates for spill control if these are considered economic.

The auxiliary spillway provides for much larger floods that are extremely unlikely to occur, but that need to be passed without dam overtopping if they do occur.

Given the very rare nature of these events, it is considered acceptable that there be some repairable damage to the auxiliary spillway during its use, provided that it does not lead to collapse of the dam and uncontrolled release of the stored reservoir. This design philosophy leads to auxiliary spillways commonly being unlined channels cut in rock, with the expectation of limited erosion occurring if they are used.

Auxiliary spillways commonly include a breachable embankment (or fuse plug) at their upstream end. This is a low embankment constructed from relatively erodible material in the downstream shoulder, and an armoured upstream face. When the reservoir level rises and overtops the breachable embankment, it rapidly erodes and opens up a large channel for auxiliary spillway flow. This is similar in nature to opening a hydraulic gate. The breachable embankment can then be rebuilt when the flood has passed and water levels have receded. To reduce the potential surge in flow downstream of such an embankment, they are often built in sections to allow for sequential breaching.

The combination of primary and auxiliary spillway has been adopted for the Lee Valley Dam. The primary spillway could be provided using a number of arrangements, and these are discussed in the following Section.

A4.3 Options for primary spillway

A number of options are available for the primary spillway. The selection of optimum option cannot be considered in isolation, as each will produce different quantities of rockfill excavation which may be used in the embankment, or may need to be cut to waste. Preliminary design for a range of spillway types has been carried out to provide a comparison of spillway costs, embankment costs, and other related factors to be considered in deciding the preferred arrangement.

The following primary spillway types have been considered:

- Ogee weir with chute at left abutment
  - Simple overflow weir, less prone to blockage and able to include provision for fish passage.
- Labyrinth weir with chute at left abutment
  - Overflow weir with zigzagging crest. A long weir in a short space with higher capacity at small flows than an Ogee weir of similar overall width.
- Bell-mouth weir and dropshaft with conduit release under embankment
  - A tower with inverted bell shaped entry connecting the reservoir with the diversion conduits minimising the amount of excavation required. Can be prone to blockage.
- Bell-mouth weir and dropshaft with release through left bank tunnel
  - An inlet tower connecting the reservoir to a tunnel through an abutment.

Gated spillways were not considered economic for Lee Valley Dam as they provide a level of reservoir control not required at this stage.

A4.4 Spillway entry hydraulics and sizing

App 1.1.1 Introduction

Hydraulic calculations were carried out for a range of spillway types and allowable flood rise heights. Three embankment crest elevations were adopted (to match the embankment cost estimates carried out), effectively setting the maximum allowable flood rise under OBF and MDF conditions. Flood routing was then carried out and spillway sizes were than estimated to provide the required flood flow characteristics. Each spillway type is considered in the following sub-sections. Approach channel head loss and effect on the weir hydraulics is addressed in more detail in section 11.

A4.4.1 Ogee weir control

A straight ogee or nappe-crested weir was evaluated to determine the length required to pass the design flows. The nappe-surface profile is a proven and efficient weir type and requires little maintenance.

The ogee weir discharge is governed by the weir equation:

$Q = C_d L H^{3/2}$ Where Q = flow in cubic meters,

Cd is the discharge coefficient,

L = weir length, and

H = total approach head of the flow.

The discharge coefficient for the Ogee weir was evaluated using the values provided in the "Applied fluid dynamics handbook", page 209 (Belvins 1984). For all options it was assumed that there was adequate capacity to ensure no downstream submergence effects on the weir.

The weir crest was set to the NTWL of 197 m RL and the design head for the weir set at the maximum reservoir rise during the OBF. The discharge coefficient was then interpolated over the range of heads experienced during the design storms to give a stage discharge relationship for each embankment crest height. The width of the weir could then be calculated for the allowable rise given an embankment height or crest level.

This was done by routing the 200yr synthetic hydrographs (Section 7) though a reservoir located at chainage 12600 and calculating the maximum rise for a given weir width. Table App 1 gives the minimum weir width requirements for the OBF. These weir widths were then used in the calculation of the auxiliary weir capacity required with the additional 0.5 m reservoir rise available during the MDF.

Table App 1: Straight Ogee Weir Sizing for OBF

Dam Crest Level (m RL)	OBF Flood Rise (m)	Crest Length (m)	Design Discharge (m3/s)
200	2.077	59.5	394
201	3.077	32.0	384
202	4.077	20.5	372
203	5.077	14.5	360
204	6.077	10.54	345

A4.4.2 Labyrinth weir control

Labyrinth weirs have high discharges at low stages due to their long crest length. A labyrinth control structure was investigated for a range of heights and labyrinth 'cycles' for a flow of 400m^3/s to evaluate the best option. The cycle angles evaluated were 15, 18, 20 and 25 degrees and for a particular head, the cost reduced with increasing cycle angles. A cycle angle of 20 degrees was chosen for flow routing, balancing the cost against the need to minimise the width of the spillway. The method outlined by Tullis (1995) was used to calculate the flow over a half round labyrinth weir crest.

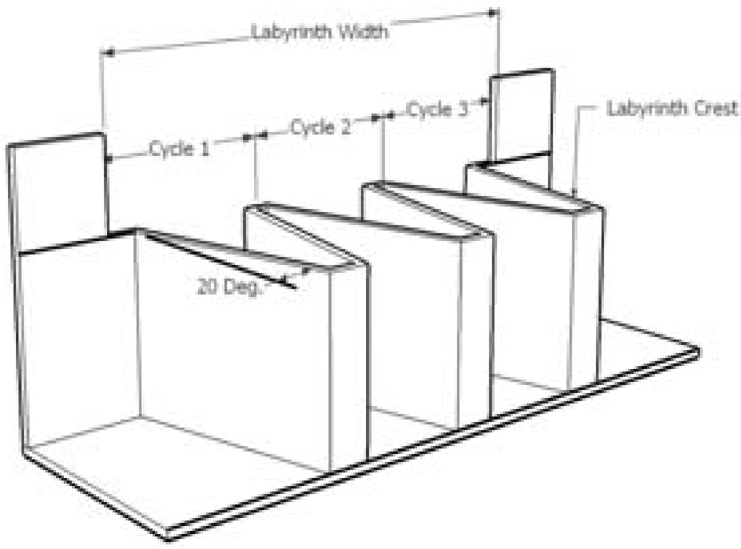

Figure App 4: Layout and Details of Labyrinth Weir

The discharge equation provided by Tullis as follows is similar to the standard weir equation.

$$Q_L := C_T \cdot L \cdot \frac{2}{3} \cdot \sqrt{2g} \cdot H_t^{1.5}$$

Where: Q_T = Weir discharge

C_T = the labyrinth weir discharge coefficient

L = the length of the weir along the labyrinth crest

H_t = the total upstream head over the crest

The depth in the approach channel with respect to weir height, or height of weir was assumed to be 3 m. The ratio of head to approach depth was used to calculate a weir coefficient taken from Figure 3, Design of Labyrinth Weirs, Tullis (1995). Similar to the ogee weir, the labyrinth crest was set to the NTWL of 197 m RL and the stage discharge relationship was calculated for various embankment heights and allowable reservoir rises.

Once the stage discharge relationship was known, the design storm hydrographs were routed through the reservoir to determine the weir lengths required for the options investigated. When a weir length had been established for a particular scenario, a cycle configuration was determined. For all options, three cycles were used although this could be adapted with little impact on the capacity or cost at a later stage.

Table App 2 shows the labyrinth crest length, overall crest length and design discharge for the range of embankment crest heights.

Table App 2:Labyrinth ($\alpha = 20^o$) Weir Sizing for OBF

Dam Crest Level (m RL)	OBF Flood Rise (m)	Crest Length (m)	Weir With (m)	Design Discharge (m3/s)
200	2.077	74.9	34	388
201	3.077	46.3	21.9	373
202	4.077	32.9	16.2	357
203	5.077	25.1	12.9	341
204	6.077	19.6	10.6	327

A4.4.3 Bell-mouth outlet

A bell-mouth spillway consists of a vertical tube with a flared overflow inlet control connecting at the base of the reservoir into a relatively flat, closed channel or tunnel. Bell-mouth inlet hydraulics were evaluated following the design method outlined in Design of Small Dams, (USBR 1987). A circular nappe-shaped crest (Figure App 5) was adopted as the control for the feasibility design.

The equation for weir flow over the bell-mouth crest is as follows:

$Q = C_o(2\pi R_s)H_o^{3/2}$ Where: Q = Discharge over the weir

C_o = Coefficient of discharge

R_s = Outside diameter of the nappe crest

H_o = Hydraulic head of approach flow.

The discharge coefficient, Co is proportional to the approach flow depth, height of weir and radius of the bell-mouth. The coefficient was taken from Figure 9-57(Pg.-410) for a weir height to radius ratio (P/Rs) of 2.

The discharge characteristics of the spillway are expected to change with increasing head. As the reservoir level gets higher the control may move from the weir to become orifice, tube, or ultimately full pipe flow controlled. USBR (1987) advises that weir flow will govern until a head to weir radius (H_0/R_s) of 0.45 at which point the flow will become partly submerged. Orifice control will govern when this ratio reaches 1 after which there is less increase in spillway flow with increasing head. Under orifice conditions the spillway conduit must be aerated to prevent pressure flow and siphoning. Vortex suppression vanes are often incorporated onto the crest of a bell-mouth spillway as the development of vortex flows are typically deemed undesirable.

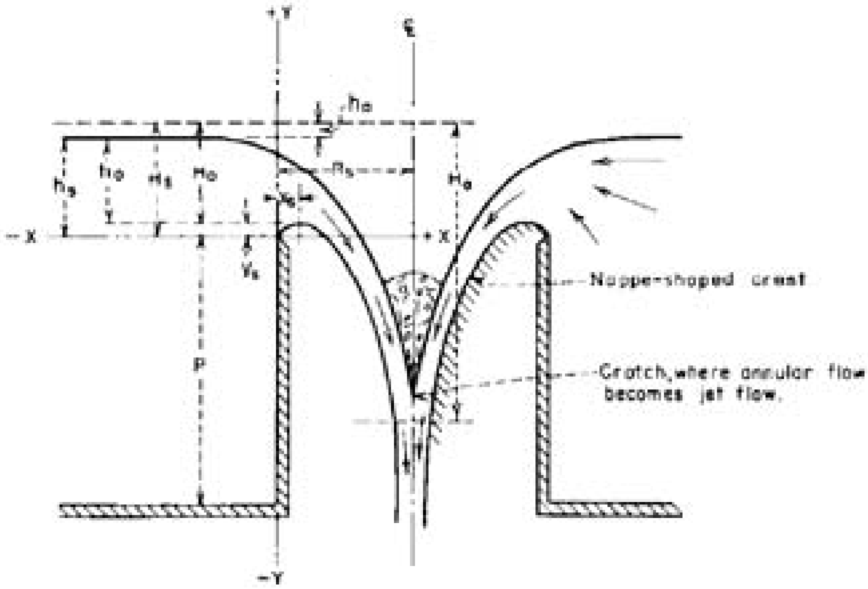

Figure App 5: Elements of nappe-shaped profile for a circular weir (Design of Small Dams Figure 9-56 (1987))

The lip of the inlet was set at the NTWL of 197m and diameter of the inlet calculated for the allowable rise given embankment crest heights of 200 m RL to 204 m RL.

This was done by routing the 200yr synthetic hydrographs (section 7) though a reservoir located at chainage 12600 and calculating the maximum rise for a given circular crest diameter. The bell-mouth diameters and resulting discharge for given heads is given in Table App 3.

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Table App 3: Bell-mouth Sizing for OBF

Dam Crest Level (m RL)	OBF Flood Rise (m)	Bell-mouth Diameter (m)	Design Discharge (m3/s)
200	2.077	19.5	392
201	3.077	12	376
202	4.077	10	347
203	5.077	9	325
204	6.077	8.3	302

A4.4.4 Auxiliary spillway sizing

During an MDF, the spillway system is required to pass almost twice that of the OBF. Although the MDF flood rise of 0.5 m will allow some additional flow over the primary spillway, an auxiliary spillway is required to pass the remainder.

With the difference between the OBF and MDF flood rise for the Lee Valley Dam of 0.5 m, the auxiliary spillway is required to pass a considerable flow with very little available head. A fixed overflow spillway such as an ogee weir or broad crested weir with a crest height above the OBF flood rise would be impractically wide. For this reason, a fuse plug spillway was evaluated as an economical means of providing flow capacity once the OBF has been exceeded.

The auxiliary channel was evaluated with an invert of 194 m RL and with an effective crest at the maximum OBF flood rise. This gave a fuse plug height range of 3.077 m to 9.077 m for the options considered.

During an MDF, the primary spillway will operate on its own until the OBF flood rise is exceeded. Once the OBF flood rise is exceeded the auxiliary spillway will breach and thereafter both spillways will operate together to the additional 0.5 m flood rise. Once water levels drop below the primary spillway crest level, the auxiliary spillway will continue to operate, drawing the water down to the level of the approach channel until the breachable embankment is replaced.

The fuse plug was assumed to 'fuse' over 4 minutes for the purposes of flood routing, and once gone the channel discharge was calculated as a broad crested weir with a discharge coefficient of 1.5. A single fuse plug was used in the outline design. To reduce any surge in flows, fuse plug spillways are often designed with multiple segments. This should be further investigated during more detailed design phases.

Similar to the primary weirs, the width of the fuse weir was calculated for the allowable rise during the MDF given an embankment height or crest level. This was done by routing the PMP hydrograph (section 7) though the reservoir to calculate the maximum rise for a given weir width and primary weir combination.

Weir widths and discharges for ogee, labyrinth and bell-mouth spillways can be found in Table App 4, Table App 5, Table App 6 and in Figure App 6.

Table App 4: Fuse-Plug Sizing for MDF with Ogee Primary Weir

Dam Crest Level (m RL)	MDF Flood Rise (m)	Ogee Weir Width (m)	Ogee Weir Peak Discharge (m^3/s)	Auxiliary Crest Length (m)	Auxiliary Weir Peak Discharge (m^3/s)	Combined Peak MDF Discharge (m^3/s)
200	2.577	59.5	567	26	514	1081
202	4.577	20.5	449	19.5	606	1036
204	6.577	10.54	395	14.5	641	1056

Table App 5: Fuse-Plug Sizing for MDF with Labyrinth Primary Weir

Dam Crest Level (m RL)	MDF Flood Rise (m)	Labyrinth Weir Width (m)	Labyrinth Weir Peak Discharge (m^3/s)	Auxiliary Crest Length (m)	Auxiliary Weir Peak Discharge (m^3/s)	Combined Peak MDF Discharge (m^3/s)
200	2.577	34	492	29.5	580	1073
202	4.577	16.2	401	20.5	641	1034
204	6.577	10.6	355	15.5	680	1043

Table App 6: Fuse-Plug Sizing for MDF with Bell-mouth Primary Weir

Dam Crest Level (m RL)	MDF Flood Rise (m)	Bell-mouth Weir Dia. (m)	Bell-mouth Weir Peak Discharge (m^3/s)	Auxiliary Crest Length (m)	Auxiliary Weir Peak Discharge (m^3/s)	Combined Peak MDF Discharge (m^3/s)
200	2.577	19.5	528	28	550	1078
202	4.577	10	368	22	672	1038
204	6.577	8.3	309	17	729	1040

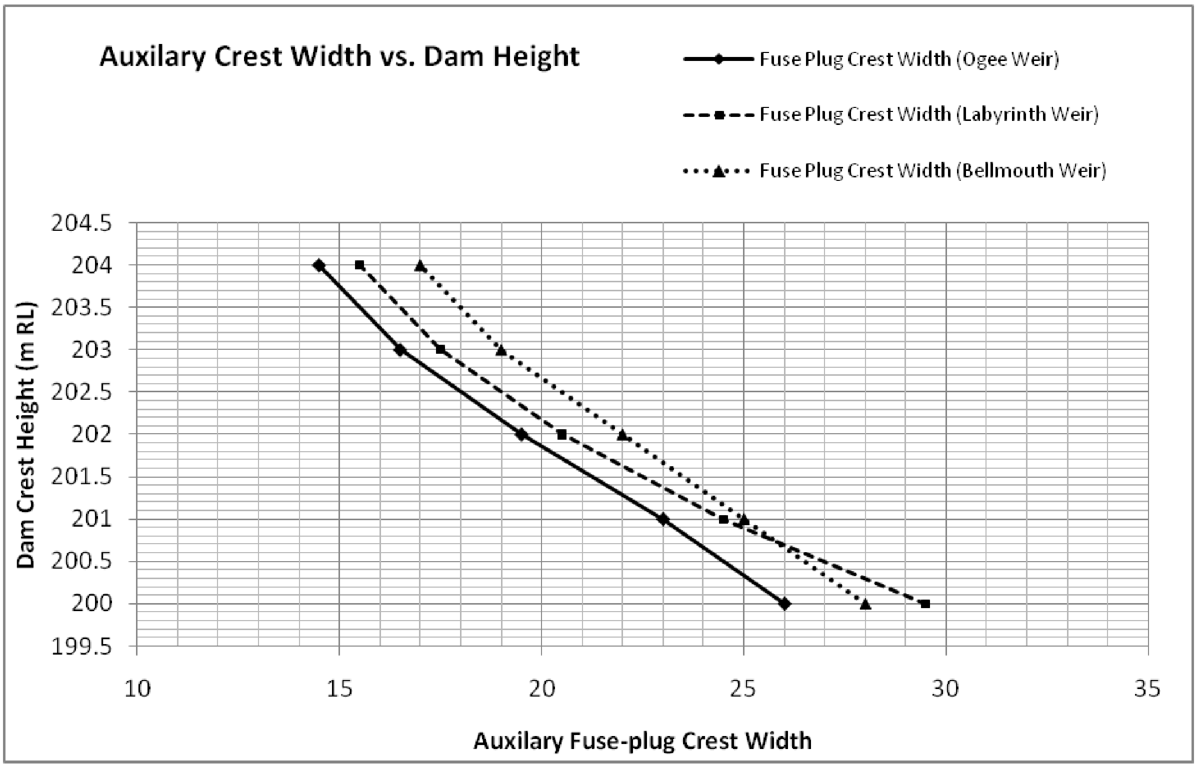


Figure App 6: Evaluation of fuse plug crest widths

A4.5 Preliminary arrangement for ogee weir primary spillway

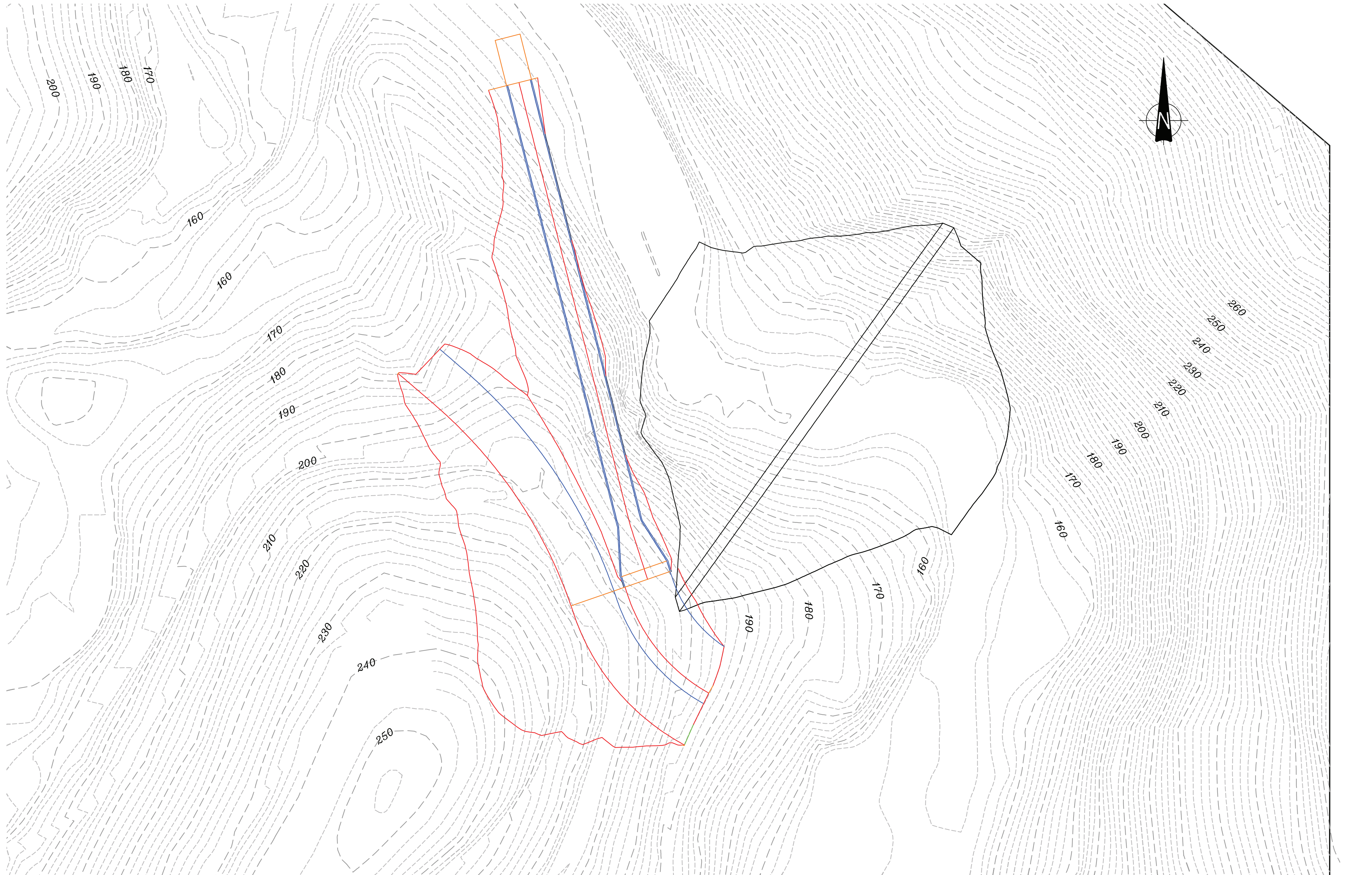
A4.5.1 Introduction

Two weir and chute arrangements were considered. Both were located on the left abutment of the dam as seen in Figure App 7 and Figure App 8.

Option A has the weir approximately in line with the embankment centreline. A 195 m long chute would be cut into the eastern side of the ridge and convey flow to the river approximately 80 m downstream of the dam exiting at a slight angle to the river. The chute would be terminated by a hydraulic jump basin. An auxiliary spillway would receive flow via the same approach channel before conveying it west across the ridge to Anslow Creek.

Option B includes a relatively flat spillway approach channel taking low velocity flow around 80 m past the embankment centreline. The approach channel would end in a control structure before a side discharge into a steep 100 m chute built at a 40 degree angle to the river. This would enable a shorter chute rejoining the river only 30 m from the toe of the dam. The chute would likely be terminated in a flip bucket energy dissipater. Energy dissipation was not considered in this options study as the dissipation required is comparable across the options.

An auxiliary spillway would receive flow via the same approach channel before conveying it west across the ridge to Anslow Creek.



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SCALES (AT A3 SIZE) AS SHOWN		
PROJECT No. PROJECT		

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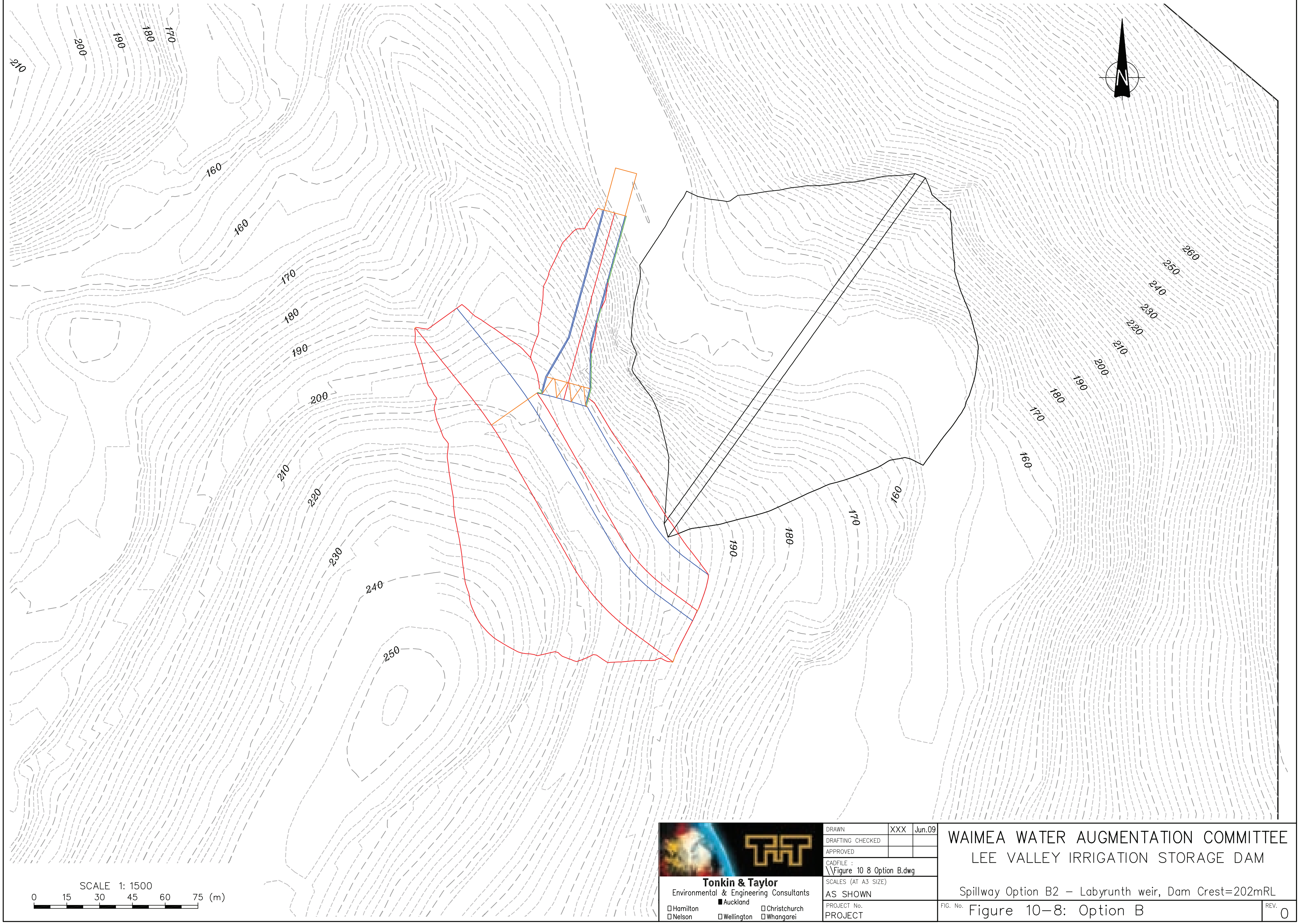
WAIMEA WATER AUGMENTATION COMMITTEE

LEE VALLEY IRRIGATION STORAGE DAM

Spillway Option A2 – Ogee weir, Dam Crest=202mRL

FIG. No. Figure 10–7: Option A

REV. 0



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PROJECT No. PROJECT		

WAIMEA WATER AUGMENTATION COMMITTEE
LEE VALLEY IRRIGATION STORAGE DAM

Spillway Option B2 – Labyrunth weir, Dam Crest=202mRL

FIG. No. Figure 10–8: Option B

REV. 0

A4.5.2 Option A arrangement

The approach channel is designed to give sufficient depth to the control structure and to minimise approach velocities and turbulence. The channel is approximately 60 m in length and has an invert of 194 m RL. Some additional width was added to the approach channel to allow for structures such as weir inlet wing walls. Velocities in the approach channel range between 2 – 4 m/s for the various options being considered. Being cut into rock, the approach channel would not require lining for erosion control but a lining may be required to prevent seepage. Some refinement in the arrangement of the approach channel will be undertaken as part of the refined arrangement in Section 11 to further reduce approach velocities and turbulence.

The weir spills directly into a 195 m long concrete chute with a longitudinal grade of 20%. Submergence effects were taken into account by calculating the available energy head at the top of the chute and providing adequate vertical drop between the crest and the chute. The chute typically begins 7 m below the crest of the weir giving a chute inlet invert level of approximately 190 m depending on the weir width.

Chute hydraulic conditions were checked for the peak flow expected during the MDF. Calculations show that a gradually varying flow profile is expected, indicating that the flow regime remains supercritical at all stages in the chute, particularly during any changes in width. Calculations were undertaken using a manning's roughness of 0.013. For ogee weir and chute combinations, this water surface profile was calculated from the crest down to ensure no submergence effects.

The estimated water surface profile was adjusted to allow for flow bulking due to air entrainment on the chute in accordance with the method outlined by the USACE (Hydraulic Design of Flood Control Channels EM 1110-2-1601 1994). This method provides a relationship between the Froude number of the flow and the flow bulking ratio. The spillway chute side wall height was designed to give a minimum of 1 m freeboard above this aerated flow profile to make allowance for spillway waves which might occur downstream of a bridge over the spillway, if this is required, and from the chute contraction.

Flow conditions at the downstream end of the chute were calculated and the chute width adjusted to minimise the width of the chute while maintaining a reasonable wall height of between 3-5m. Velocities at the toe of the 20% chute ranged from 25 – 29 m/s with Froude numbers between 7 and 9. The peak velocities and unit discharge (approximately 40 cumecs/m) are within the range of precedent spillways where cavitation has not been a significant problem.

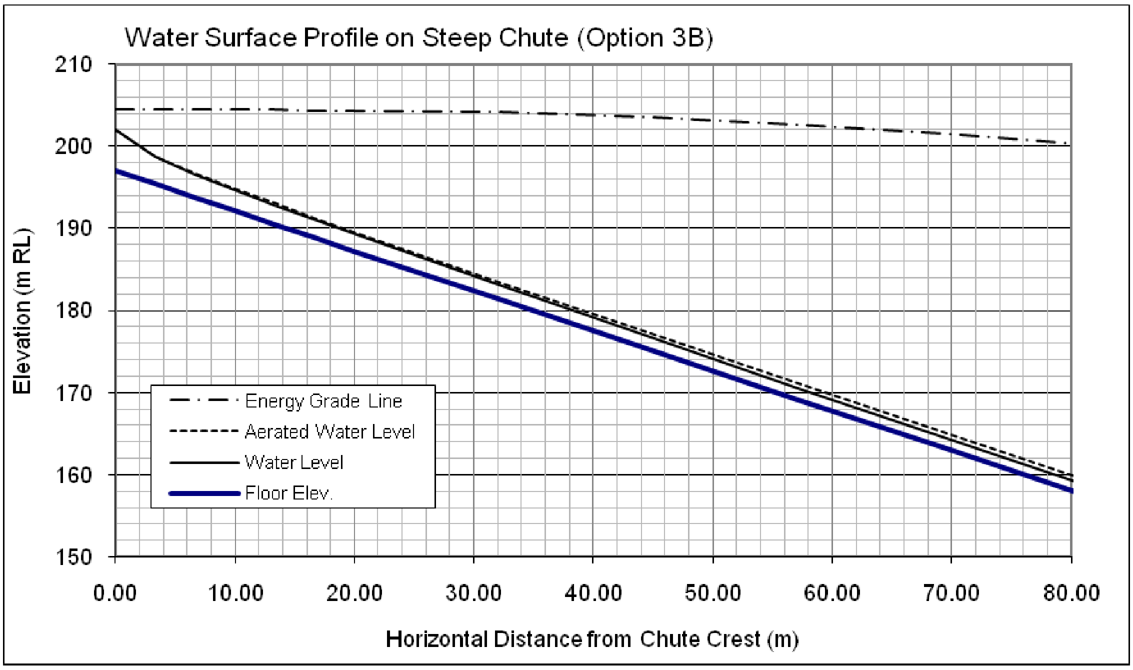


Figure App 9: Gradually varied flow profile and bulking on chute

The terminal structure was not investigated in depth as costs are expected to be similar across the various options. For the 20% chute, a horizontal hydraulic jump basin is expected to provide adequate dissipation of the energy at the toe of the chutes before discharge to the stream. For steeper chutes, a flip bucket and plunge pool are likely to be more economic.

A4.5.3 Option B arrangement

To enable a steeper chute to be built the approach channel for configuration B is much longer with an average length of around 130 m. As the channel extends past the embankment centreline, it has been assumed concrete lined to provide an impermeable barrier to minimise seepage and slope stability issues below. The velocities and depths of the channel remain the same as for the previous Option A with a channel invert of 194 m RL.

The chute has been designed as a rectangular concrete channel sloping up to 49% into the river below the dam. Design was per the methodology for Option A. The chute takes flow from the approach channel at an angle, and if this option is attractive the hydraulic implications of this, combined with the slight curvature of the channel will need careful consideration.

Flow profiles and bulking were calculated for the steep chutes and the resulting velocities were calculated between 28 – 30 m/s at the toe of the chute with Froude numbers between 6.5 – 9.

Due to the high velocities and the sharp approach angle at which the flows enter the river, a flip bucket spillway is likely to be the most efficient option for dissipating the energy resulting from a steep chute.

A4.6 Bell-mouth spillway option arrangement

An intake tower would be built in the reservoir at the upstream end of the construction diversion. The intake would be topped by a bell-mouth inlet at NTWL and convey flow through the embankment diversion tunnel or culvert before discharge at the toe of the dam via an energy dissipation basin. The tunnel or culvert outlet would also serve as a conveyance structure for the irrigation, low-level outflows and hydropower penstocks (if considered feasible) with a power house situated at the downstream toe of the dam. The bell-mouth tower may also double as an intake tower and spillway. As with the previous options, an auxiliary spillway channel would also be incorporated to convey flow west to Anslow Creek. A tunnel outlet arrangement option was not investigated as initial costing showed tunnelling to be prohibitively expensive.

The bell-mouth spillway discharge channel consists of a vertical throat, elbow and culverted outlet. The location of the inlet structure would be determined by the development of the diversion works.

The throat radius was determined using the transition curve equations provided by Design of Small Dams (USBR 1987). Pressure flow is to be avoided in these structures as flows can become unstable and surge. The box culverts assumed for construction diversion are adopted as the outlet for the bell-mouth.

The head losses in the shaft and culverts used as permanent bell-mouth spillway conduits, were calculated assuming they flow 75% full. The additional 25% area is provided to allow for flow bulking and ensure partially full flow. Aeration at the inlet should also be provided to ensure pressure flow does not develop.

The velocities at the culvert exits during the OBF were approximately 20 m/s.

A terminal structure would be sized to accommodate this and ensure a transition of flow given the tail-water levels calculated. This tail-water level was calculated using the downstream geometry and the 1D flow modelling software HEC-RAS.

As for previous spillway configuration options, an auxiliary spillway was included to pass flows exceeding the OBF. The most likely location for this spillway is over the left hand abutment, discharging to the Anslow Creek, similar to the Ogee weir option.

Seismic loading on a shaft for the bell-mouth presents significant structural issues, and the need for large quantities of structural concrete and reinforcing steel. The height of the shaft also presents significant construction difficulties, adding to cost. These factors have been allowed for in element costing to a preliminary level, appropriate for comparing the various spillway options.

A4.7 Preliminary arrangements for labyrinth primary spillway

Weir and chute arrangements for this option are the same as considered for the ogee weir configurations (refer Figure App 7 & Figure App 8)) but with a labyrinth primary weir.

The height of the vertical downstream face and entry to the chute was designed to ensure no submergence of the weir crest.

A4.8 Preliminary option cost comparisons

Comparative costs (excluding items likely common to all options) were evaluated for the range of CFRD embankment height and spillway type combinations. The major costs for spillway construction are earthworks and concrete. Concrete volume depends on freeboard allowance. Low freeboard will require a wide weir and channel, whereas high freeboard will require a narrower weir. The earthworks volumes in both the embankment and spillway need to be evaluated so as to evaluate needs for cut to waste or borrow to fill. Spillway and dam cost combinations can be compared to arrive at the apparent optimum crest elevation and spillway option.

Spillway and embankment options were each modelled in a 3D CAD modelling package (12D) which was used to obtain cut and fill quantities. Earthworks quantities for the embankment were calculated for a range of dam heights. Although various zones such as the concrete lining and filters will need to be imported, rock-fill zones 3A and 3B which make up the majority of the embankment volume may potentially be sourced in part from the spillway cut (see Figure App 10).

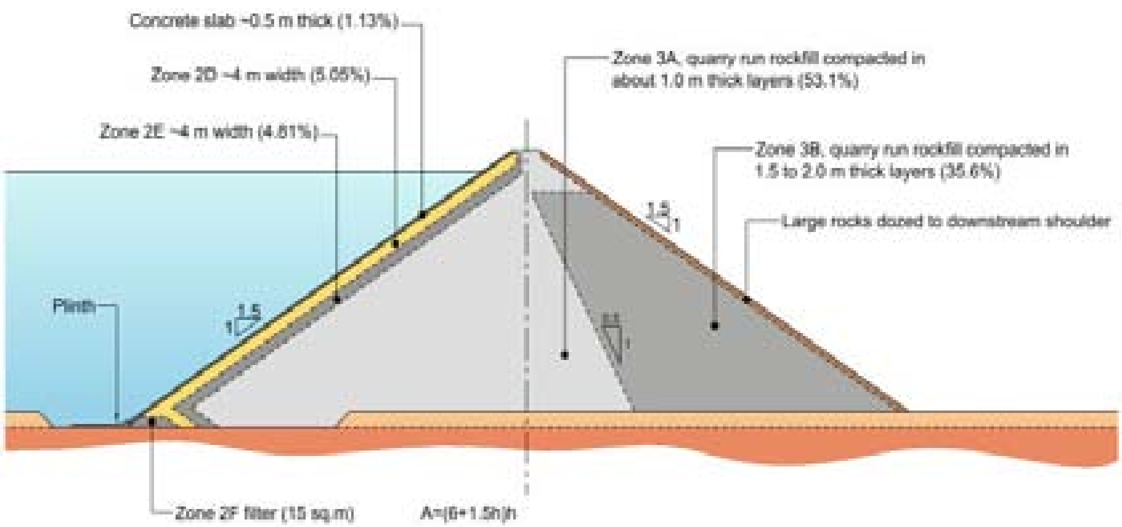


Figure App 10: CFRD Embankment Zoning

Zone 3A is the structural core of the dam and requires high quality rock fill to minimise settlement on reservoir filling. Zone 3B supports Zone 3A but has reduced strength and stiffness requirements. Although most of this volume may come from the spillway cut, some uncertainty remains regarding the quality of rock that will be sourced from a spillway cut on the left abutment.

Two scenarios were evaluated for the earthworks balance across the site.

1. High rock fill quality in spillway cut. In this case, all rock won from the spillway cut may be used in the embankment plus additional borrow may be undertaken (within the spillway area) at the same cost to provide any shortfall.

2. Poor rock fill quality in spillway cut. In this case, spillway cut can be used in Zone 3B only. Any shortfall to make up the 3B volume may be taken as additional borrow from the spillway area. However, Zone 3A must be entirely borrowed from high quality alluvial gravels identified upstream of the embankment location and any spillway cut over and above the volume required for Zone 3B will need to be wasted.

Given these assumptions an earthworks balance incorporating the spillway cut and embankment fill requirements for various crest levels was developed for the site as seen in Table App 7.

The total cost evaluation for the spillway was calculated to include the structure costs plus any waste generated from the earthworks but exclude excavation costs where material was used as embankment fill. The total cost evaluation for the embankment will include concrete lining filters and rock fill, the cost of importing material from the spillway cut and any material required to be borrowed at increased cost from elsewhere.

Table App 8 shows the total cost index for each of the options split between good and poor quality spillway rock. Figure App 11 and Figure App 12 show the total cost index vs. crest height summary for both good and poor quality rock respectively. Costs were indexed against a base index of 100 for an ogee weir option configuration A with a dam height of 202 m RL.

Table App 7: Earthworks Balance

Option - DS Dam			WL	Total Cut	Fill Zone 3B	Fill Zone 3A	Good Rock: Cut to Fill Zone 3B	Good Rock: Cut to Fill Zone 3A	Good Rock: Cut to Waste	Good Rock: Borrow to Fill Zone 3	Poor Rock: Cut to Fill Zone 3B	Poor Rock: Cut to Waste	Poor Rock: Borrow to Fill Zone 3
			mRL	m³	m³	m³	m³	m³	m³	m³	m³	m³	m³
A	1	Ogee	200	578,490	127,038	188,936	127,038	188,936	262,516	0	127,038	451,452	188,936
A	2	Ogee	202	334,297	140,491	210,177	140,491	210,177	0	0	140,491	193,806	210,177
A	3	Ogee	204	220,748	154,807	232,564	154,807	232,564	0	0	154,807	65,941	232,564
A	1	Labyrinth	200	304,474	127,038	188,936	127,038	188,936	0	0	127,038	177,436	188,936
A	2	Labyrinth	202	202,375	140,491	210,177	140,491	210,177	0	0	140,491	61,884	210,177
A	3	Labyrinth	204	221,391	154,807	232,564	154,807	232,564	0	0	154,807	66,584	232,564
B	1	Ogee	200	576,708	127,038	188,936	127,038	188,936	260,734	0	127,038	449,670	188,936
B	2	Ogee	202	178,064	140,491	210,177	140,491	210,177	0	0	140,491	37,573	210,177
B	3	Ogee	204	156,873	154,807	232,564	154,807	232,564	0	0	154,807	2,066	232,564
B	1	Labyrinth	200	369,908	127,038	188,936	127,038	188,936	53,934	0	127,038	242,870	188,936
B	2	Labyrinth	202	167,304	140,491	210,177	140,491	210,177	0	0	140,491	26,813	210,177
B	3	Labyrinth	204	153,127	154,807	232,564	154,807	232,564	0	0	154,807	0	232,564
D	1	Bellmouth	200	112,110	127,038	188,936	127,038	188,936	0	0	127,038	0	188,936
D	2	Bellmouth	202	107,353	140,491	210,177	140,491	210,177	0	0	140,491	0	210,177
D	3	Bellmouth	204	108,004	154,807	232,564	154,807	232,564	0	0	154,807	0	232,564

Table App 8: Comparative cost summary

Option Configuration and Weir Type	Dam Crest level (m RL)	Cost Index Good Quality Rock	Cost Index Poor Quality Rock
Option A Ogee	200	**120**	**137**
Option A Ogee	202	**100**	**118**
Option A Ogee	204	**107**	**119**
Option A Labyrinth	200	**114**	**130**
Option A Labyrinth	202	**107**	**117**
Option A Labyrinth	204	**108**	**119**
Option B Ogee	200	**144**	**162**
Option B Ogee	202	**113**	**122**
Option B Ogee	204	**116**	**123**
Option B Labyrinth	200	**128**	**145**
Option B Labyrinth	202	**113**	**121**
Option B Labyrinth	204	**116**	**123**
Option D Bell-mouth	200	**108**	**113**
Option D Bell-mouth	202	**115**	**122**
Option D Bell-mouth	204	**130**	**137**

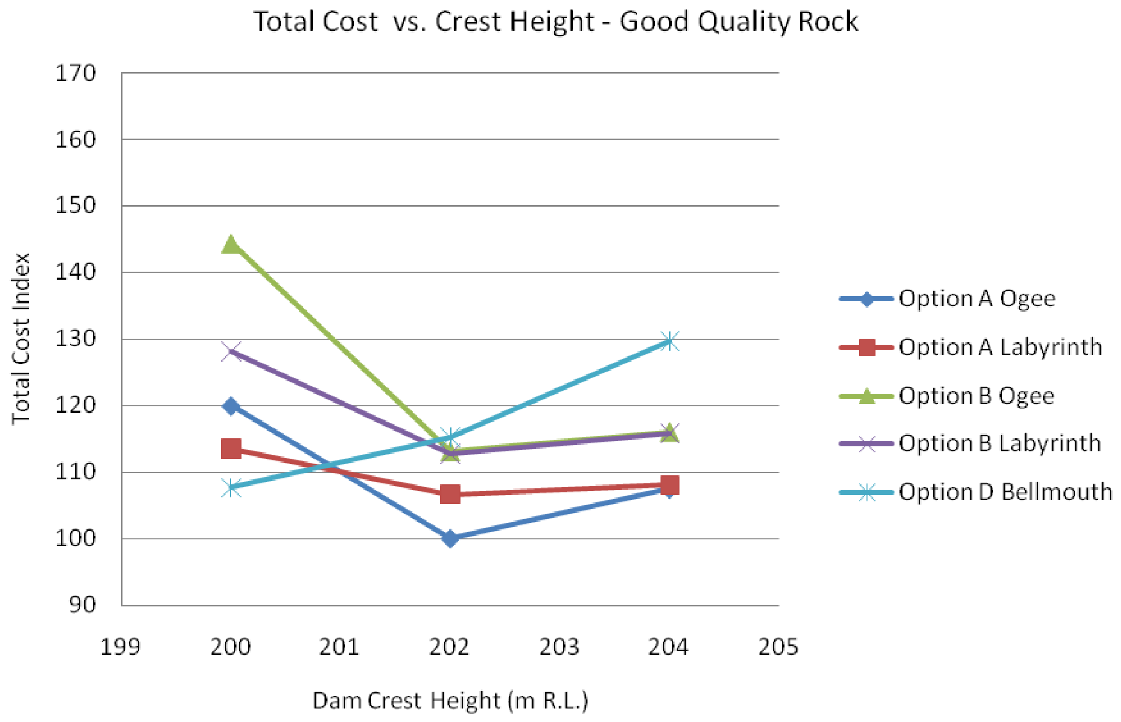


Figure App 11: Total cost vs. Crest height – good rock quality

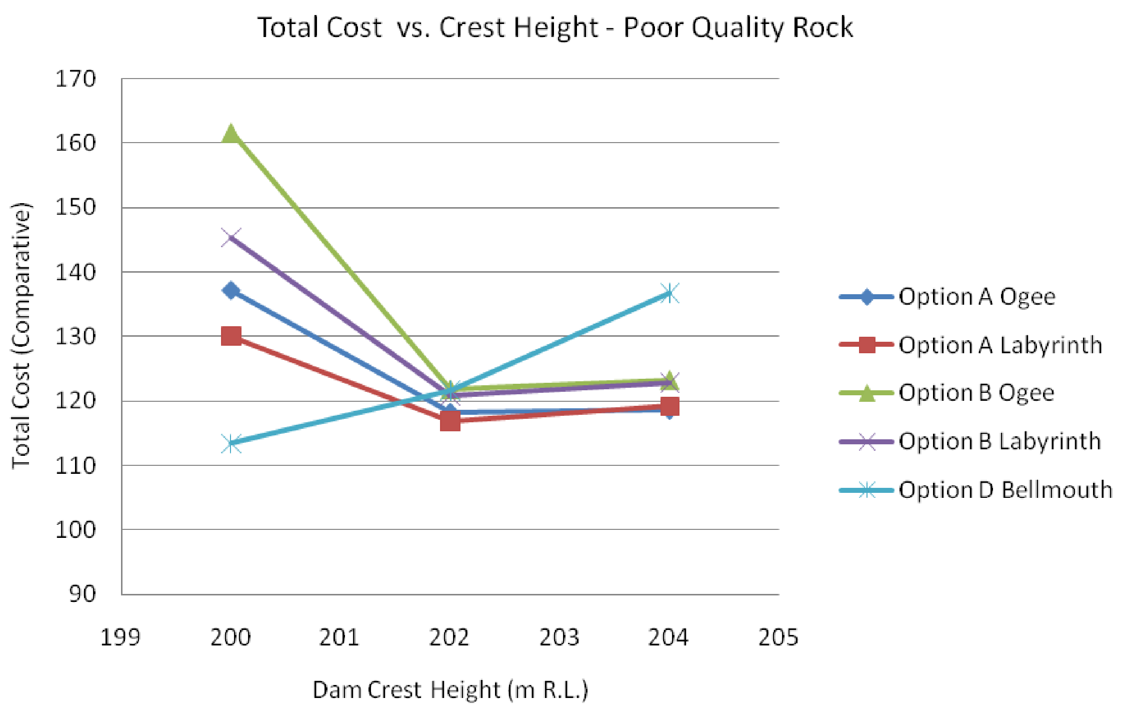


Figure App 12: Total cost vs. Crest height – poor rock quality

A4.9 Spillway and embankment selection

Total cost index curves show an economic embankment height at 202 m RL for all options with the exception of the bell-mouth spillway. The bell-mouth spillway option indicates that a lower embankment (and larger diameter spillway crest) is more cost effective. The curves indicate that a bell-mouth spillway is overall the most cost effective option, but this is only marginally so for the case of good quality rock. The cost effectiveness of the bell-mouth stems from use of construction diversion culverts as the outlet system.

If good quality rock is assumed it appears that after the bell-mouth spillway, the ogee weir and chute for configuration A at 202 m RL is the next most economical option. For the poor rock case the next best option is a labyrinth weir at 202 m RL.

This outcome is understandable as the ogee weir spillways generate the most cut and therefore an economical embankment when the rock is good, while the labyrinth weir is narrower and produces less cut to waste if the rock quality is poor.

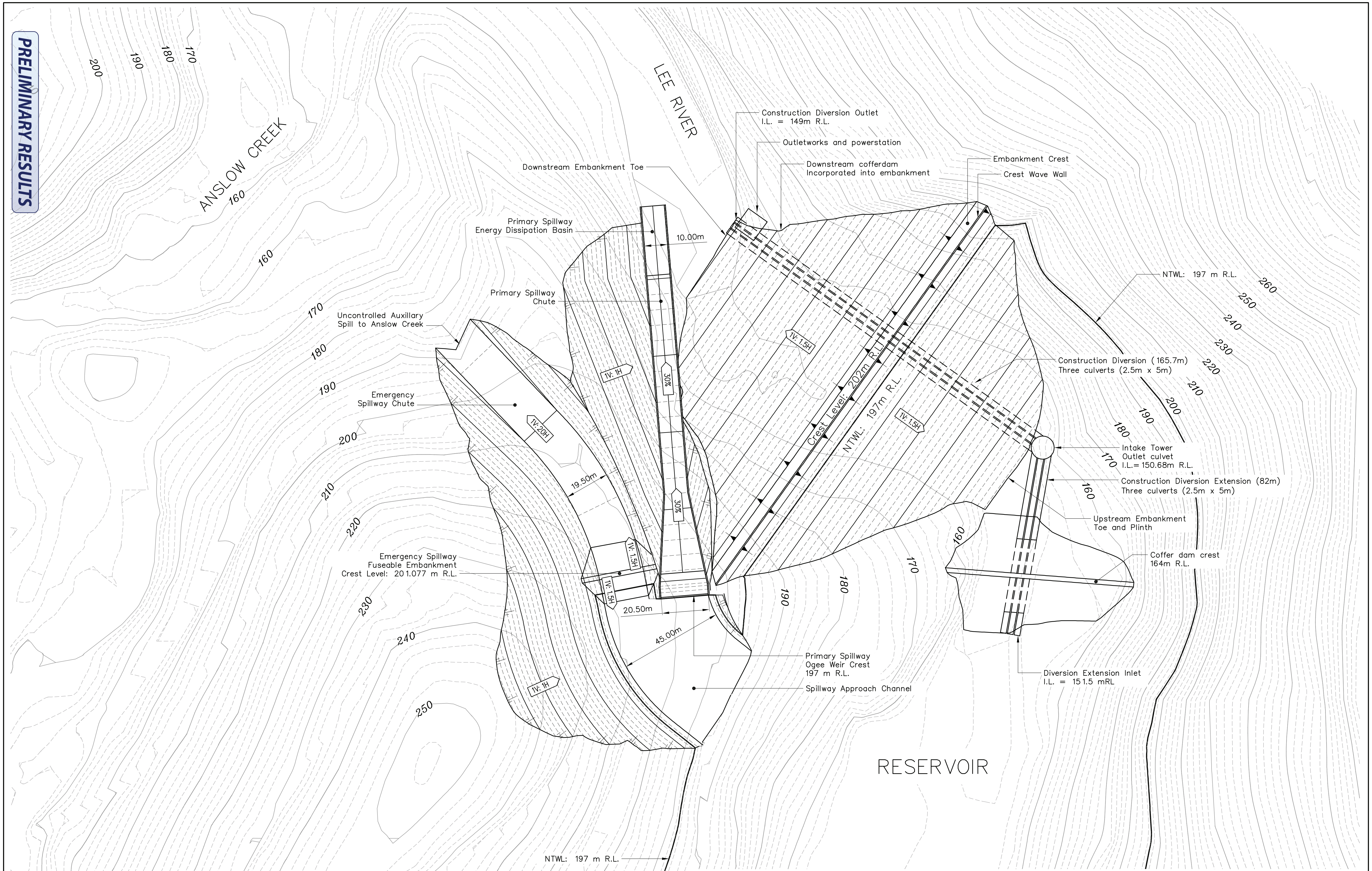
A labyrinth weir option in poor rock will cost more than an Ogee weir built in good rock. The difference in cost between both ogee and labyrinth in poor rock is small.

A bell-mouth spillway introduces significant disadvantages for passage of native fish, and likely significant operational difficulties associated with accessing the irrigation outlet works, as these may need to be shared with spillway passage. Due to its height, the towers resistance to seismic forces will have a large impact on the cost. The likely hydraulic interaction of the spillway with any outlet tower works would also disadvantage this option.

Based on the above work and discussions, the following parameters have been adopted for preliminary design and costing:

- embankment crest at RL 202 m.
- ogee weir (adjacent to embankment centre line) with chute primary spillway
- auxiliary spillway with fusible embankment 19.5 m wide adjacent to the ogee weir and discharging to Anslow Creek
- construction diversion consisting of 3, 2.5 m x 5 m square box culverts with separate upstream coffer dam with crest at RL 163 m
- outlet tower with outlet via steel pipe housed in diversion culvert.

An outline arrangement for the preferred option is provided in Figure App 13. Note this figure is simplified in that it does not include requirements for vehicle access or many of the details of the dam, but is included to show the development of the design. More detail is developed in section 11.



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WAIMEA WATER AUGMENTATION COMMITTEE
LEE VALLEY IRRIGATION STORAGE DAM

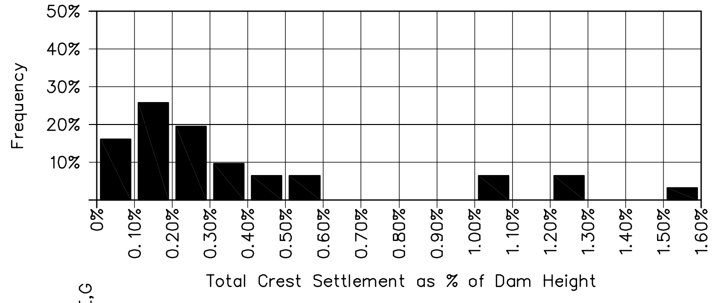
Spillway Option A2 – Ogee weir, Dam Crest=202mRL

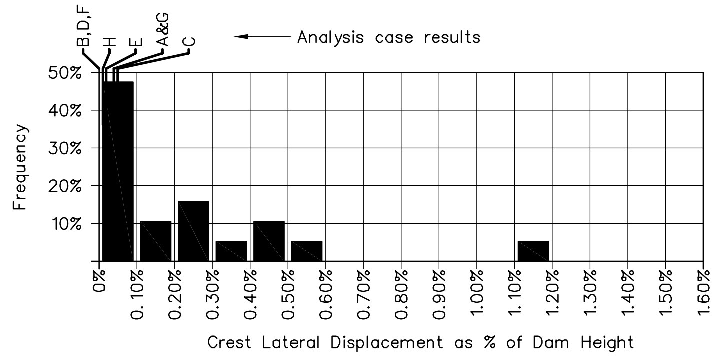
FIG. No. Figure 10–13: Outline Arrangement

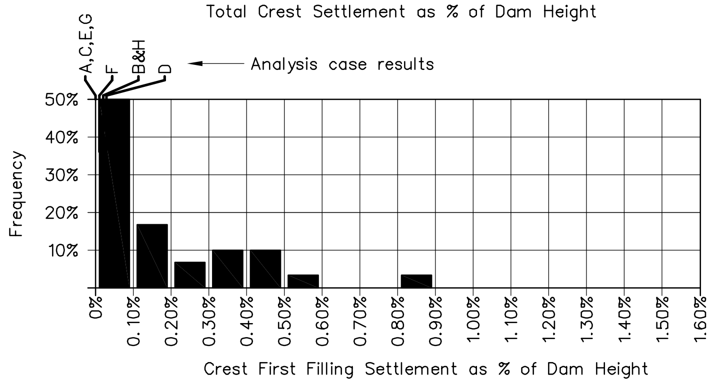
REV. 0

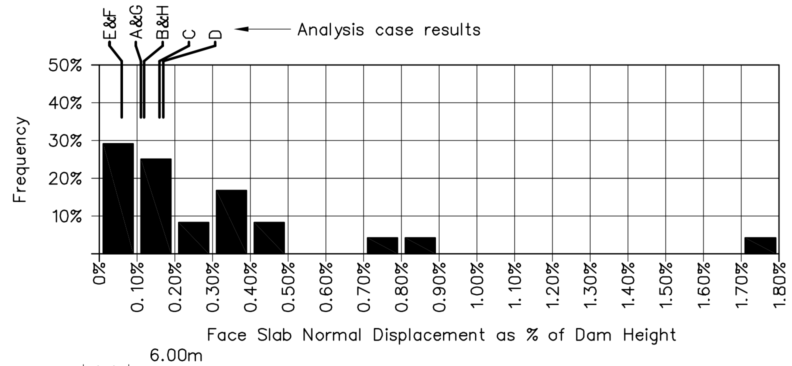
Appendix B: Feasibility Design Drawings

Sheet	Title
Design Assessment	
• A-01	Embankment Settlement Assessment
• A-02	Plinth Design Assessment
Dam Arrangement	
• B-01	Dam General Arrangement
• B-02	Dam Foundation Cut and Stripping
• B-03	Foundation Isopach
• B-04	Detailed Plan Spillway
• B-05	Detailed Plan Dam
Embankment	
• C-01	Embankment Cross Sections
• C-02	Embankment Details
• C-03	Elevation of Upstream Face
Spillway	
• D-01	Spillway Long Sections
• D-02	Spillway Cross Sections
• D-03	Spillway Details
Reservoir Outlet	
• E-01	Reservoir Outlet Arrangement
• E-02	Reservoir Outlet Sections
Construction Methodology	
• F-01	Construction Flood Management
Hydro Option	
• G-01	Hydro General Arrangement









Analysis Key			
Case Name	Zone 3A Material	Zone 3B Material	Poisson's Ratio
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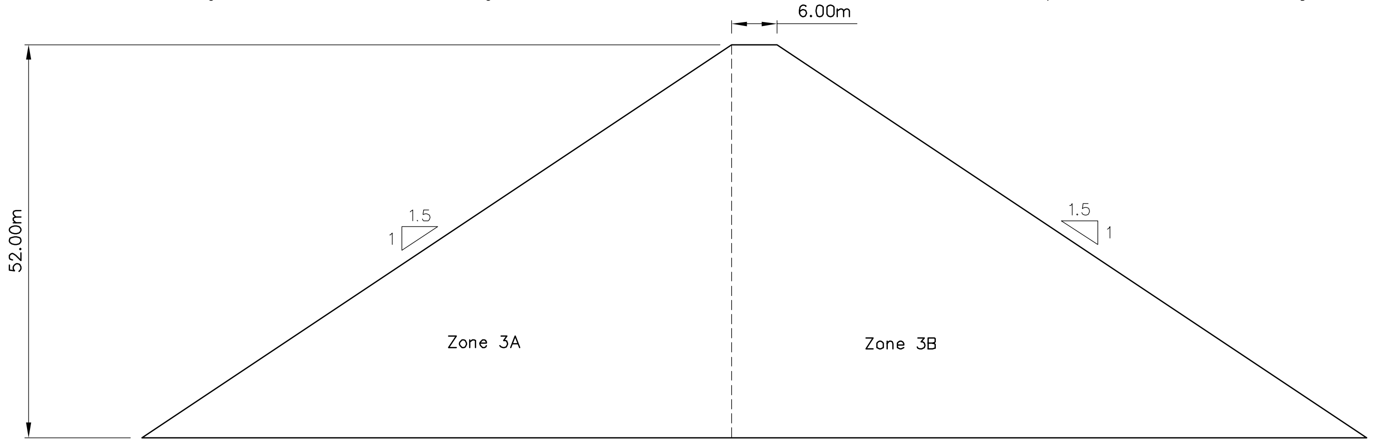
B	Class 1 rockfill	Class 1 rockfill	0.05
C	Class 3 Rockfill	Class 3 rockfill	0.30
D	Class 3 rockfill	Class 3 rockfill	0.05
E	Alluvium	Alluvium	0.30
F	Alluvium	Alluvium	0.05
G	Class 1 rockfill	Class 3 rockfill	0.30
H	Class 1 rockfill	Class 3 rockfill	0.05



SIMPLIFIED EMBANKMENT MODEL

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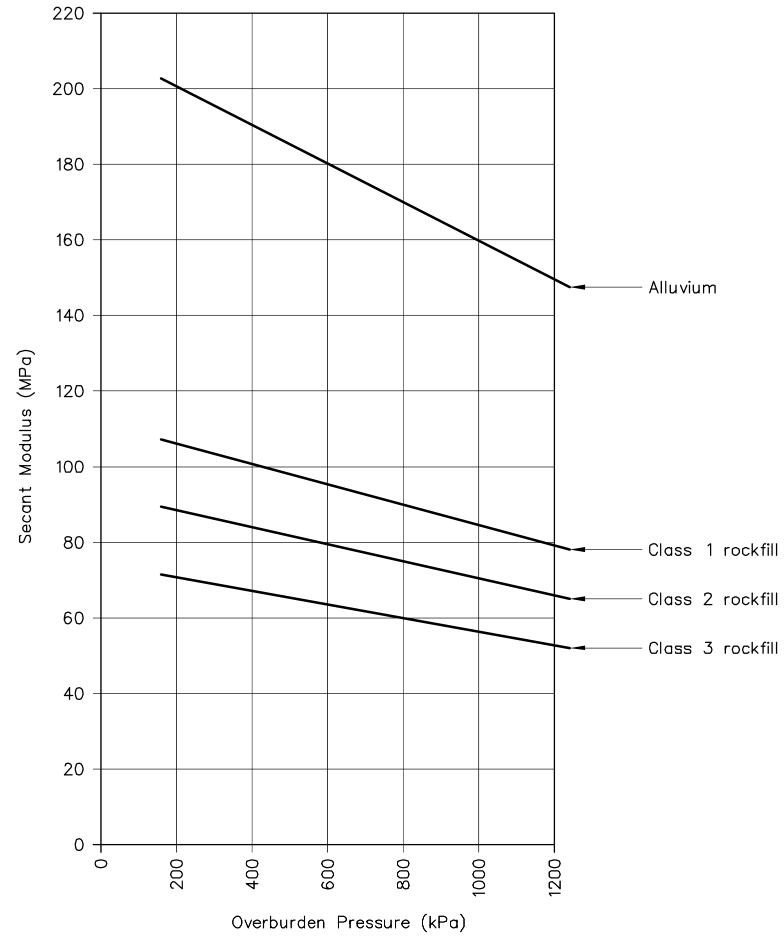

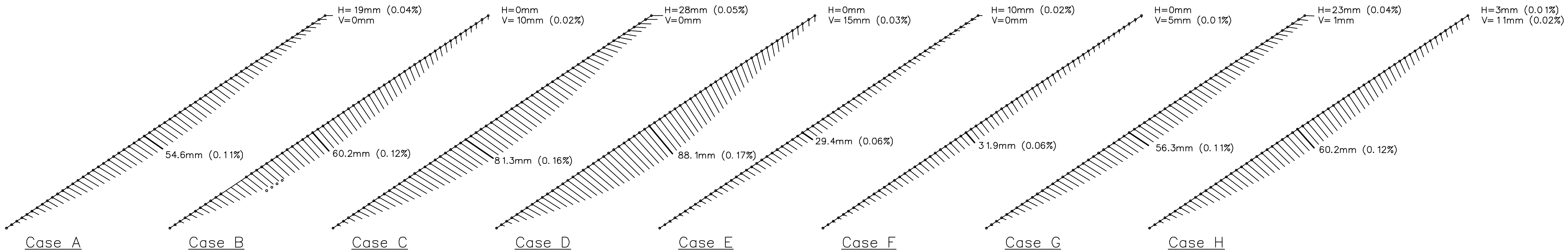

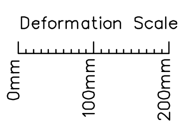

DEFORMATION PLOTS FOR UPSTREAM FACE OF DAM

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Waimea Water Augmentation Committee
Lee Valley Dam - Feasibility Design
A-Design Assessment
Embankment Settlement Assessment

FIG. No. A-01
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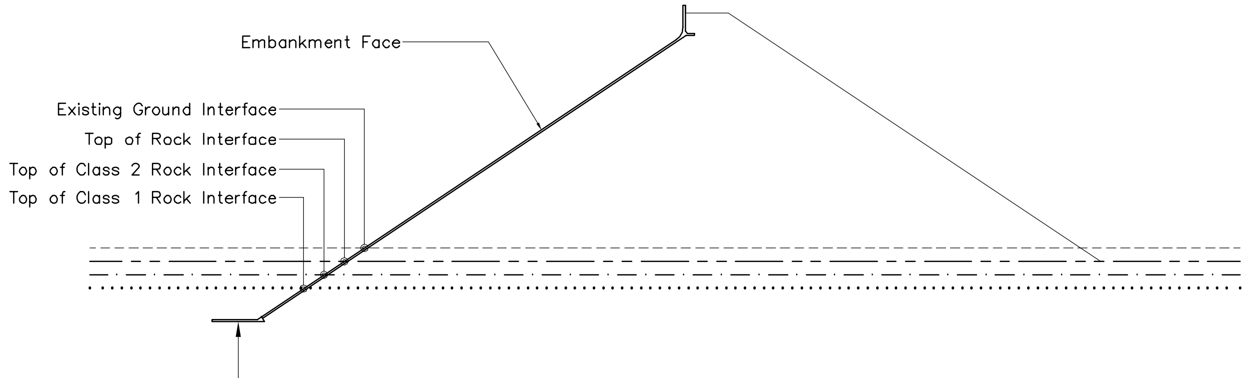

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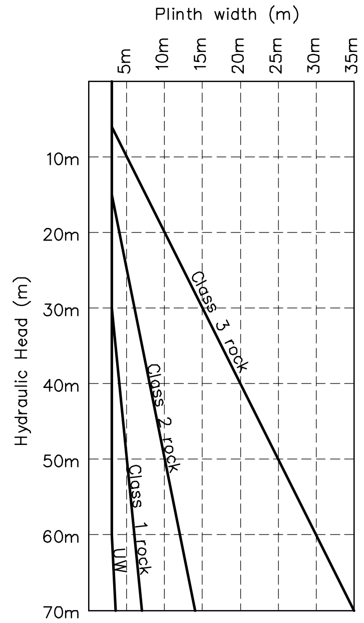

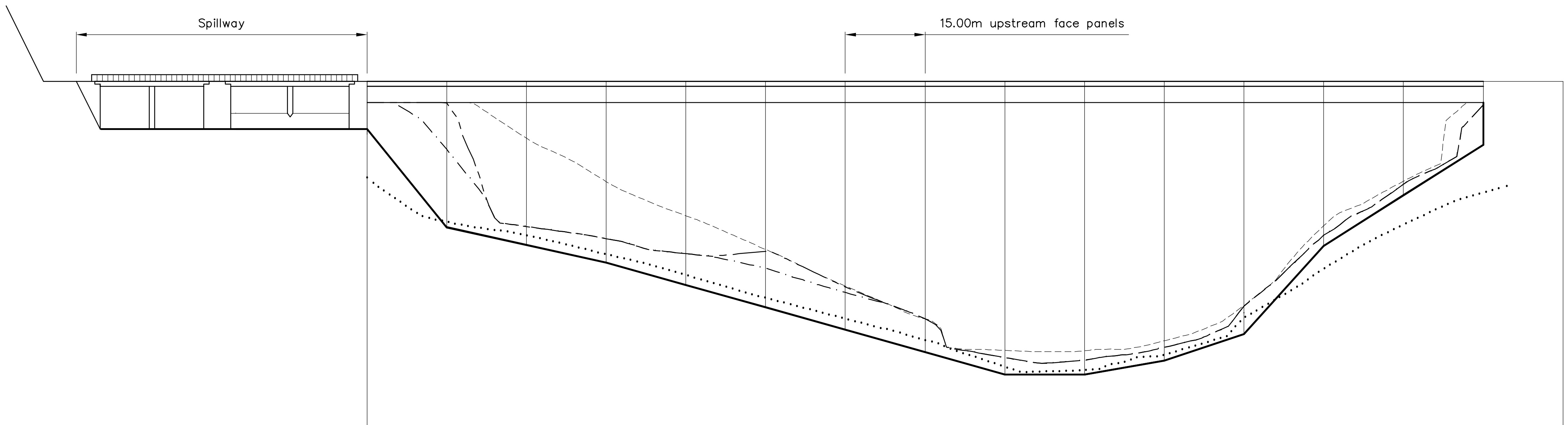

	Spillway															
Design Plinth Width	19.2	5.50	5.00	5.00	5.00	5.00	5.00	5.50	6.00	6.00	6.00	5.50	10.00	6.50	5.00	5.00
Min Plinth Elevation (m)	192.00	174.50	171.20	167.87	163.66	159.44	155.23	151.02	146.80	146.80	146.80	149.44	154.39	171.01	180.52	190.02
Max Hydraulic Head (m)	10.00	27.50	30.80	34.13	38.34	42.56	46.33	50.55	55.20	55.20	55.20	52.56	47.61	30.99	21.48	11.98
Max Hydraulic Gradient (m/m)	2.00	5.00	6.16	6.83	7.67	8.51	9.27	9.19	9.20	9.20	9.20	9.56	4.76	4.77	4.30	2.40
Rock Type	Class 2	Class 2	Class 1	Class 1	Class 1	Class 1	Class 1	Class 1	Class 1	Class 1	Class 1	Class 2	Class 2	Class 2	Class 2	Class 2

ELEVATION 3/B05
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SCALE 1:1000
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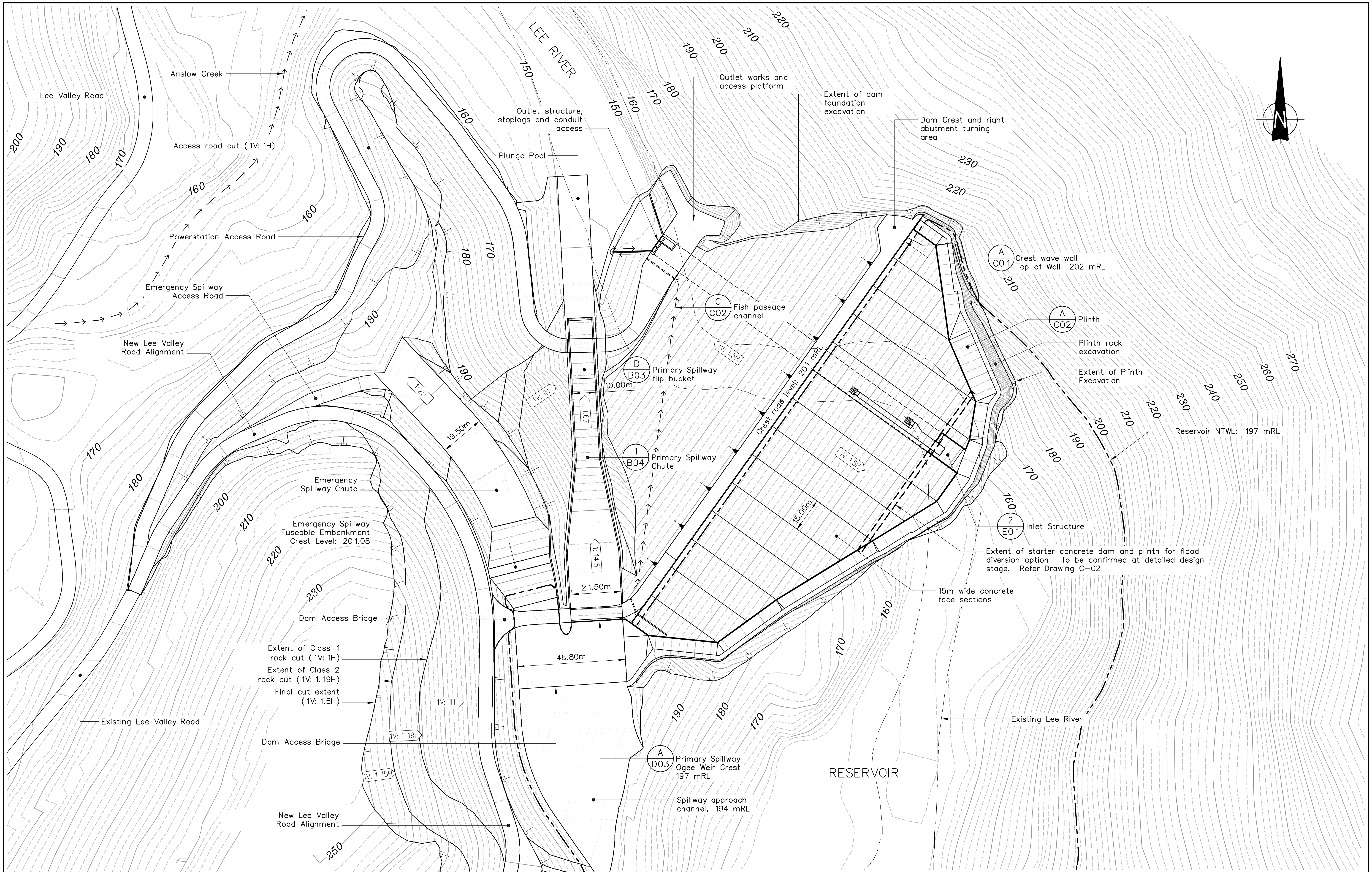
Lee Valley Dam – Feasibility Design
A–Design Assessment
Plinth Design Assessment

FIG. No. A–02
REV. 0



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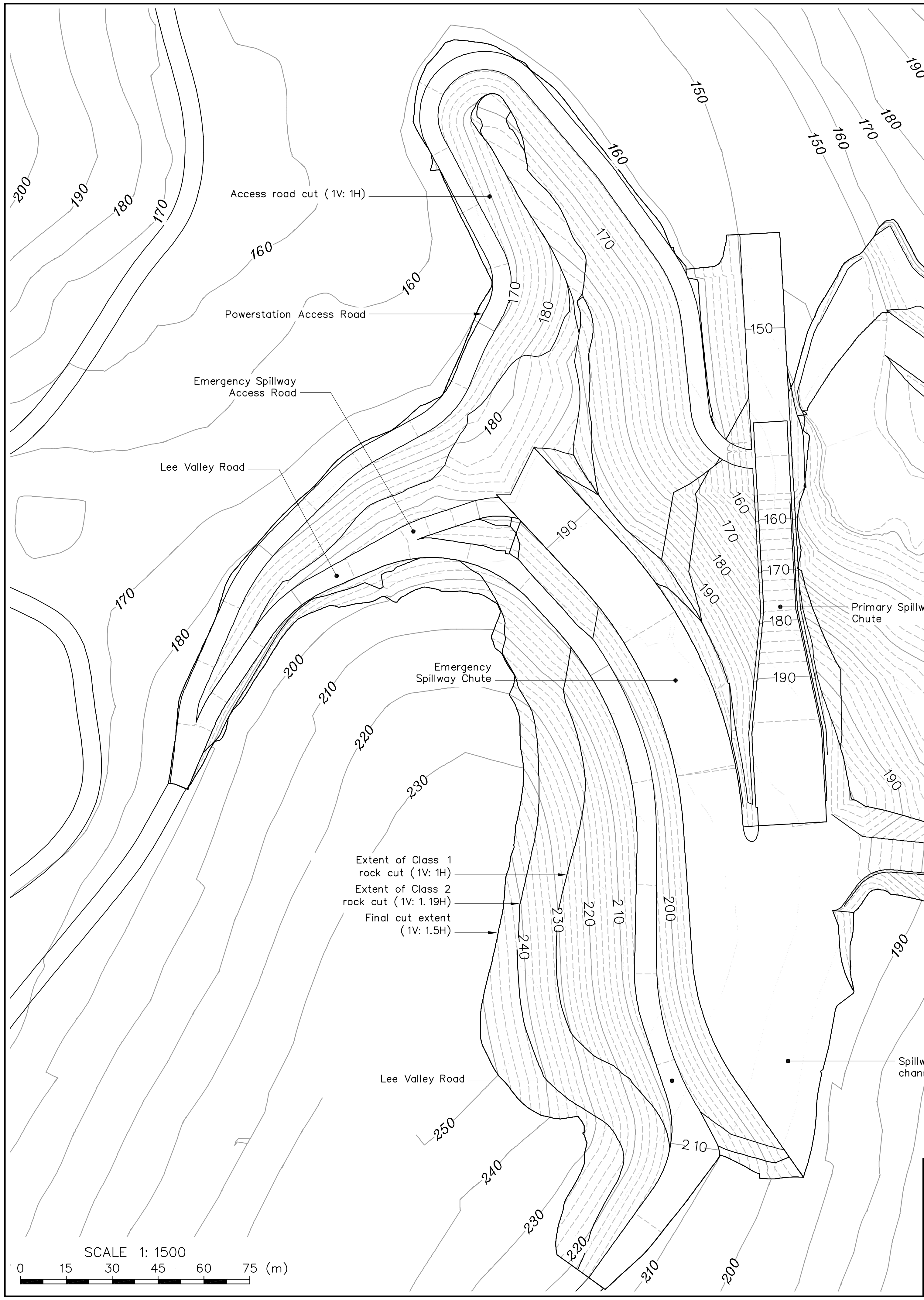
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
B–Dam Arrangement
Dam General Arrangement

FIG. No. B-01

REV. 2



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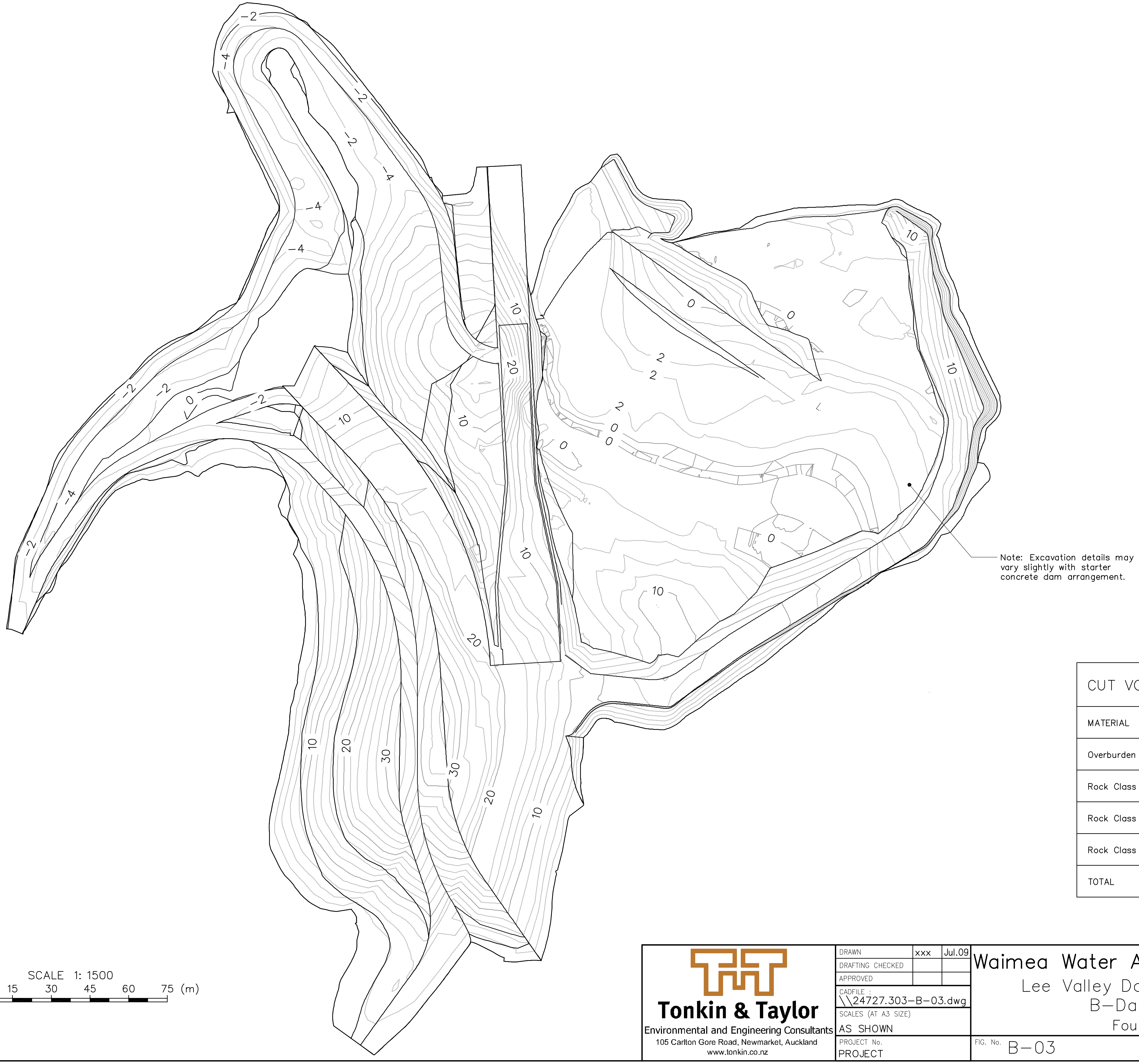
SCALES (AT A3 SIZE) AS SHOWN
PROJECT No. PROJECT

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
B–Dam Arrangement
Dam Foundation Cut and Stripping

FIG. No. B–02
REV. 1



SCALE 1: 1500

0 15 30 45 60 75 (m)

CUT VOLUMES	
MATERIAL	VOLUME – CUBIC METERS
Overburden	122,000
Rock Class 3	178,300
Rock Class 2	83,000
Rock Class 1	88,000
TOTAL	471,300

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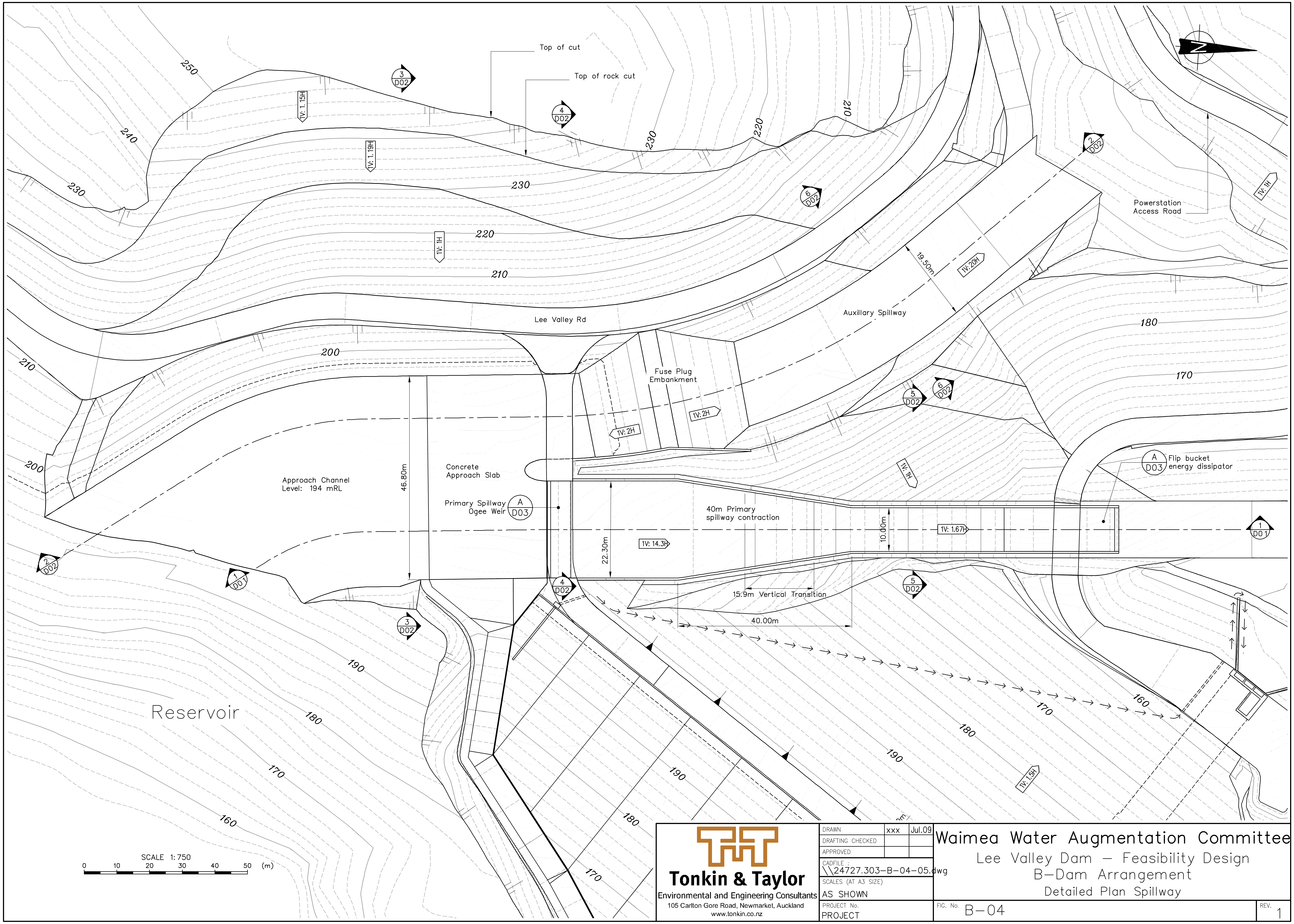
Waimea Water Augmentation Committee

Lee Valley Dam – Feasibility Design
B–Dam Arrangement
Foundation Isopach

FIG. No. B–03

REV. 2



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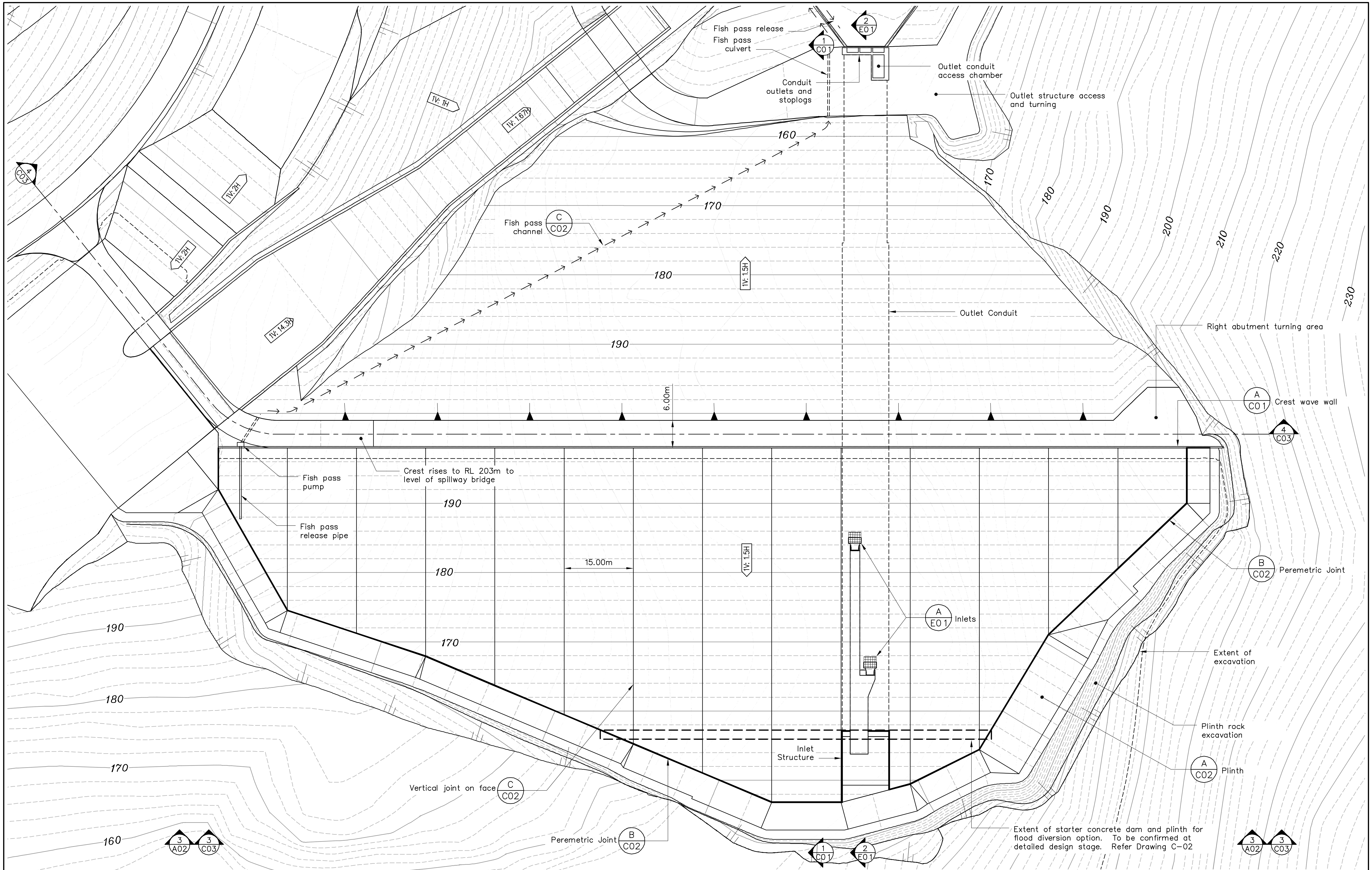
PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
B–Dam Arrangement
Detailed Plan Spillway

FIG. No. B–04 REV. 1



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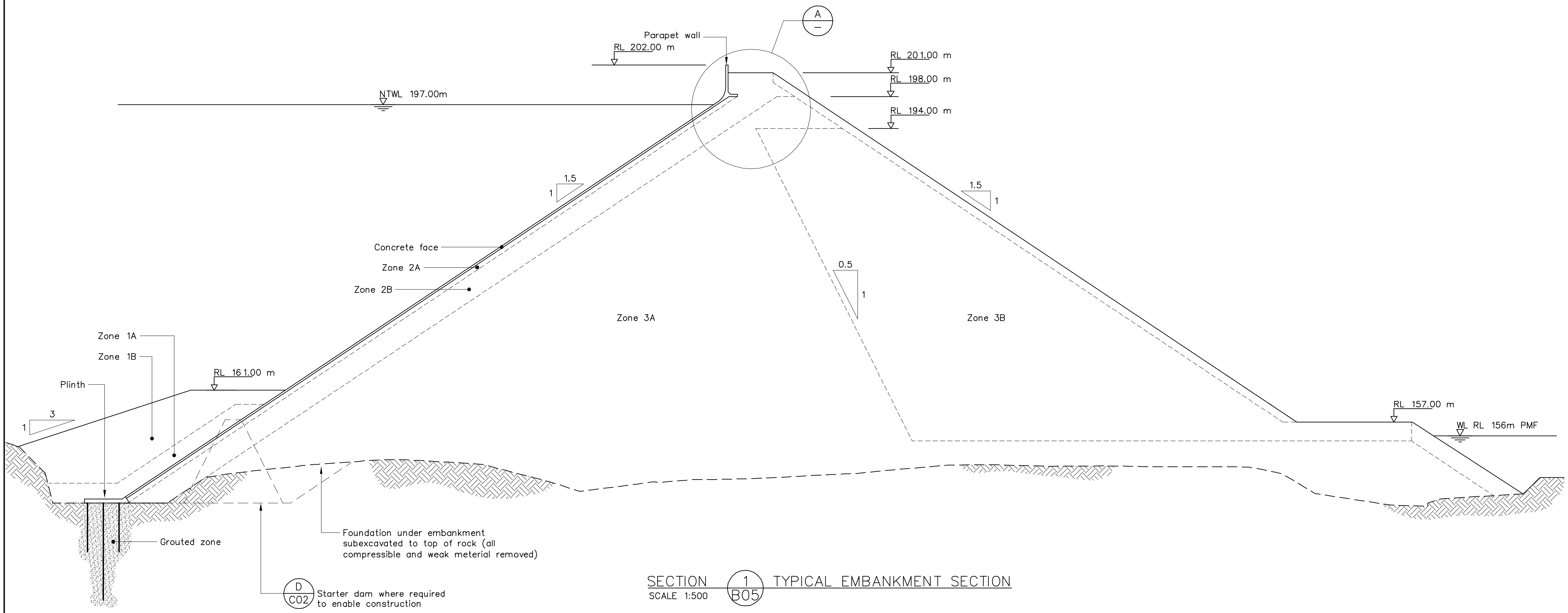
PROJECT No. PROJECT		

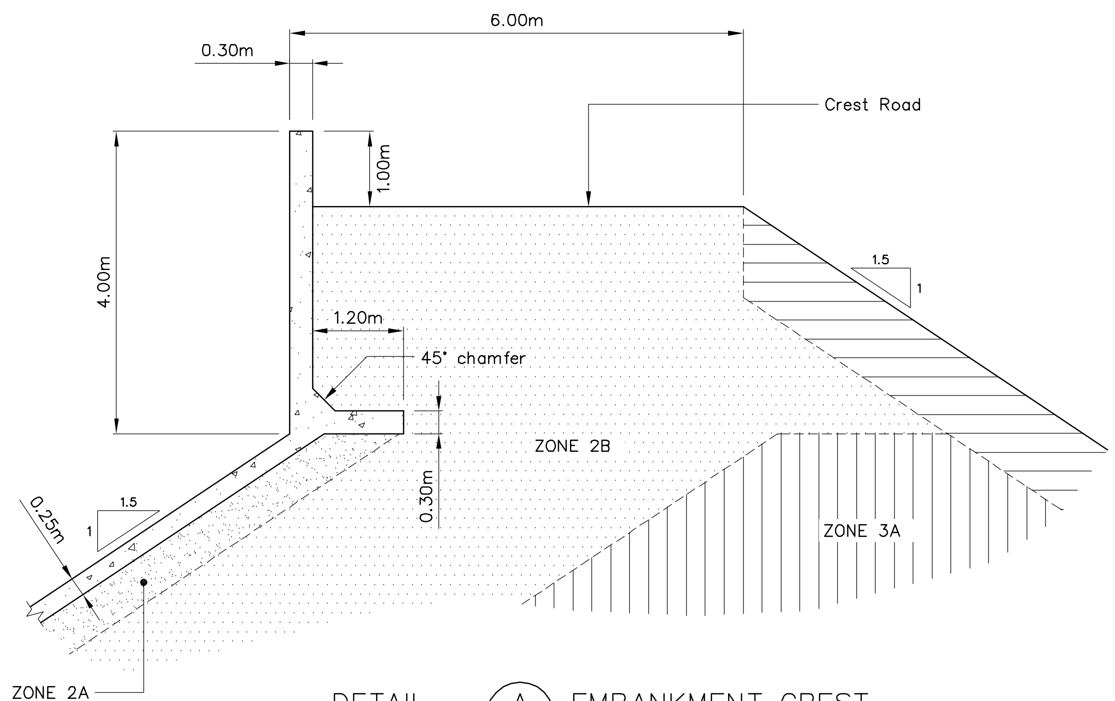
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
B-Dam Arrangement
Detailed Plan Dam

FIG. No. B-05

REV. 2



SECTION 1 B05 TYPICAL EMBANKMENT SECTION
SCALE 1:500



DETAIL A EMBANKMENT CREST
SCALE 1:100

FILL VOLUMES

MATERIAL	VOLUME – CUBIC METERS
Zone 1A	2,100
Zone 1B	7,300
Zone 2A	7,200
Zone 2B	30,500
Zone 3A	215,000
Zone 3B	144,000
TOTAL	406,100

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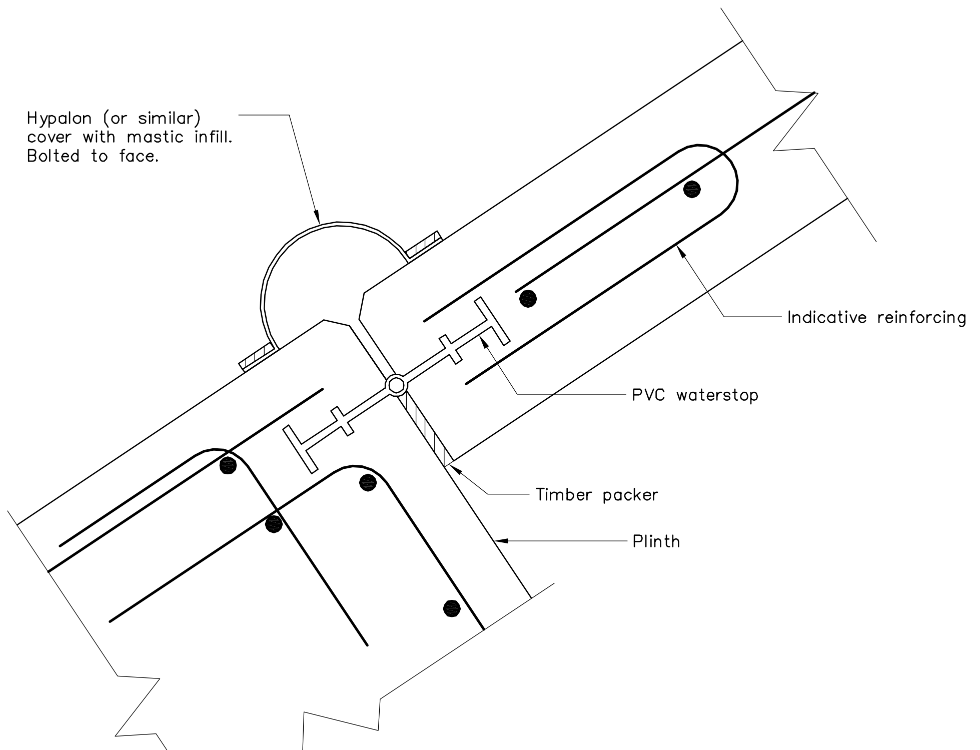
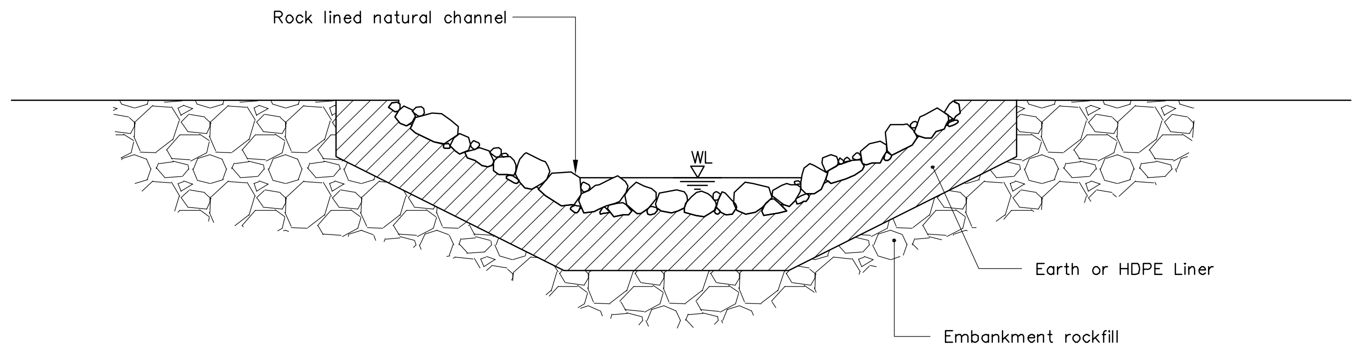
PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
C–Embankment
Embankment Cross Sections

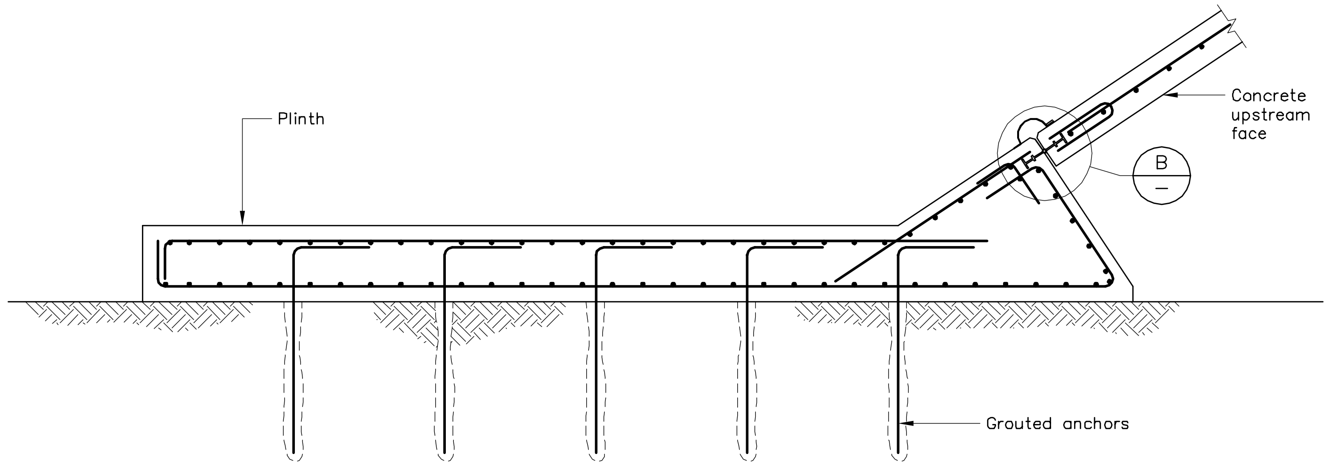
FIG. No. C–01
REV. 2



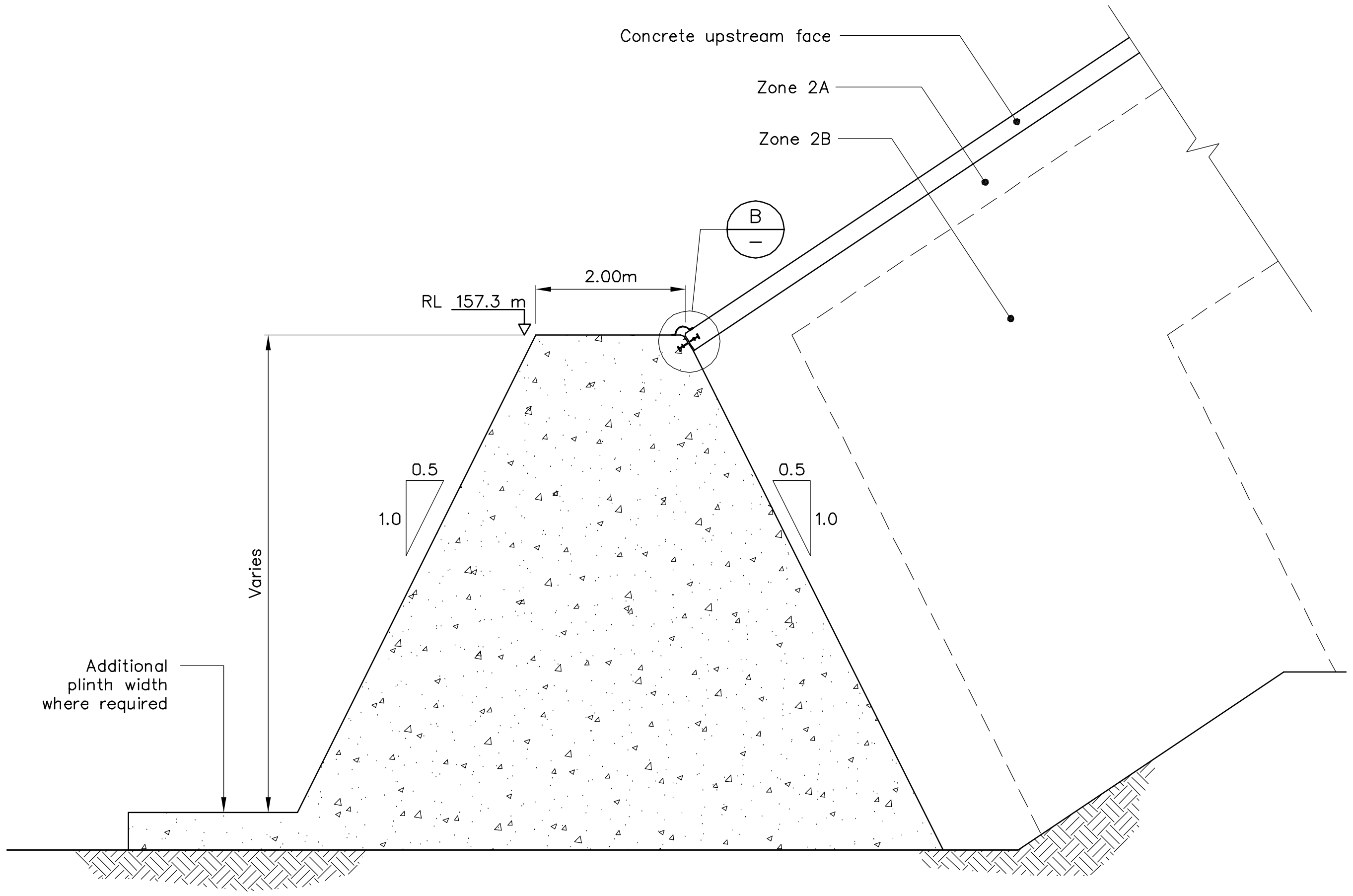
DETAIL B/B05 PERIMETRIC JOINT
SCALE 1: 10



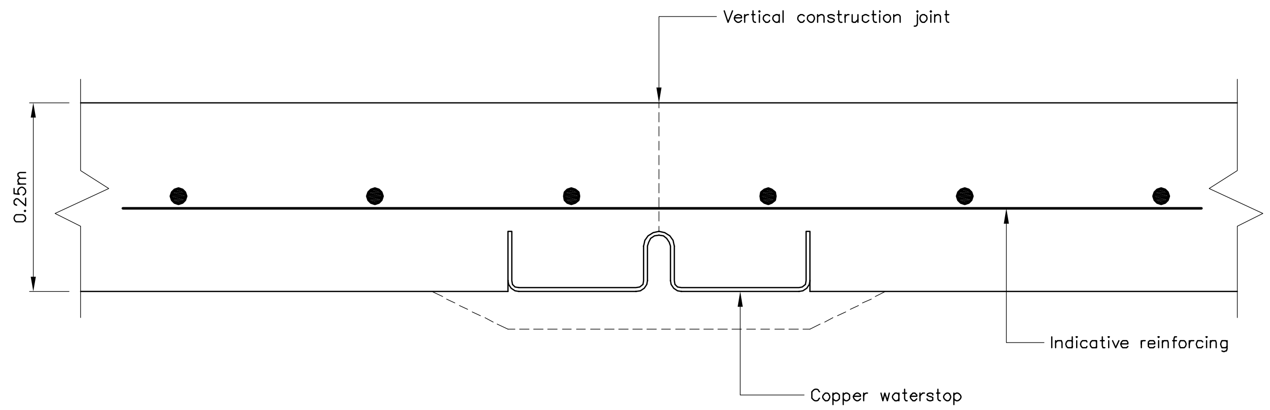
DETAIL C/B01 Fish passage channel
SCALE 1:20



DETAIL A/B05 PLINTH DETAIL
SCALE 1:50



DETAIL D/E01 STARTER DAM
SCALE 1: 100



DETAIL C/B05 VERTICAL JOINT ON FACE
SCALE 1: 10

SCALE 1: 10
0 0.1 0.2 0.3 0.4 0.5 (m)

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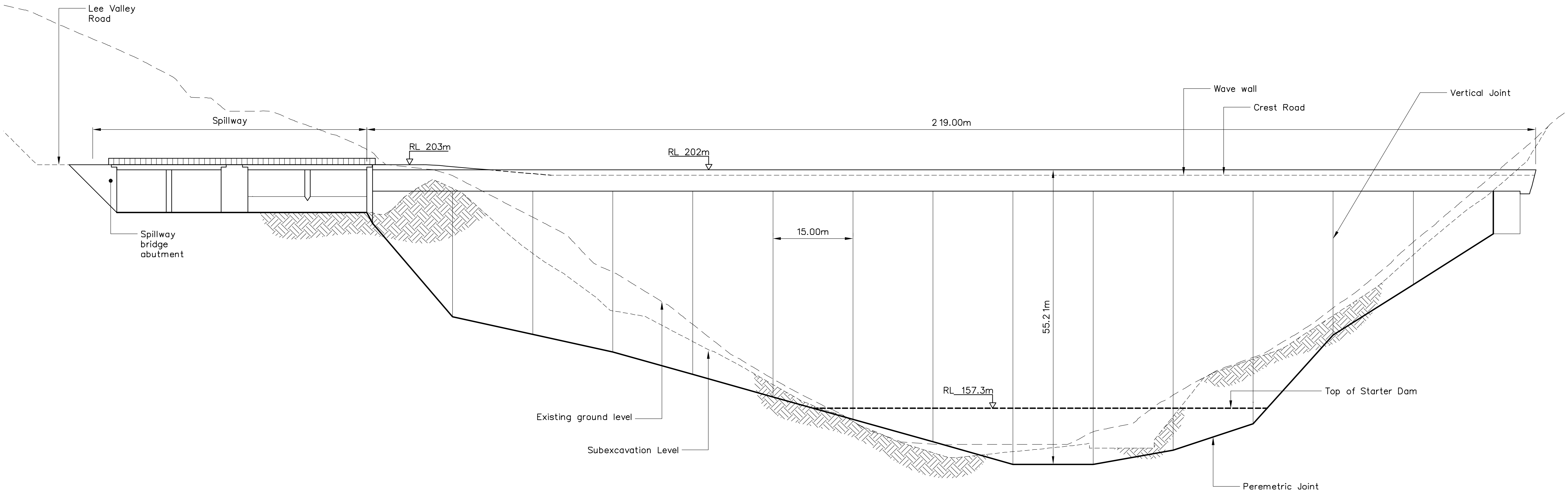
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
C–Embankment
Embankment Details

FIG. No. C–02
REV. 2



ELEVATION 4 B05 Dam Upstream Face
SCALE 1:750

SCALE 1: 750
0 10 20 30 40 50 (m)

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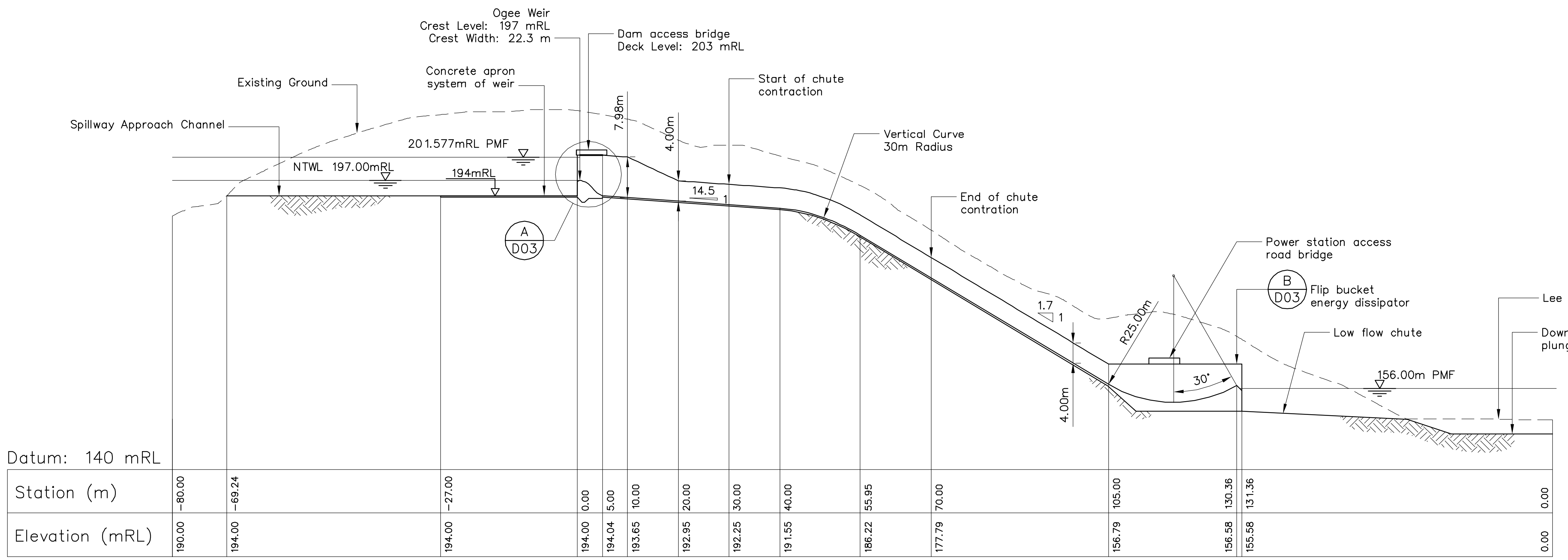
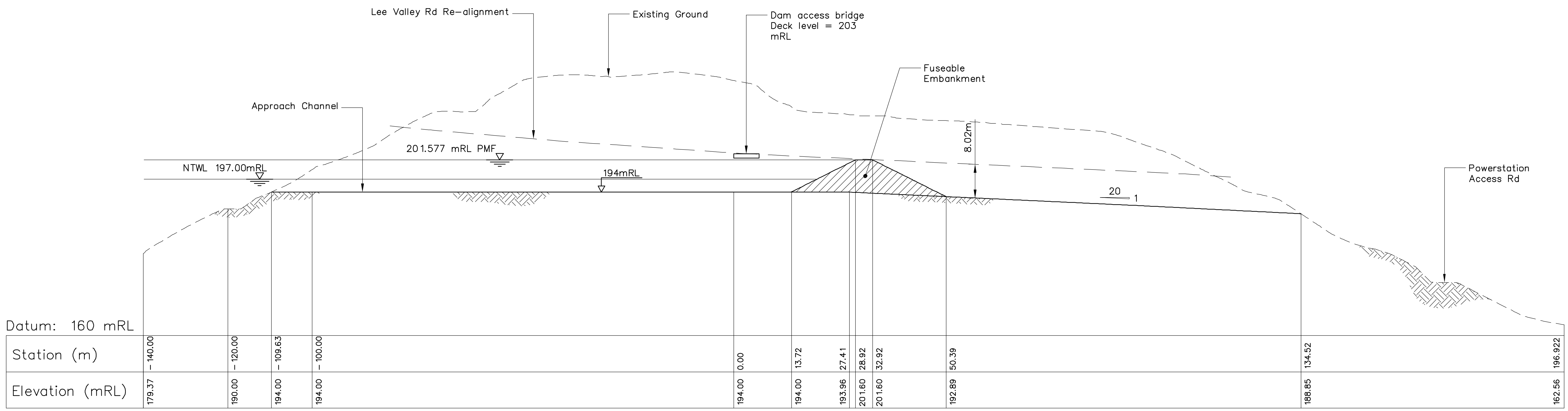
PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
C–Embankment
Elevation of Upstream Face

FIG. No. C–03
REV. 1



SECTION 1 B04 Primary Spillway
SCALE 1:1000



SECTION 2 B04 Auxillary Spillway
SCALE 1:1000

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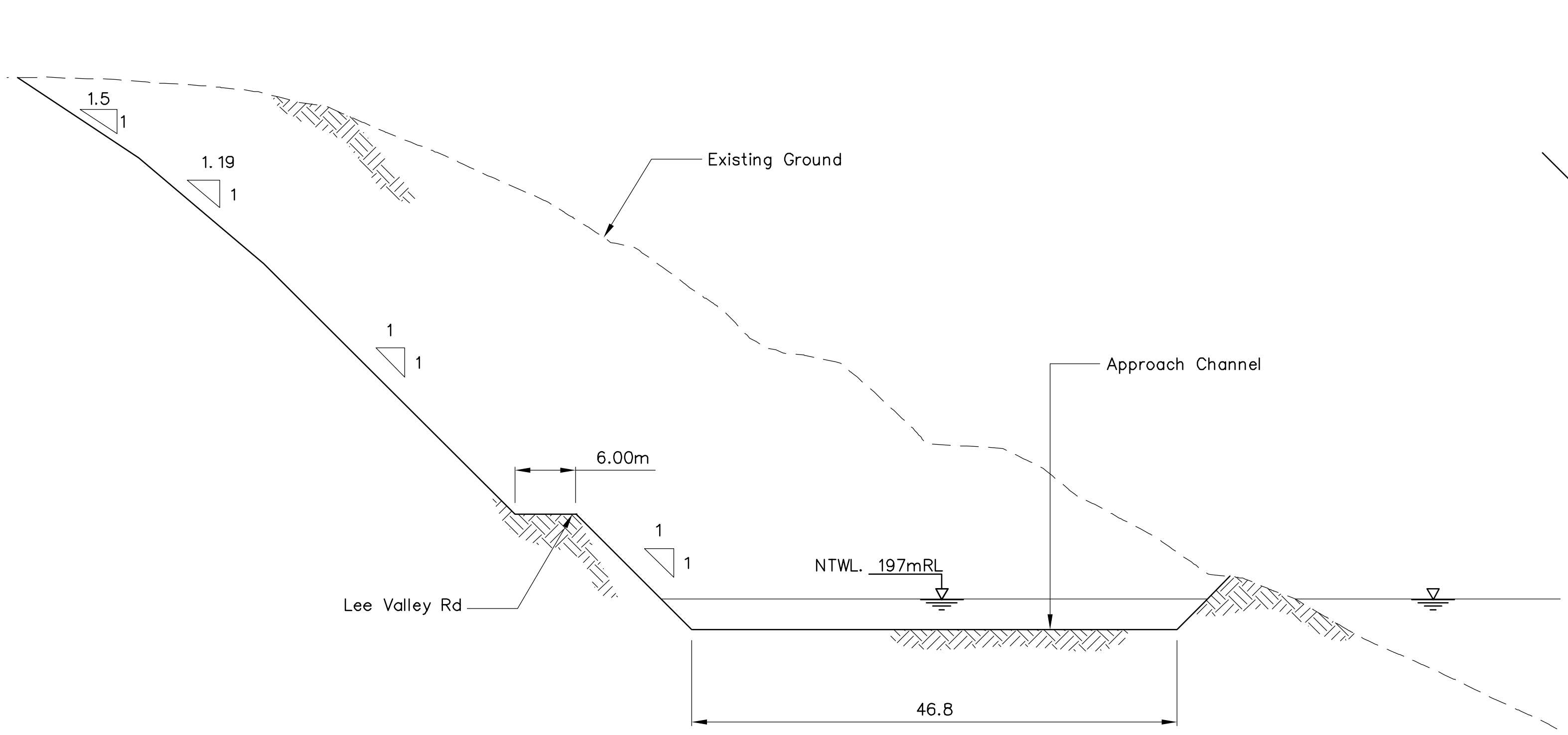
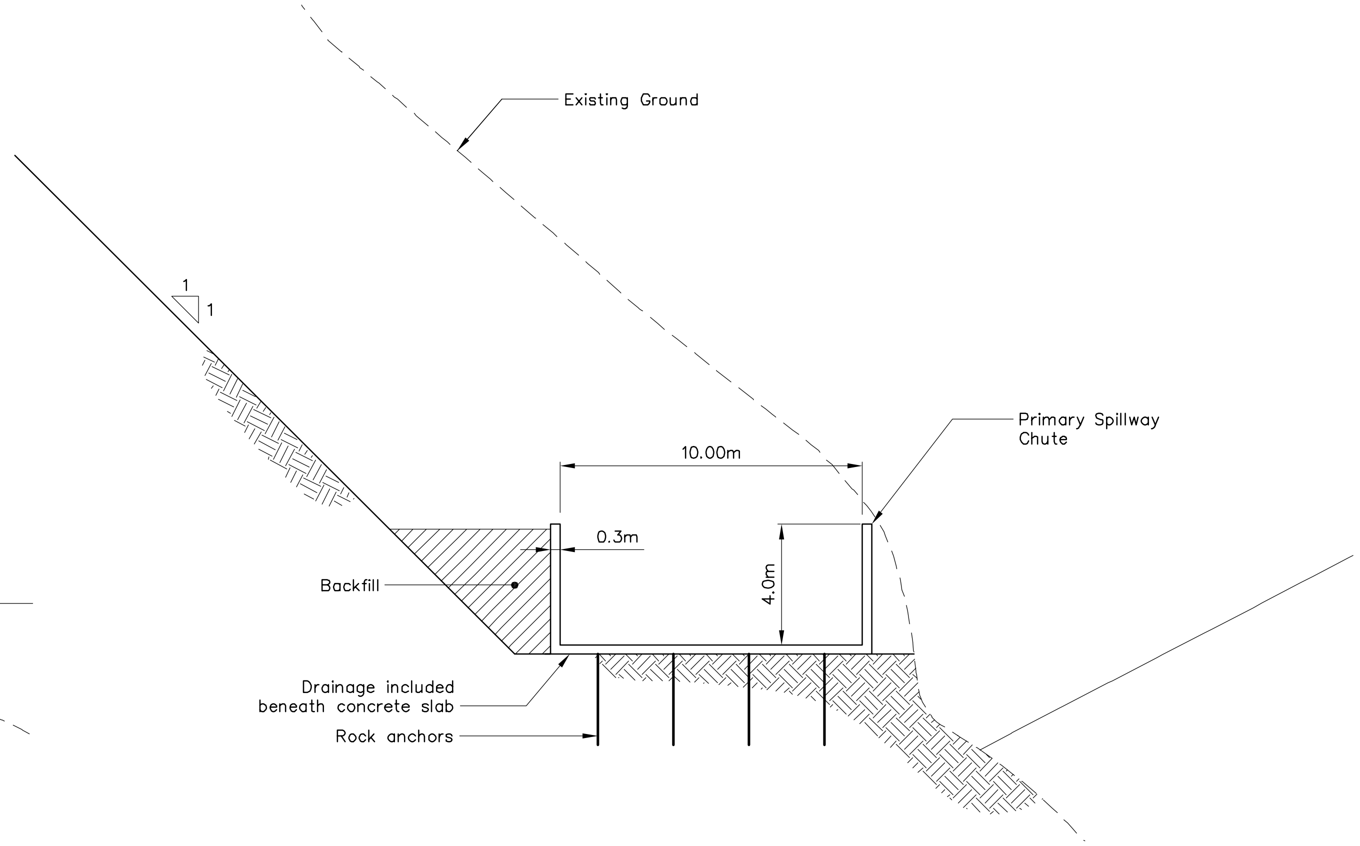
PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
D-Spillway
Spillway Long Sections

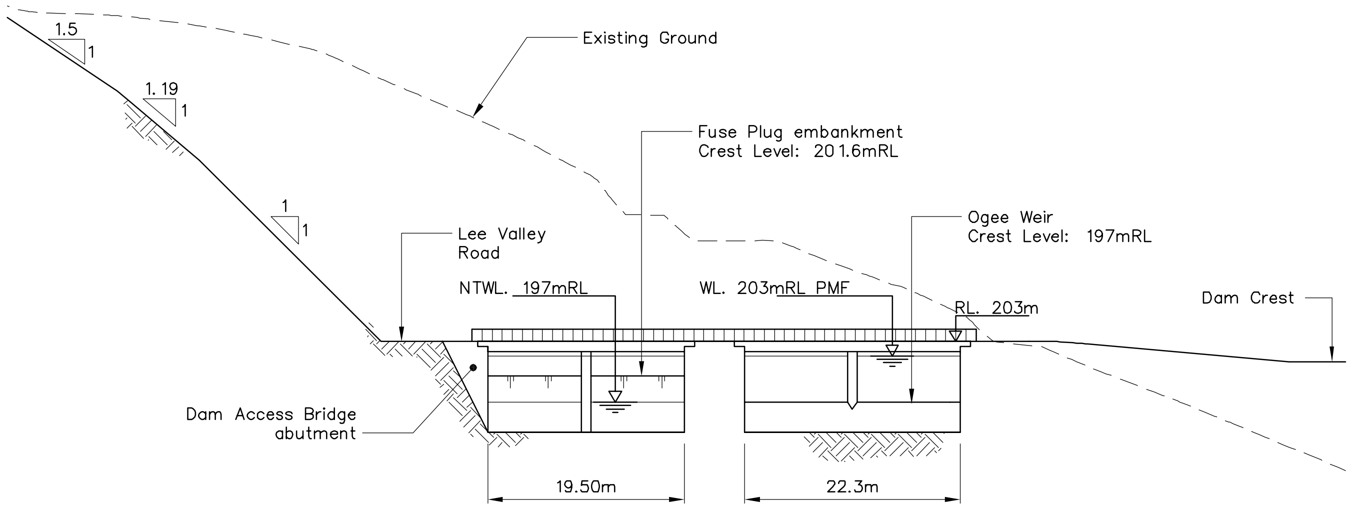
FIG. No. D-01
REV. 1



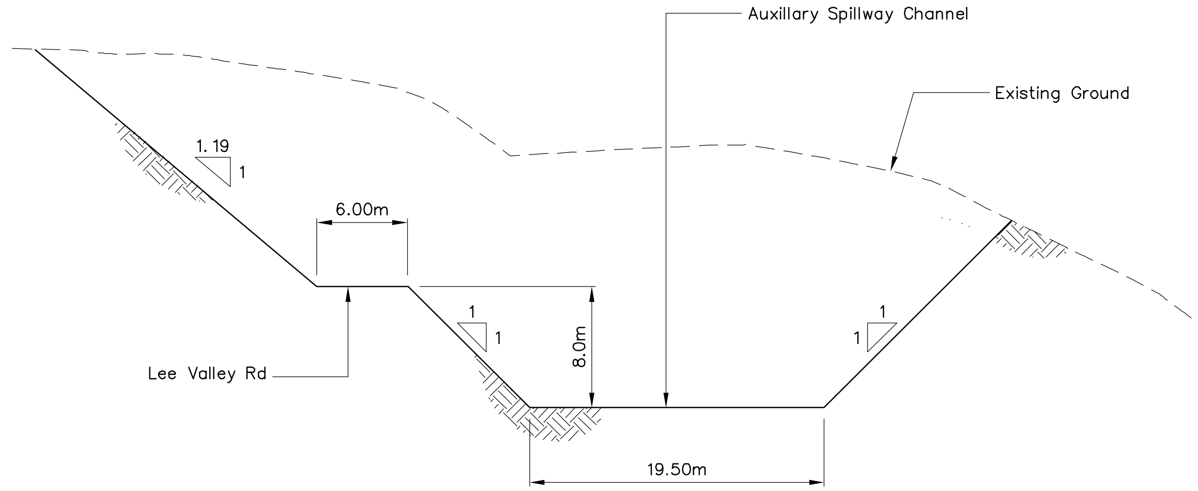
SECTION 3 B04 Spillway Approach Channel
SCALE 1:750



SECTION 5 B04 Spillway Chute
SCALE 1:250



SECTION 4 B04 Spillway Bridge and Weir Configuration
SCALE 1:750



SECTION 6 B04 Auxiliary Spillway Channel
SCALE 1:500

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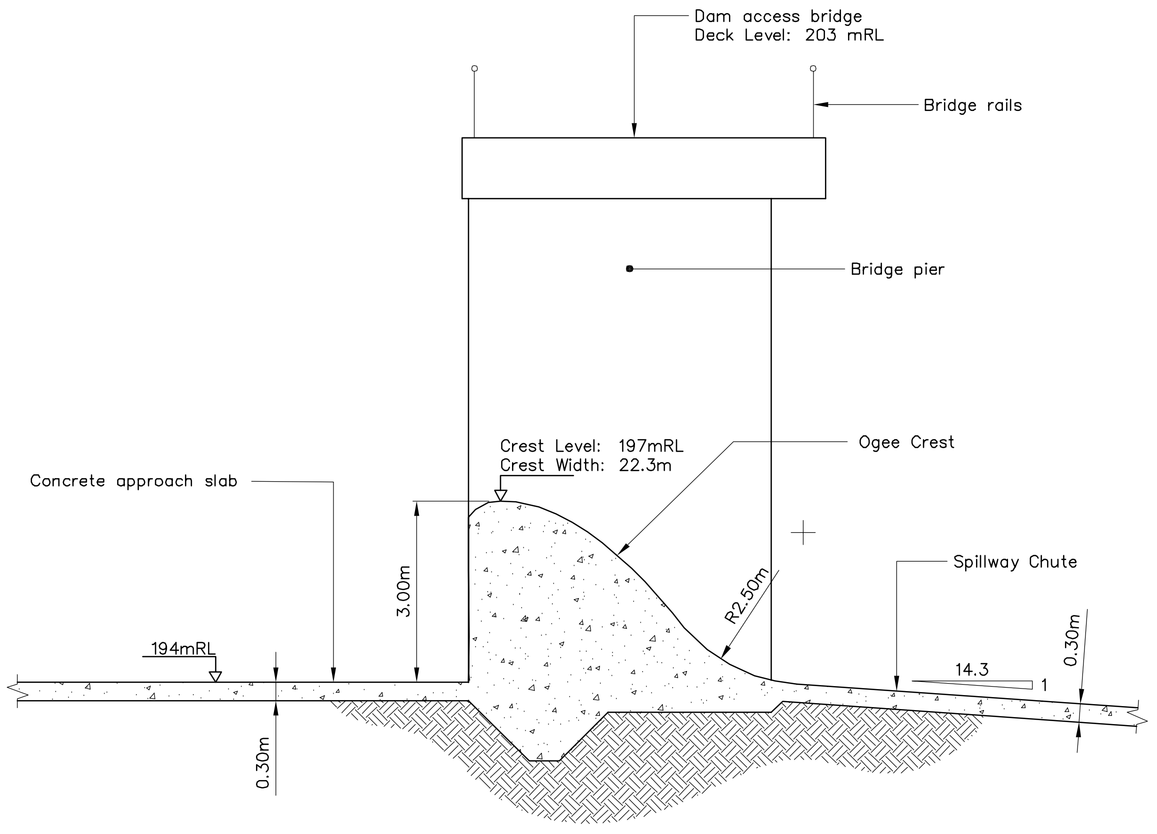
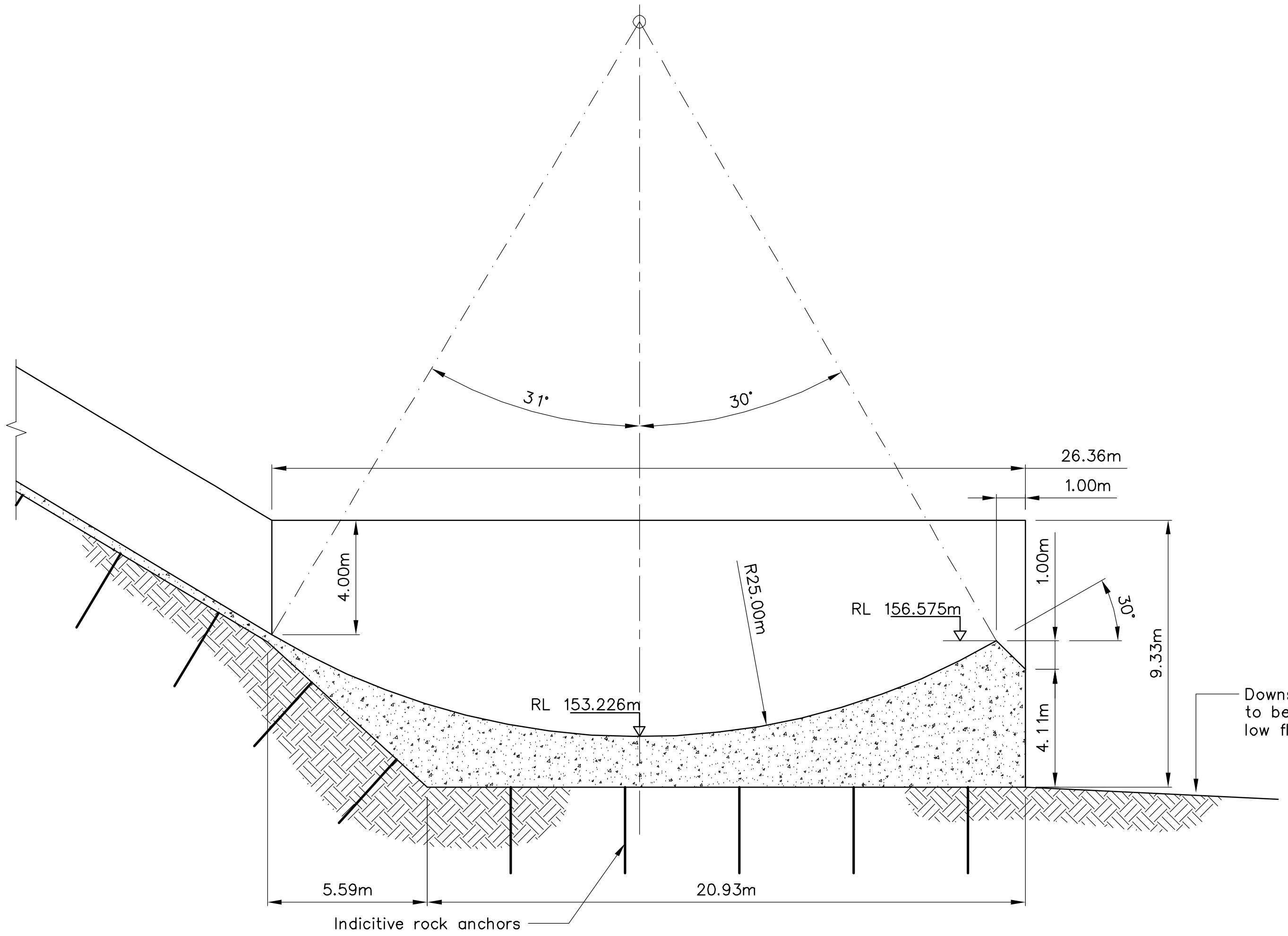
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
D–Spillway
Spillway Cross Sections

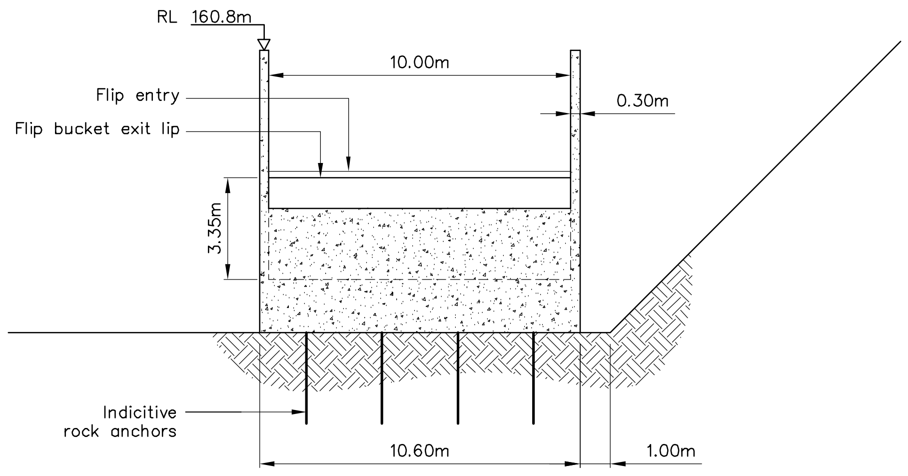
FIG. No. D–02
REV. 0



DETAIL A / D01 OGEE WEIR
SCALE 1:125



DETAIL B / B04 Spillway Flip Bucket
SCALE 1:250



SECTION 1 / —
SCALE 1:250

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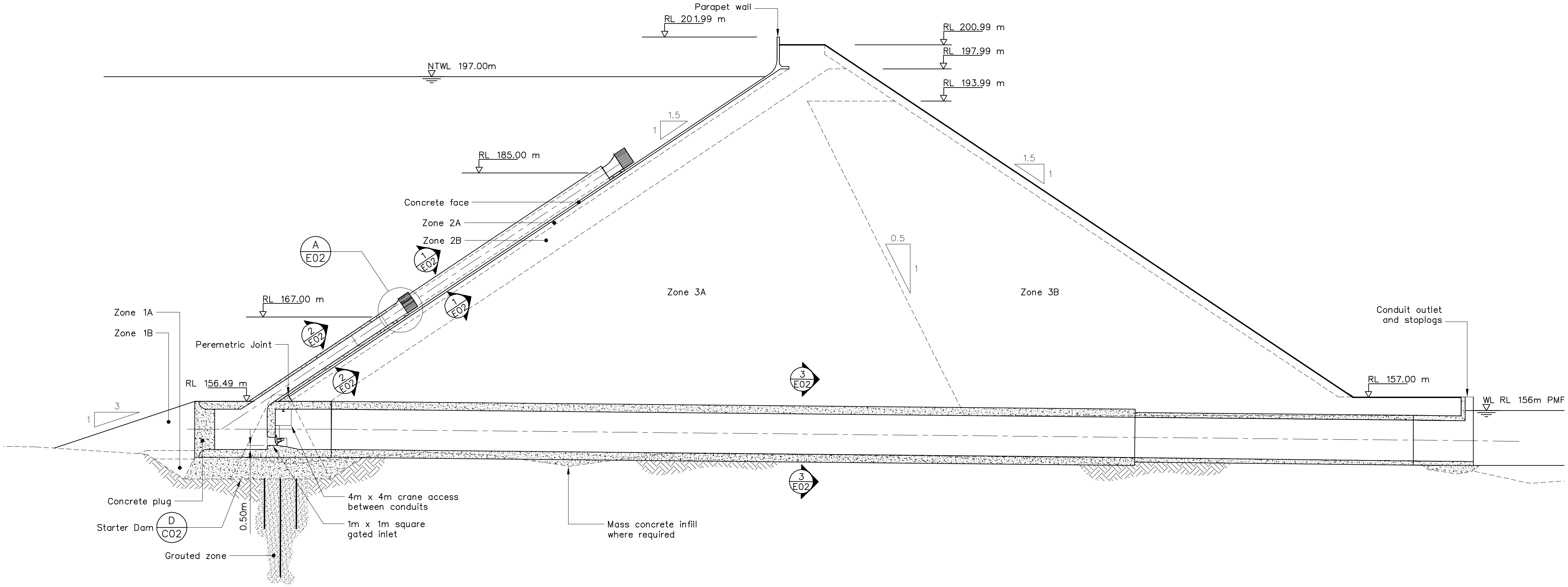
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
D–Spillway
Spillway Details

FIG. No. D–03
REV. 2



SECTION 2/B05 Outlet works

SCALE 1:500

SCALE 1: 500

0 5 10 15 20 25 (m)

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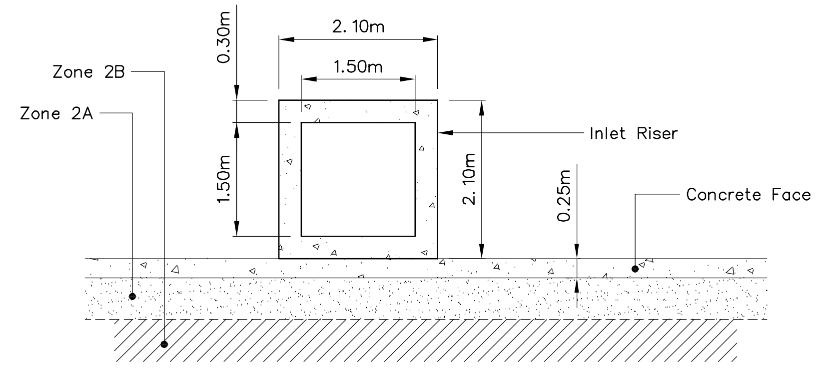
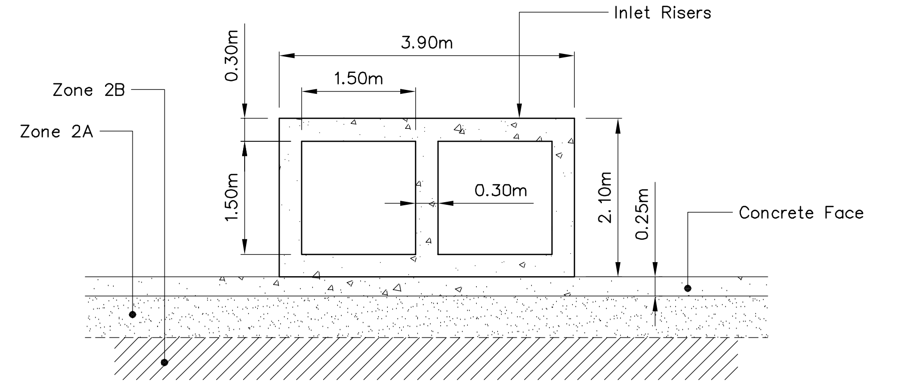
Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
E–Reservoir Outlet
Reservoir Outlet Arrangement

FIG. No. E–01

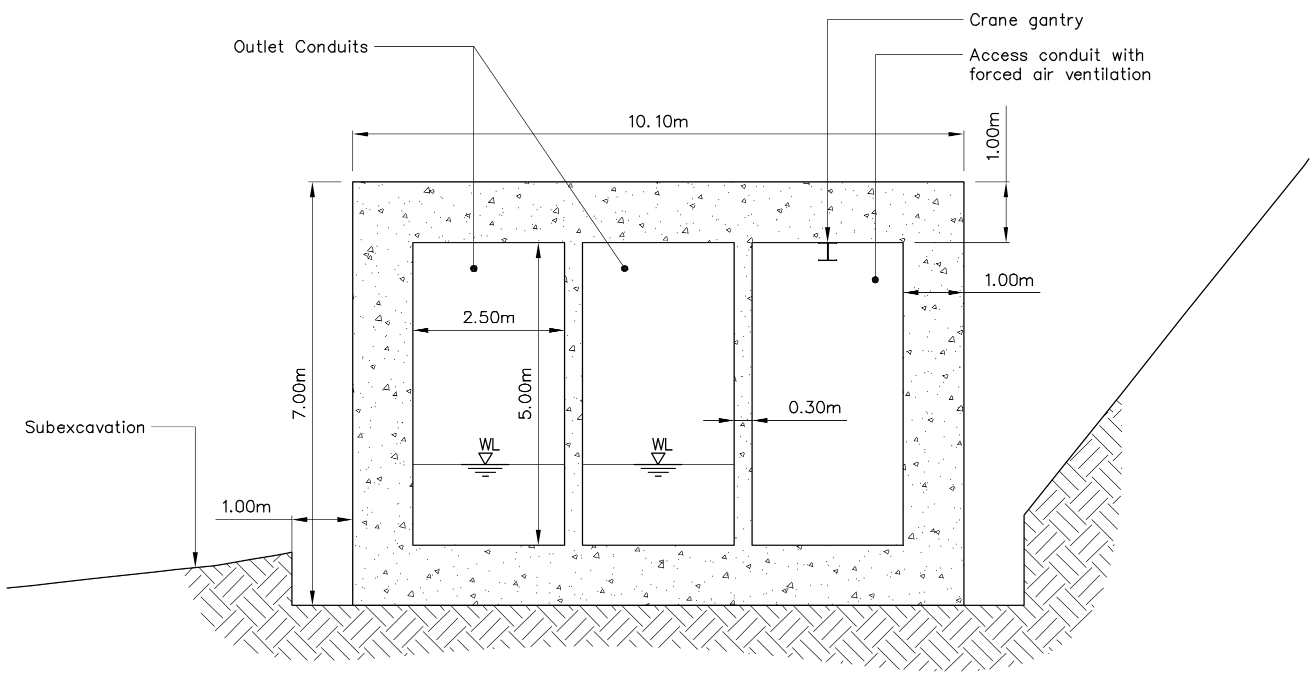
REV. 1



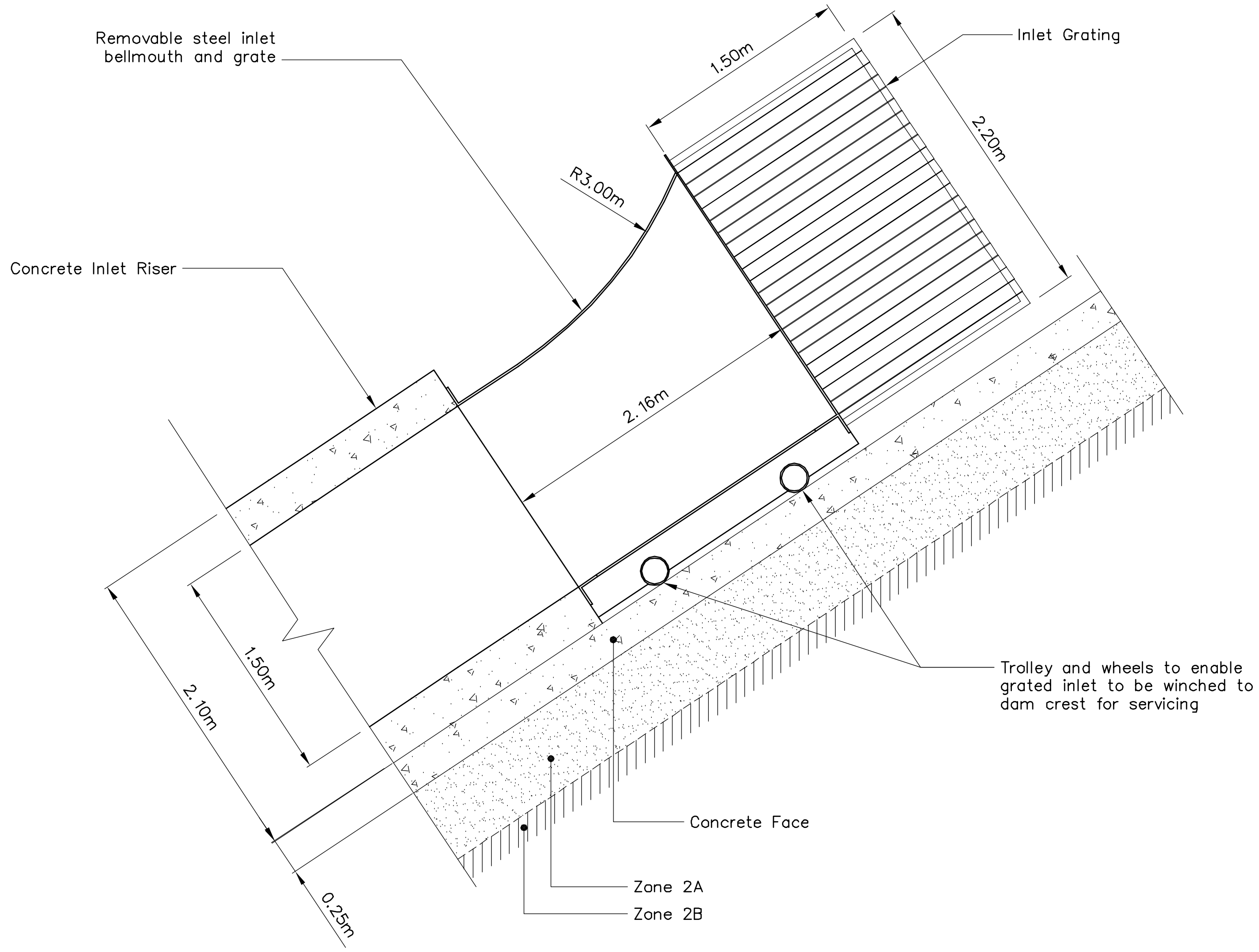
SECTION 1 Single Riser
SCALE 1:100 E01



SECTION 1 Double Riser
SCALE 1:100 E01



SECTION 3 Outlet Conduits
SCALE 1:125 E01



DETAIL A Inlet Grate and Bellmouth
SCALE 1:50 E01

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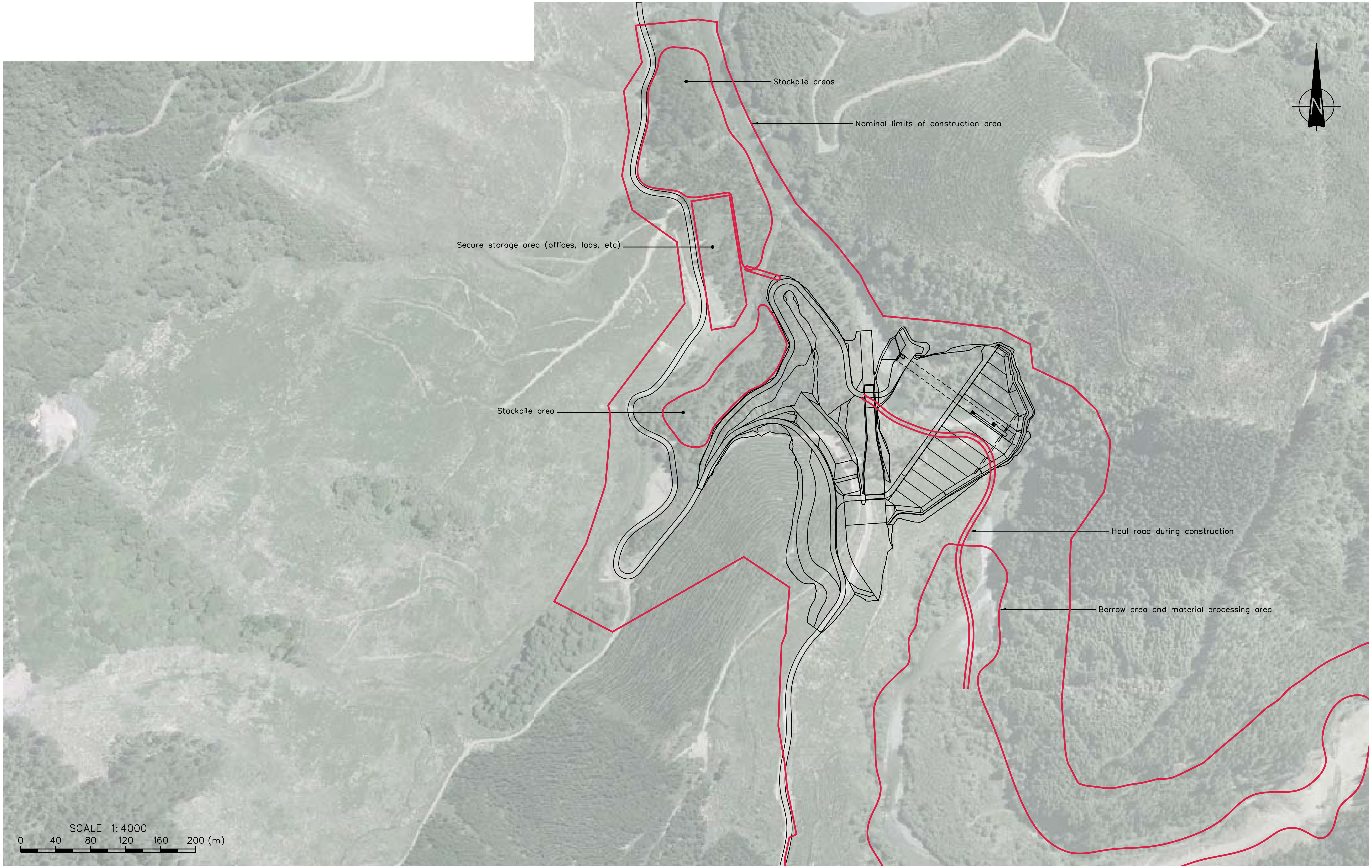
	PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
E–Reservoir Outlet
Reservoir Outlet Sections

FIG. No. E–02 REV. 0



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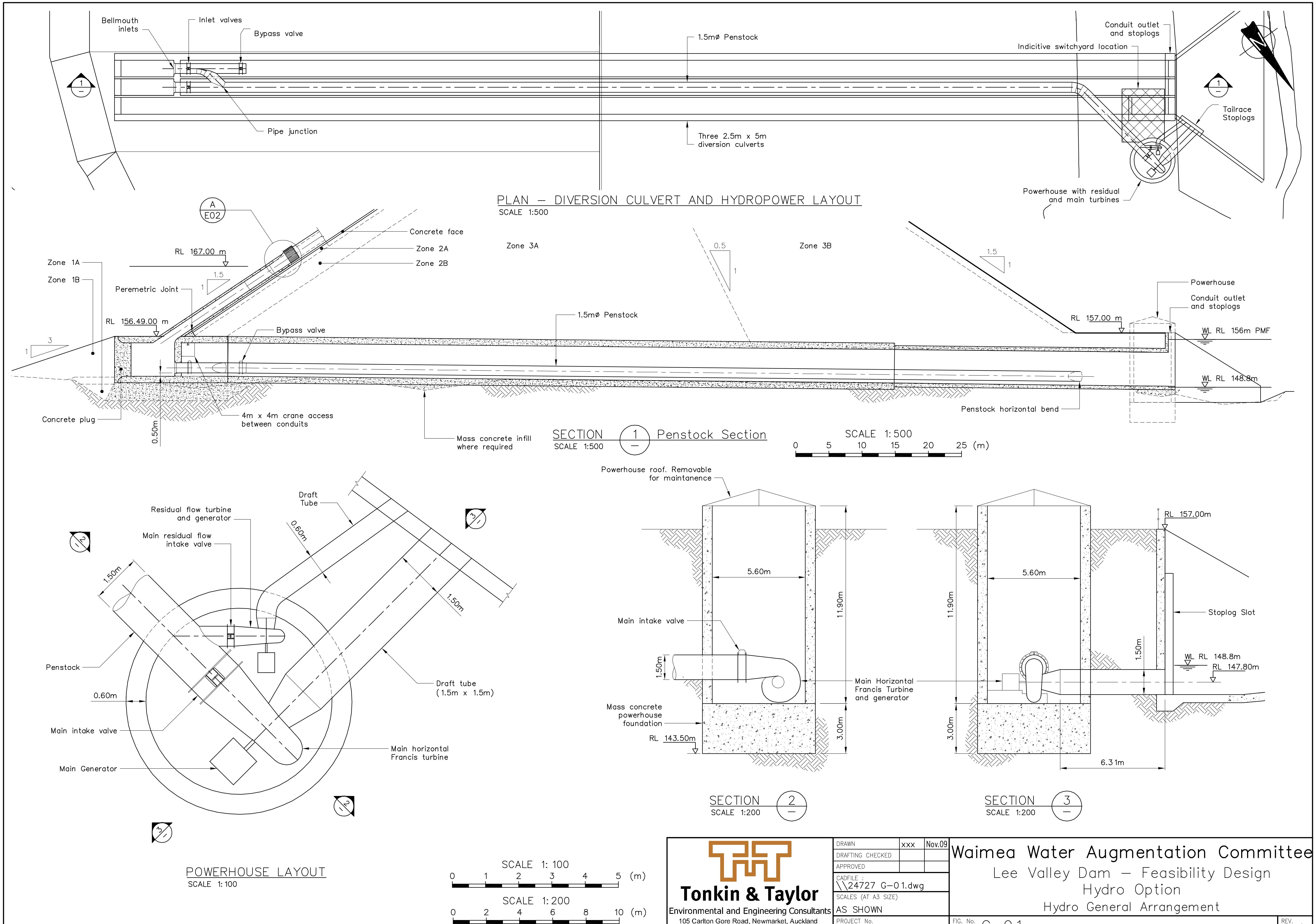
PROJECT No. PROJECT		

Waimea Water Augmentation Committee
Lee Valley Dam – Feasibility Design
F–Construction Methodology
Construction Flood Management

FIG. No. F–01

REV. 1



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Waimea Water Augmentation Committee

Lee Valley Dam – Feasibility Design
Hydro Option
Hydro General Arrangement

FIG. No. G-01

REV. 1

Appendix C: Cost Evaluation

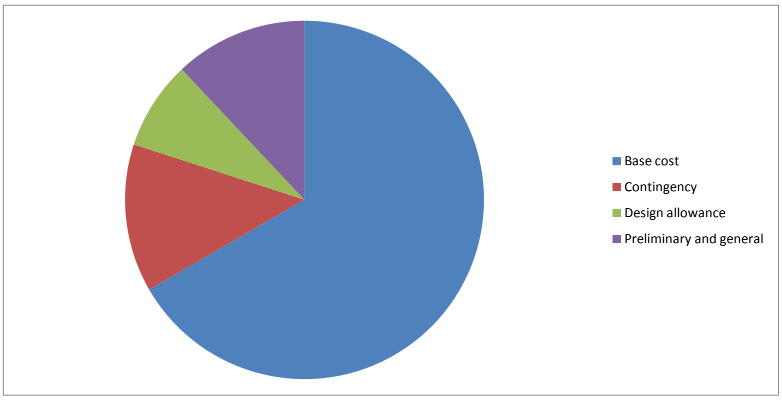
Components	% Cost	Current Cost Estimate Civil	E&M	Total	Previous Cost Estimate Civil	E&M	Total	Percent Variation Civil	E&M	Total
Site Preparation & Diversion Works	24.4%	$9,290,582	$0	$9,290,582	$10,316,730	$0	$10,316,730	-10%	100%	-10%
Plinth Work	11.1%	$4,233,405	$0	$4,233,405	$4,379,565	$0	$4,379,565	-3%	100%	-3%
Embankment	39.7%	$15,143,520	$0	$15,143,520	$14,916,046	$0	$14,916,046	+2%	100%	+2%
Spillway	18.1%	$6,915,061	$0	$6,915,061	$6,600,268	$0	$6,600,268	+5%	100%	+5%
Outlet Works	3.5%	$915,064	$434,868	$1,349,931	$941,034	$434,868	$1,375,902	-3%	0%	-2%
Fishpass	0.2%	$87,375	$0	$87,375	$87,375	$0	$87,375	+0%	100%	+0%
Access	3.0%	$1,127,258	$0	$1,127,258	$1,127,258	$0	$1,127,258	+0%	100%	+0%
TOTALS	100.0%	$37,712,264	$434,868	$38,147,131	$38,368,275	$434,868	$38,803,143	-2%	0%	-2%

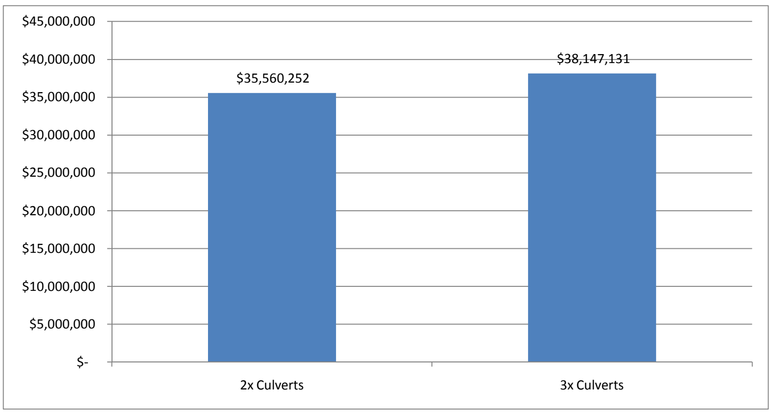
	Civil	E&M	Total	
Base Cost Included Above	$ 25,141,509	$ 343,769	$ 25,485,278	Base cost
Contingency Allowance Included Above	$ 5,028,302	$ 34,377	$ 5,062,679	Contingency
Design Allowance Included Above	$ 3,016,981	$ 18,907	$ 3,035,888	Design allowance
P&G Allowance Included Above	$ 4,525,472	$ 37,815	$ 4,563,286	Preliminary and general
TOTALS	$ 37,712,264	$ 434,868	$ 38,147,131	

Contingency Allowance Calculated Explicitly $ 2,687,244.00

Costs with 2x Culverts

Site Preparation & Diversion Works	18.9%	$ 6,703,702.05	$ -	$6,703,702	2x Culverts	$ 35,560,252
Plinth Work	11.9%	$4,233,405	$0	$4,233,405	3x Culverts	$ 38,147,131
Embankment	42.6%	$15,143,520	$0	$15,143,520		
Spillway	19.4%	$6,915,061	$0	$6,915,061		
Outlet Works	3.8%	$915,064	$434,868	$1,349,931		
Fishpass	0.2%	$87,375	$0	$87,375		
Access	3.2%	$1,127,258	$0	$1,127,258		
TOTALS		$ 35,125,384.26	$ 434,867.53	$ 35,560,251.79		





Description	Code	Unit	Rate
Earthworks & Related			
Soil cut to fill (Zone 1A)	cutToFill1A	cu.m	$ 5.00
Soil cut to fill (Zone 1B)	cutToFill1B	cu.m	$ 5.00
Soil cut to fill incl processing (Zone 2A)	cutToFill2A	cu.m	$ 20.00
Soil cut to fill incl processing (Zone 2B)	cutToFill2B	cu.m	$ 20.00
Rock cut to fill (Zone 3A)	cutToFill3A	cu.m	$ 10.00
Rock cut to fill (Zone 3B)	cutToFill3B	cu.m	$ 10.00
Rock cut to waste	cutToWasteRock	cu.m	$ 8.00
Soil cut to waste	cutToWasteSoil	cu.m	$ 4.00
Foundation cleanup for concrete placement	fndCleanConc	sq.m	$ 10.00
Road aggregate	roadAgg	cu.m	$ 60.00
Shotcrete slope protection	slopeProtect	sq.m	$ 100.00
100 dia HDPE pipe	pipeHDPE100	m	$ 20.00
Dowell anchor drilled into rock	anchorDowell	no	$ 200.00
Heavy rock armour	armRockHeavy	cu.m	$ 100.00
Treatment of foundation defects at plinth	defectTreat	no	$ 4,000.00
Drill and grout	drillGrout	m	$ 500.00
Seconday Curtian Drill and grout	SecDrillGrout	m	$ 400.00
Tertiary Curtian Drill and grout	TerDrillGrout	m	$ 300.00
Geotextile	geoTex	sq.m	$ 10.00
Grouted rock	groutRock	cu.m	$ 150.00
750 dia concrete culvert	culvert750	m	$ 750.00
Sheet Piling	SteelSheetPile	t	$ 4,000.00
Bulk borrow to fill	borToFill	cu.m	$ 5.00
Liner placement	liner	cu.m	$ 12.00
Liner protection armour	linerPro	cu.m	$ 6.00
Wave armour	waveArm	cu.m	$ 12.00
Topsoil stripping	strip	cu.m	$ 2.00
Topsoiling and grassing	topsoil	sq.m	$ 2.00
Hydroseeding	hydroseed	sq.m	$ 2.00
Filter material	filter	cu.m	$ 60.00
Coarse filter	corFilter	cu.m	$ 60.00
Drainage material	drainage	cu.m	$ 60.00
Drainage pipe	drnPipe	m	$ 30.00
Steel sheet piling	SteelSheetPile	t	$ 4,000.00
Crest fence (farm type)	wireFence	m	$ 10.00
Heavy armour	armourHeavy	cu.m	$ 60.00
Coffer dam placement	cofferPlace	cu.m	$ 12.00
Coffer dam removal	cofferRemove	cu.m	$ 8.00
Piles	piles	m	$ 1,000.00
Heavy rubber bearings	heavyBearing	no	$ 4,000.00
Light rubber bearings	lightBearing	no	$ 2,000.00
Structural & Power Station Related			
Mass concrete	massConc	cu.m	$ 350.00
Structural concrete including reo	struConc	cu.m	$ 800.00
Formwork - straight	formStr	sq.m	$ 150.00
Formwork - curved	formCur	sq.m	$ 350.00
Formwork - slip formed	formSlip	sq.m	$ 100.00
Reinforcing steel	reoSteel	t	$ 3,200.00
Structural steel	strSteel	t	$ 5,000.00
Roller compacted concrete	RCC	cu.m	$ 150.00
Perimetric joint waterstop	periWS	m	$ 100.00
Waterstop joint	watStop	m	$ 50.00
Radial gate steel	steelRadGate	t	$ 18,000.00
Stoplog gate steel	steelStopLog	t	$ 10,000.00
General hydraulic structure steel	steelHydGen	t	$ 12,000.00
Steel pipe	steelPipe	t	$ 4,000.00
2 L/s capacity pump	puml2LS	No	$ 2,000.00
Road bridge cost based on plan area	bridgePlan	sq.m	$ 1,250.00
Retaining wall based on face area	retainWall	sq.m	$ 300.00
Instrumentation			

Flow monitoring equipment	flowMon	no	$ 1,500.00
Deformation marker	defMark	no	$ 1,000.00

Contingencies, percentages etc

Civil minor items	5%
Civil contingency	20%
Civil engineering	10%
Civil P&G	15%
E&M minor items	5%
E&M contingency	10%
E&M engineering	5%
E&M P&G	10%

This sheet contains base information for titles, revs etc on all other sheets & is dynmically linked.

Project:	LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES	
JobNo:	24727.303	
Current Ver & date:	Ver 01	9-Nov-09

Release History	Ver	Date	Notes
	00	9-Nov-09	First cut at construction cost estimate, prior to internal/external review
	01	9-Nov-09	Following constructor review and prelim optimisation

LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES Ver 01
ITEM 1 Site Preparation & Diversion Works 9-Nov-09

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
1.1	**Diversion Through Permanent Culverts**						
1.1.1	Excavation	cu.m	8,711.	cutToWasteSoil	$ 4	$ 34,844.00	
1.1.2	Diversion Culverts (Upstream Section)						
1.1.2.1	Concrete	cu.m	4,382.	massConc	$ 350	$ 1,533,700.00	
1.1.2.2	Steel	t	645.3	reoSteel	$ 3,200	$ 2,064,960.00	
1.1.2.3	Formwork	sq.m	6,798.	formStr	$ 150	$ 1,019,700.00	
1.1.3	Diversion Culverts (Downstream Section)						
1.1.3.1	Concrete	cu.m	826.56	massConc	$ 350	$ 289,296.00	
1.1.3.2	Steel	t	166.9	reoSteel	$ 3,200	$ 534,080.00	
1.1.3.3	Formwork	sq.m	2,045.9	formStr	$ 150	$ 306,885.00	
	Subtotal						$ 5,783,465.00
1.2	**Diversion Through Temporary Culverts (upstream end)**						
1.2.1	Upstream coffer dam	cu.m	1,800.	cutToWasteSoil	$ 4	$ 7,200.00	
1.2.2	50m long by 5 m high Sheet Piling	t	11.3	SteelSheetPile	$ 4,000	$ 45,200.00	
1.2.3	Miscellaneous sealing concrete	cu.m	20	massConc	$ 350	$ 7,000.00	
1.2.4	Dewatering allowance during low plinth construction	LS	1		$ 100,000	$ 100,000.00	
	Subtotal						$ 159,400.00
1.2	**Dam Site Preparation**						
1.2.1	Stripping (Cut to Waste)	cu.m	62,714.	cutToWasteSoil	$ 4	$ 250,856.00	
	Subtotal						$ 250,856.00
	SUBTOTAL CIVIL ITEMS					$ 6,193,721.00	
	Civil contingency allowance		$ 6,193,721.00		20%	$ 1,238,744.20	
	CIVIL INCL CONTINGENCY					$ 7,432,465.20	
	Civil Engineering		$ 7,432,465.20		10%	$ 743,246.52	
	Civil P&G		$ 7,432,465.20		15%	$ 1,114,869.78	
	TOTAL CIVIL						$ 9,290,581.50
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 9,290,581.50**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
2.1	**Preparation**						
2.1.1	Excavation (Cut to Fill)	cu.m	21,262.	cutToFill3B	$ 10	$ 212,620.00	
2.1.2	Foundation Cleaning (plan area)	sq.m	5,833.	fndCleanConc	$ 10	$ 58,330.00	
2.1.3	Defects (1 per 200 sq.m)	no	30.	defectTreat	$ 4,000	$ 120,000.00	
2.1.4	Slope Reinforcement (Face area of 0.25H:1V Slope)	sq.m	1,986.	slopeProtect	$ 100	$ 198,600.00	
	Subtotal						$ 589,550.00
2.2	**Plinth**						
2.2.1	Drilling and Grouting						
2.2.2.1	Length of 15m deep Primary Curtain Grouting	m	394.	drillGrout	$ 500	$ 197,000.00	
2.2.2.2	Length of 7m deep Secondary Curtian Grouting	m	184.	SecDrillGrout	$ 400	$ 73,600.00	
2.2.2.3	Length of 7m deep Tertiary Curtian Grouting	m	368.	TerDrillGrout	$ 300	$ 110,400.00	
2.2.2	Grouted Anchor Bars (1m depth at 1m centers)	no	1,778.	anchorDowell	$ 200	$ 355,600.00	
2.2.3	Plinth						
2.2.3.1	Concrete	cu.m	889.	massConc	$ 350	$ 311,150.00	
2.2.3.2	Formwork	t	352.5	reoSteel	$ 3,200	$ 1,128,000.00	
2.2.3.3	Steel	sq.m	177.8	formStr	$ 150	$ 26,670.00	
2.2.4	Peremetric Joint (waterstop and Hypalon cover)	m	303.	periWS	$ 100	$ 30,300.00	
	Subtotal						$ 2,232,720.00
	SUBTOTAL CIVIL ITEMS					$ 2,822,270.00	
	Civil contingency allowance		$ 2,822,270.00		20%	$ 564,454.00	
	CIVIL INCL CONTINGENCY					$ 3,386,724.00	
	Civil Engineering		$ 3,386,724.00		10%	$ 338,672.40	
	Civil P&G		$ 3,386,724.00		15%	$ 508,008.60	
	TOTAL CIVIL						$ 4,233,405.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 4,233,405.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
3.1	**Rockfill**						
3.1.1	Zone 1A Cut to Fill	cu.m	0.	cutToFill1A	$ 5	$ -	
3.1.2	Zone 1A Borrow to Fill	cu.m	2,065.	cutToFill1A	$ 5	$ 10,325.00	
3.1.3	Zone 1B Cut to Fill	cu.m	7,332.	cutToFill1B	$ 5	$ 36,660.00	
3.1.4	Zone 1B Borrow to Fill	cu.m	0.	cutToFill1B	$ 5	$ -	
3.1.5	Zone 2A Cut to Fill	cu.m	0.	cutToFill2A	$ 20	$ -	
3.1.6	Zone 2A Borrow to Fill	cu.m	7,190.	cutToFill2A	$ 20	$ 143,800.00	
3.1.7	Zone 2B Cut to Fill	cu.m	0.	cutToFill2B	$ 20	$ -	
3.1.8	Zone 2B Borrow to Fill	cu.m	30,480.	cutToFill2B	$ 20	$ 609,600.00	
3.1.9	Zone 3A Cut to Fill	cu.m	88,242.	cutToFill3A	$ 10	$ 882,420.00	
3.1.10	Zone 3A Borrow to Fill	cu.m	127,209.	cutToFill3A	$ 10	$ 1,272,090.00	
3.1.11	Zone 3B Cut to Fill	cu.m	144,084.	cutToFill3B	$ 10	$ 1,440,840.00	
3.1.12	Zone 3B Borrow to Fill	cu.m	0.	cutToFill3B	$ 10	$ -	
	Subtotal						$ 4,395,735.00
3.2	**Concrete Face**						
3.2.1	Concrete (250mm)	cu.m	3,194.62	massConc	$ 350	$ 1,118,116.13	
3.2.2	Formwork (15m slipformed panel areas)	sq.m	12,778.47	formSlip	$ 100	$ 1,277,847.00	
3.2.3	Steel	t	638.92	reoSteel	$ 3,200	$ 2,044,555.20	
3.2.4	Concrete underfill (100mm)	cu.m	1,277.85	massConc	$ 350	$ 447,246.45	
3.2.5	Vertical joint waterstop	m	896.	watStop	$ 50	$ 44,800.00	
	Subtotal						$ 4,932,564.78
3.3	**Crest**						
3.3.1	Parapet Wall						
3.3.1.1	Concrete	cu.m	377.	massConc	$ 350	$ 131,950.00	
3.3.1.2	Formwork	sq.m	2,175.	formStr	$ 150	$ 326,250.00	
3.3.1.3	Steel	t	75.4	reoSteel	$ 3,200	$ 241,280.00	
3.3.2	Road Aggregate (300mm)	cu.m	315.	roadAgg	$ 60	$ 18,900.00	
	Subtotal						$ 718,380.00
3.4	**Instrumentation**						
3.4.1	Flow monitoring equipment	no	6.	flowMon	$ 1,500	$ 9,000.00	
3.4.2	Deformation Markers (U/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
3.4.3	Deformation Markers (D/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
	Subtotal						$ 49,000.00
3.5	**Electrical and Mechanical**						
3.5.1	Description						
3.5.1.1	Description				$ -	$ -	
	Subtotal						$ -
	SUBTOTAL CIVIL ITEMS					$ 10,095,679.78	
	Civil contingency allowance		$ 10,095,679.78		20%	$ 2,019,135.96	
	CIVIL INCL CONTINGENCY					$ 12,114,815.73	
	Civil Engineering		$ 12,114,815.73		10%	$ 1,211,481.57	
	Civil P&G		$ 12,114,815.73		15%	$ 1,817,222.36	
	TOTAL CIVIL						$ 15,143,519.66
	SUBTOTAL TOTAL E&M ITEMS					$ -	
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 15,143,519.66**

LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES **Ver 01**
ITEM 4 Spillway **9-Nov-09**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
4.1	**Bulk Earthworks**						
4.1.1	Cut to Waste - Soil	cu.m	105,223.	cutToWasteSoil	$ 4	$ 420,892.00	
4.1.2	Cut to Waste - Rock	cu.m	122,289.	cutToWasteRock	$ 8	$ 978,312.00	
4.1.3	Slope protection and reinforcement	sq.m	1,000.	slopeProtect	$ 100	$ 100,000.00	
	Subtotal						$ 1,499,204.00
4.2	**Fuse Embankment**						
4.2.1	Armoring to U/S Face (0.5m thk)	cu.m	199.	armourHeavy	$ 60	$ 11,940.00	
4.2.2	Filter Layer (0.5m thk)	cu.m	199.	cutToFill2A	$ 20	$ 3,980.00	
4.2.3	Inclined Geotextile	sq.m	398.	geoTex	$ 10	$ 3,980.00	
4.2.4	Bulk Fill	cu.m	3,493.	cutToFill2A	$ 20	$ 69,860.00	
4.2.5	Concrete Slab						
4.2.5.1	Concrete	cu.m	768.	massConc	$ 350	$ 268,800.00	
4.2.5.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.5.3	Steel (0.1t/cu.m)	t	76.8	reoSteel	$ 3,200	$ 245,760.00	
4.2.6	Approach Slab						
4.2.6.1	Concrete	cu.m	216.	massConc	$ 350	$ 75,600.00	
4.2.6.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.6.3	Steel (0.1t/cu.m)	t	21.6	reoSteel	$ 3,200	$ 69,120.00	
	Subtotal						$ 749,040.00
4.3	**Primary Spillway Chute**						
4.3.1	Ogee Weir						
4.3.1.1	Concrete	cu.m	278.75	massConc	$ 350	$ 97,562.50	
4.3.1.2	Formwork	sq.m	178.4	formCur	$ 350	$ 62,440.00	
4.3.1.3	Steel (0.05t/cu.m)	t	13.9	reoSteel	$ 3,200	$ 44,480.00	
4.3.2	Approach Slab						
4.3.2.1	Concrete	cu.m	164.5	massConc	$ 350	$ 57,575.00	
4.3.2.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.3.2.3	Steel (0.1t/cu.m)	t	16.45	reoSteel	$ 3,200	$ 52,640.00	
4.3.2.4	Foundation preparation	sq.m	658.	fndCleanConc	$ 10	$ 6,580.00	
4.3.3	Chute Floor						
4.3.3.1	Concrete	cu.m	499.9	massConc	$ 350	$ 174,965.00	
4.3.3.2	Formwork (Slipformed)	sq.m	758.5	formSlip	$ 100	$ 75,850.00	
4.3.3.3	Steel (0.1t/cu.m)	t	49.9	reoSteel	$ 3,200	$ 159,680.00	
4.3.3.4	Waterstop joints	m	400.	watStop	$ 50	$ 20,000.00	
4.3.3.5	Foundation preparation	sq.m	760.	fndCleanConc	$ 10	$ 7,600.00	
4.3.4	Chute Walls						
4.3.4.1	Concrete	cu.m	382.6	massConc	$ 350	$ 133,910.00	
4.3.4.2	Formwork	sq.m	1,275.33	formStr	$ 150	$ 191,300.00	
4.3.4.3	Steel (0.2t/cu.m)	t	76.52	reoSteel	$ 3,200	$ 244,864.00	
4.3.5	Wall Backfill	cu.m	1,087.	cutToFill2A	$ 20	$ 21,740.00	
4.3.6	Chute Under Drainage (drains at 10m centers)						
4.3.6.1	Filter Material	cu.m	11.21	roadAgg	$ 60	$ 672.60	
4.3.6.2	Pipe (100 dia HDPE perforated)	m	170.	pipeHDPE100	$ 20	$ 3,400.00	
4.3.9	Chute Slab Anchors (1 Per 10 sq.m)	no	167	anchorDowell	$ 200	$ 33,326.67	
	Subtotal						$ 1,388,585.77
4.4	**Flip Bucket**						
4.4.1	Bucket						
4.4.1.1	Concrete	cu.m	681.5	massConc	$ 350	$ 238,525.00	
4.4.1.2	Formwork and curved surface formation	sq.m	438.16	formCur	$ 350	$ 153,356.00	
4.4.1.3	Steel (0.1t/cu.m)	t	68.15	reoSteel	$ 3,200	$ 218,080.00	
4.4.2	Bucket Rock Anchors (6 assumed)	no	6.	anchorDowell	$ 200	$ 1,200.00	
4.4.3	Low flow channel lining						
4.4.3.1	Concrete	cu.m	375.	massConc	$ 350	$ 131,250.00	
4.4.3.2	Formwork (Slipformed)	sq.m	0.	formStr	$ 150	$ -	
4.4.3.3	Steel (0.05t/cu.m)	t	18.75	reoSteel	$ 3,200	$ 60,000.00	
	Subtotal						$ 802,411.00
4.5	**Plungepool**						
4.5.1	Excavation	cu.m	2600	cutToWasteRock	$ 8	$ 20,800.00	
4.5.2	Rock Armour	cu.m	1,500.	armRockHeavy	$ 100	$ 150,000.00	
	Subtotal						$ 170,800.00
	SUBTOTAL CIVIL ITEMS					$ 4,610,040.77	
	Civil contingency allowance		$ 4,610,040.77		20%	$ 922,008.15	
	CIVIL INCL CONTINGENCY					$ 5,532,048.92	
	Civil Engineering		$ 5,532,048.92		10%	$ 553,204.89	
	Civil P&G		$ 5,532,048.92		15%	$ 829,807.34	
	TOTAL CIVIL						$ 6,915,061.15
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 6,915,061.15**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
5.1	**Inclined Inlets**						
5.1.1	Conduits on U/S Face						
5.1.1.1	Concrete	cu.m	139.94	massConc	$ 350	$ 48,979.00	
5.1.1.2	Formwork (may be precast)	sq.m	921.	formStr	$ 150	$ 138,150.00	
5.1.1.3	Steel (0.2t/cu.m)	t	27.99	reoSteel	$ 3,200	$ 89,561.60	
5.1.3	Rails embedded in upstream face	t	1.66	strSteel	$ 5,000	$ 8,300.00	
	Subtotal						$ 284,990.60
5.2	**Outlet Conduits**						
5.2.1	Box Inlet Plug						
5.2.1.1	Concrete	cu.m	105.	massConc	$ 350	$ 36,750.00	
5.2.1.2	Formwork (may be precast)	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.1.3	Steel (0.05t/cu.m)	t	5.25	reoSteel	$ 3,200	$ 16,800.00	
5.2.2	Gate Structure						
5.2.2.1	Concrete	cu.m	37.5	massConc	$ 350	$ 13,125.00	
5.2.2.2	Formwork	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.2.3	Steel (0.1t/cu.m)	t	3.75	reoSteel	$ 3,200	$ 12,000.00	
5.2.3	Overhead Beam	t	8.46	strSteel	$ 5,000	$ 42,282.00	
	Subtotal						$ 143,457.00
5.3	**Outlet Structure**						
5.3.1	Outlet Stoplog Concrete Structure						
5.3.1.1	Concrete	cu.m	12.5	massConc	$ 350	$ 4,375.00	
5.3.1.2	Formwork	sq.m	25.	formStr	$ 150	$ 3,750.00	
5.3.1.3	Steel	t	1.25	reoSteel	$ 3,200	$ 4,000.00	
5.3.2	Outlet Wingwall Structure						
5.3.2.1	Concrete	cu.m	143.	massConc	$ 350	$ 50,050.00	
5.3.2.2	Formwork	sq.m	186.	formStr	$ 150	$ 27,900.00	
5.3.2.3	Steel (0.2t/cu.m)	t	28.6	reoSteel	$ 3,200	$ 91,520.00	
	Subtotal						$ 181,595.00
5.4	**Gates, Screens and Related**						
5.4.1	Removable Bellmouth Inlet and Screen	t	5.997	steelHydGen	$ 12,000	$ 71,968.80	
5.4.2	Removable Stoplogs for intake structure	t	3.25	steelStopLog	$ 10,000	$ 32,500.00	
5.4.3	Stoplog and Screen Derrick and Winch for intake structure	t	5.	steelHydGen	$ 12,000	$ 60,000.00	
5.4.4	Radial Gates 1m x 1m x 2 gates for irrigation outlet	t	9.29	steelHydGen	$ 12,000	$ 111,480.00	
5.4.5	Tailrace area outlet Stoplogs	t	6.78	steelStopLog	$ 10,000	$ 67,820.00	
	Subtotal						$ 343,768.80
	SUBTOTAL CIVIL ITEMS					$ 610,042.60	
	Civil contingency allowance		$ 610,042.60		20%	$ 122,008.52	
	CIVIL INCL CONTINGENCY					$ 732,051.12	
	Civil Engineering		$ 732,051.12		10%	$ 73,205.11	
	Civil P&G		$ 732,051.12		15%	$ 109,807.67	
	TOTAL CIVIL						$ 915,063.90
	SUBTOTAL TOTAL E&M ITEMS					$ 343,768.80	
	E&M contingency allowance		$ 343,768.80		10%	$ 34,376.88	
	E&M INCL CONTINGENCY					$ 378,145.68	
	E&M Engineering		$ 378,145.68		5%	$ 18,907.28	
	E&M P&G		$ 378,145.68		10%	$ 37,814.57	
	TOTAL E&M						$ 434,867.53
	TOTAL						**$ 1,349,931.43**

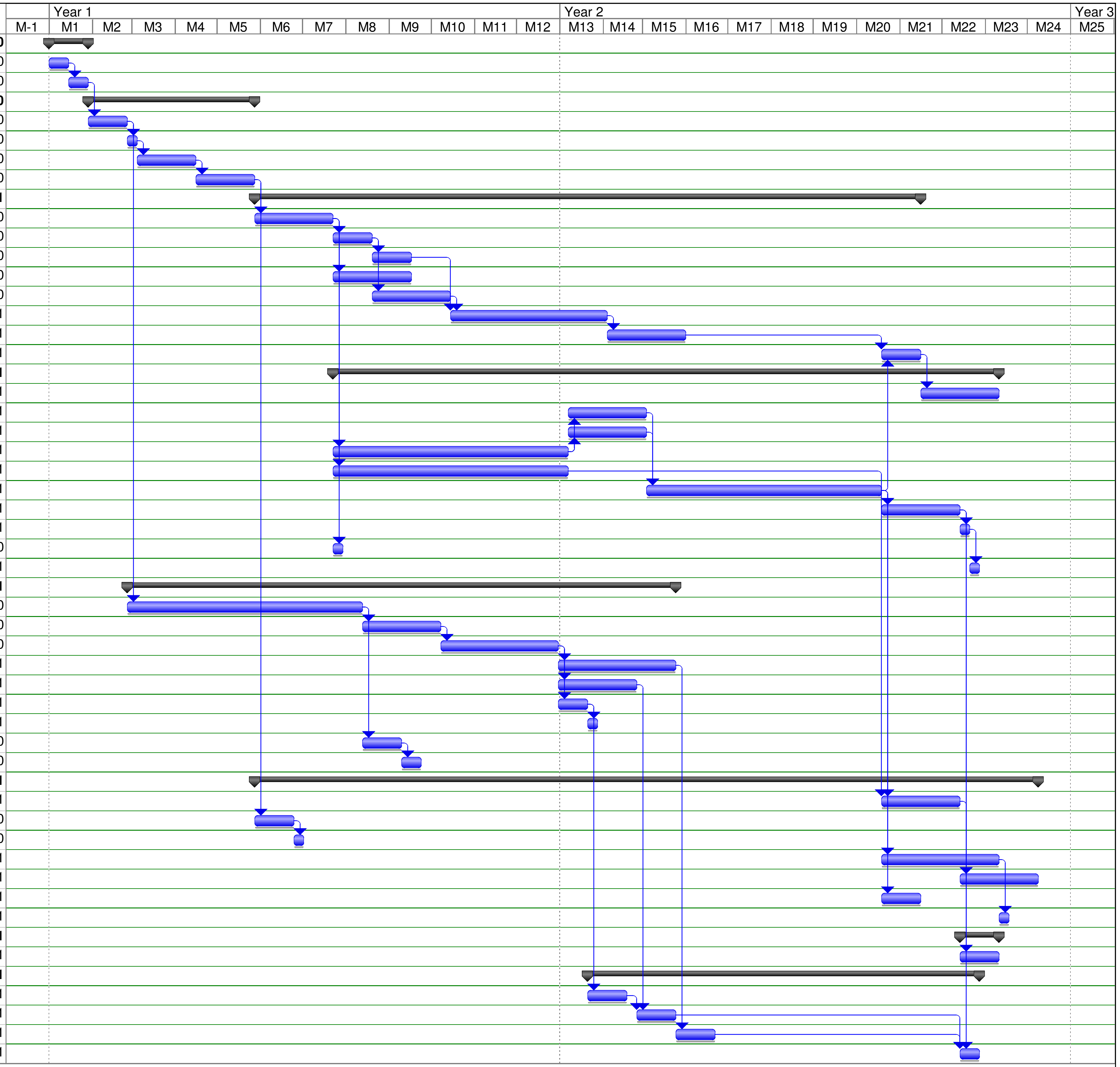
No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
6.1	**Fishpass**						
6.1.1	Pump (2l/s)	No	1.	puml2LS	$ 2,000	$ 2,000.00	
6.1.2	Grouted rock channel	cu.m	250.	groutRock	$ 150	$ 37,500.00	
6.1.3	750 mm Culvert Crossing	m	10.	culvert750	$ 750	$ 7,500.00	
6.1.4	U/S Face Sluice	m	15.	culvert750	$ 750	$ 11,250.00	
	Subtotal						$ 58,250.00
	SUBTOTAL CIVIL ITEMS					$ 58,250.00	
	Civil contingency allowance		$ 58,250.00		20%	$ 11,650.00	
	CIVIL INCL CONTINGENCY					$ 69,900.00	
	Civil Engineering		$ 69,900.00		10%	$ 6,990.00	
	Civil P&G		$ 69,900.00		15%	$ 10,485.00	
	TOTAL CIVIL						$ 87,375.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 87,375.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
7.1	**Power Station Access Road**						
7.1.1	Surfacing Aggregate (250 mm)	cu.m	742.5	roadAgg	$ 60	$ 44,550.00	
	Subtotal						$ 44,550.00
7.2	**Auxillary Spillway Access**						
7.2.1	Surfacing Aggregate (250 mm)	cu.m	132.	roadAgg	$ 60	$ 7,920.00	
	Subtotal						$ 7,920.00
7.3	**Lee Valley Road Diversion**						
7.3.1	Surfacing Aggregate (250 mm)	cu.m	767.25	roadAgg	$ 60	$ 46,035.00	
	Subtotal						$ 46,035.00
7.4	**Spillway Bridging**						
7.4.1	Auxillary Spillway Bridge (10 m Spans)	sq.m	146.	bridgePlan	$ 1,250	$ 182,500.00	
7.4.2	Abutment Retaining	sq.m	251.	retainWall	$ 300	$ 75,300.00	
7.4.3	Primary Spillway Bridge (10 m Spans)	sq.m	160.	bridgePlan	$ 1,250	$ 200,000.00	
7.4.4	Chute Bucket Bridge (10 m Span)	sq.m	106.	bridgePlan	$ 1,250	$ 132,500.00	
7.4.5	Abutment Retaining	sq.m	209.	retainWall	$ 300	$ 62,700.00	
	Subtotal						$ 653,000.00
	SUBTOTAL CIVIL ITEMS					$ 751,505.00	
	Civil contingency allowance		$ 751,505.00		20%	$ 150,301.00	
	CIVIL INCL CONTINGENCY					$ 901,806.00	
	Civil Engineering		$ 901,806.00		10%	$ 90,180.60	
	Civil P&G		$ 901,806.00		15%	$ 135,270.90	
	TOTAL CIVIL						$ 1,127,257.50
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 1,127,257.50**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
8.1	**All Zone 3A Rockfill from Borrow**						
8.1.1	Additional spillway rock cut to waste	cu.m	88,242.	cutToWasteRock	$ 8	$ 705,936.00	
	Subtotal						$ 705,936.00
8.2	**Additional Drainage at Foundation level**						
8.2.1	Material Replaced (from Zone 3A & 3B)	cu.m	-11,000.	cutToFill3A	$ 10	-$ 110,000.00	
8.2.2	Filter Drain Material	cu.m	11,000.	cutToFill2A	$ 20	$ 220,000.00	
	Subtotal						$ 110,000.00
8.3	**Additional Defect Treatment/Dental Work**						
8.3.1	Additional defects @ 1 per 50 sq.m	no	90.	defectTreat	$ 4,000	$ 360,000.00	
	Subtotal						$ 360,000.00
1.4	**Spillway Cut Slopes of 1.5H:1V**						
8.4.1	Revised Rockfill						
8.4.1.1	*Zone 1A Cut to Fill*	cu.m	0.	cutToFill1A	$ 5	$ -	
8.4.1.2	*Zone 1A Borrow to Fill*	cu.m	2,065.	cutToFill1A	$ 5	$ 10,325.00	
8.4.1.3	*Zone 1B Cut to Fill*	cu.m	7,332.	cutToFill1B	$ 5	$ 36,660.00	
8.4.1.4	*Zone 1B Borrow to Fill*	cu.m	0.	cutToFill1B	$ 5	$ -	
8.4.1.5	*Zone 2A Cut to Fill*	cu.m	0.	cutToFill2A	$ 20	$ -	
8.4.1.6	*Zone 2A Borrow to Fill*	cu.m	7,190.	cutToFill2A	$ 20	$ 143,800.00	
8.4.1.7	*Zone 2B Cut to Fill*	cu.m	0.	cutToFill2B	$ 20	$ -	
8.4.1.8	*Zone 2B Borrow to Fill*	cu.m	32,575.	cutToFill2B	$ 20	$ 651,500.00	
8.4.1.9	*Zone 3A Cut to Fill*	cu.m	119,776.	cutToFill3A	$ 10	$ 1,197,760.00	
8.4.1.10	*Zone 3A Borrow to Fill*	cu.m	95,675.	cutToFill3A	$ 10	$ 956,750.00	
8.4.1.11	*Zone 3B Cut to Fill*	cu.m	144,084.	cutToFill3B	$ 10	$ 1,440,840.00	
8.4.1.12	*Zone 3B Borrow to Fill*	cu.m	0.	cutToFill3B	$ 10	$ -	
8.4.2	Revised Spillway Cut to Waste						
8.4.1.1	*Cut to Waste - Soil*	cu.m	204,016.	cutToWasteSoil	$ 4	$ 816,064.00	
8.4.1.2	*Cut to Waste - Rock*	cu.m	144,600.	cutToWasteRock	$ 8	$ 1,156,800.00	
8.4.3	Base Rockfill Cost					-$ 4,395,735.00	
8.4.4	Base Spillway Cut to Waste Cost					-$ 1,399,204.00	
	Subtotal						$ 615,560.00
	SUBTOTAL CIVIL ITEMS					$ 1,791,496.00	
	Civil contingency allowance		$ 1,791,496.00		20%	$ 358,299.20	
	CIVIL INCL CONTINGENCY					$ 2,149,795.20	
	Civil Engineering		$ 2,149,795.20		10%	$ 214,979.52	
	Civil P&G		$ 2,149,795.20		15%	$ 322,469.28	
	TOTAL CIVIL						$ 2,687,244.00
	SUBTOTAL TOTAL E&M ITEMS					$ -	
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 2,687,244.00**

Appendix D: Outline Construction Programme

ID		Task Name	Duration	Start	Finish
1		**Preliminary and General**	**20 days**	**Fri 1/01/10**	**Thu 28/01/10**
2		Mobilisation	2 wks	Fri 1/01/10	Thu 14/01/10
3		Site offices and utilities	2 wks	Fri 15/01/10	Thu 28/01/10
4		**Site Preparation and Diversion Works**	**85 days**	**Fri 29/01/10**	**Thu 27/05/10**
5		Strip damsite	4 wks	Fri 29/01/10	Thu 25/02/10
6		Excavation for diversion	1 wk	Fri 26/02/10	Thu 4/03/10
7		Construct diversion culverts US section	6 wks	Fri 5/03/10	Thu 15/04/10
8		Construct diversion culverts DS section	6 wks	Fri 16/04/10	Thu 27/05/10
9		**Plinth**	**340 days**	**Fri 28/05/10**	**Thu 15/09/11**
10		Plinth excavation	2 mons	Fri 28/05/10	Thu 22/07/10
11		Foundation cleanup	1 mon	Fri 23/07/10	Thu 19/08/10
12		Defect treatment	1 mon	Fri 20/08/10	Thu 16/09/10
13		Slope reinforcement	2 mons	Fri 23/07/10	Thu 16/09/10
14		Drill and grout anchors	2 mons	Fri 20/08/10	Thu 14/10/10
15		Place plinth	4 mons	Fri 15/10/10	Thu 3/02/11
16		Drilling and grouting	2 mons	Fri 4/02/11	Thu 31/03/11
17		Perimetric joint installation	1 mon	Fri 19/08/11	Thu 15/09/11
18		**Embankment**	**340 days**	**Fri 23/07/10**	**Thu 10/11/11**
19		Place zone 1A and 1B	2 mons	Fri 16/09/11	Thu 10/11/11
20		Place zone 2A	2 mons	Fri 7/01/11	Thu 3/03/11
21		Place zone 2B	2 mons	Fri 7/01/11	Thu 3/03/11
22		Place zone 3A	6 mons	Fri 23/07/10	Thu 6/01/11
23		Place zone 3B	6 mons	Fri 23/07/10	Thu 6/01/11
24		Place concrete upstream face	6 mons	Fri 4/03/11	Thu 18/08/11
25		Place parapet wall	2 mons	Fri 19/08/11	Thu 13/10/11
26		Place road aggregate	1 wk	Fri 14/10/11	Thu 20/10/11
27		Install drainage collection	1 wk	Fri 23/07/10	Thu 29/07/10
28		Install deformation markers	1 wk	Fri 21/10/11	Thu 27/10/11
29		**Spillway**	**280 days**	**Fri 26/02/10**	**Thu 24/03/11**
30		Excavate spillway profile	6 mons	Fri 26/02/10	Thu 12/08/10
31		Construct flip bucket	2 mons	Fri 13/08/10	Thu 7/10/10
32		Place chute floor	3 mons	Fri 8/10/10	Thu 30/12/10
33		Place chute walls	3 mons	Fri 31/12/10	Thu 24/03/11
34		Place ogee and approach slab	2 mons	Fri 31/12/10	Thu 24/02/11
35		Place auxiliary concrete slab	3 wks	Fri 31/12/10	Thu 20/01/11
36		Place fuseable embankment	1 wk	Fri 21/01/11	Thu 27/01/11
37		Excavate plunge pool	4 wks	Fri 13/08/10	Thu 9/09/10
38		Place plunge pool armour	2 wks	Fri 10/09/10	Thu 23/09/10
39		**Reservoir Outlet**	**400 days**	**Fri 28/05/10**	**Thu 8/12/11**
40		Form outlet works in conduit	2 mons	Fri 19/08/11	Thu 13/10/11
41		Form outlet wingwalls	4 wks	Fri 28/05/10	Thu 24/06/10
42		Install outlet stop logs	1 wk	Fri 25/06/10	Thu 1/07/10
43		Place inclined concrete intakes	3 mons	Fri 19/08/11	Thu 10/11/11
44		Install radial gates	2 mons	Fri 14/10/11	Thu 8/12/11
45		Install surface rails	1 mon	Fri 19/08/11	Thu 15/09/11
46		Install intake screens	1 wk	Fri 11/11/11	Thu 17/11/11
47		**Fish Pass**	**20 days**	**Fri 14/10/11**	**Thu 10/11/11**
48		Form fish pass channel	4 wks	Fri 14/10/11	Thu 10/11/11
49		**Access**	**200 days**	**Fri 21/01/11**	**Thu 27/10/11**
50		Construct bridge over auxiliary spillway	4 wks	Fri 21/01/11	Thu 17/02/11
51		Construct upper bridge over primary spillway	4 wks	Fri 25/02/11	Thu 24/03/11
52		Construct lower bridge over primary spillway	4 wks	Fri 25/03/11	Thu 21/04/11
53		Place access road surfacing	2 wks	Fri 14/10/11	Thu 27/10/11



Project: LeeValleyDam-Construction.r
Date: Wed 18/11/09

Task		Progress		Summary		External Tasks		Deadline
Split		Milestone	◆	Project Summary		External Milestone	◇	

Appendix E: Dambreak Report

REPORT

WAIMEA WATER AUGMENTATION COMMITTEE

Lee River Dam: Dam Break Analysis and Hazard Assessment

Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE

Report prepared by:

TONKIN & TAYLOR LTD

Distribution:

WAIMEA WATER AUGMENTATION COMMITTEE	4 copies
TONKIN & TAYLOR LTD (FILE)	1 copy

December 2009

T&T Ref: 24727.304

Table of contents

1 Introduction

1.1 Background

In 2007 Tonkin & Taylor Ltd and its sub-consultants completed a Phase 1 pre-feasibility evaluation of a number of options to provide water storage for long-term irrigation and community supplies in the Waimea Basin, Tasman District. The evaluation was undertaken on behalf of the Waimea Water Augmentation Committee (WWAC). The overall purpose of the study was to identify and develop a water augmentation scheme to capture excess water for storage. The stored water would be released back into the Waimea River system during periods of high water demand and/or low natural water flows to augment those supplies, either directly or via recharging of the groundwater system.

The outcome of that Phase 1 study was to focus feasibility investigations on a water storage dam and reservoir site located in the upper Lee River catchment, a tributary of the Waimea River.

In 2007 WWAC initiated Phase 2 of the study, to take the Lee investigation programme to a feasibility level.

A potential dam site on the Lee River at a site approximately 300 metres upstream of the confluence of Anslow Creek and the Lee River was proposed. The required storage capacity of the reservoir has been determined to be approximately 13 million m^3, with a normal top water level to RL 197 m. The reservoir would extend approximately 4 km upstream from the dam, and cover an area of approximately 65 hectares (based on normal top water level).

Figure 1-1 shows the location of the proposed dam, and the indicative reservoir extent.

The Lee River joins the Wairoa River nearly 12.5 km downstream of the dam site. The Wairoa River emerges from the Wairoa Gorge near Leedale and Max's Bush and passes under the State Highway 6 (SH6) bridge to the east of Brightwater urban township. Approximately 3.8 km downstream of the SH6 bridge the Wai-iti River joins the Wairoa River. Downstream of this point, the river becomes known as the Waimea River, which flows into the sea some 4.5km km further downstream.

This report sets out the results of a dam break analysis and hazard assessment based on a dam on the Lee River at site change 12,340 m[1].

1.2 Dam break analysis

Dam break analyses are undertaken within the dam industry primarily to assess downstream hazard potential, which in turn guides the setting of standards to adopt for dam design, construction and operation. The analyses are hypothetical and entirely divorced from the chances of a dam failure ever occurring.

Certain information generated from a dam break study, such as a map delineating the potential extent of inundation from a dambreak, may be used in an Emergency Action Plan for the dam and made available to the local Civil Defence office. The predicted time for the dambreak flood wave to reach specific downstream locations provides a helpful indication of the available warning times, and may also be incorporated in the Emergency

[1] River referencing system

Action and/or Civil Defence Plans. Section 7 provides more information on the Dam Break Avoidance and Mitigation.

The report is structured as follows:

- Introduction of the concept of establishing the Potential Impact Category (PIC) of a dam
- Sets out the scope of the dam break analysis and hazard assessment undertaken for the Lee Dam
- Description of the assessment methodology and assumptions used in making the assessment
- Identification and discussion of the model results
- Description of the hazard as a consequence of a dam failure
- Recommendation of potential impact category
- Discussion of dam break mitigation measures.

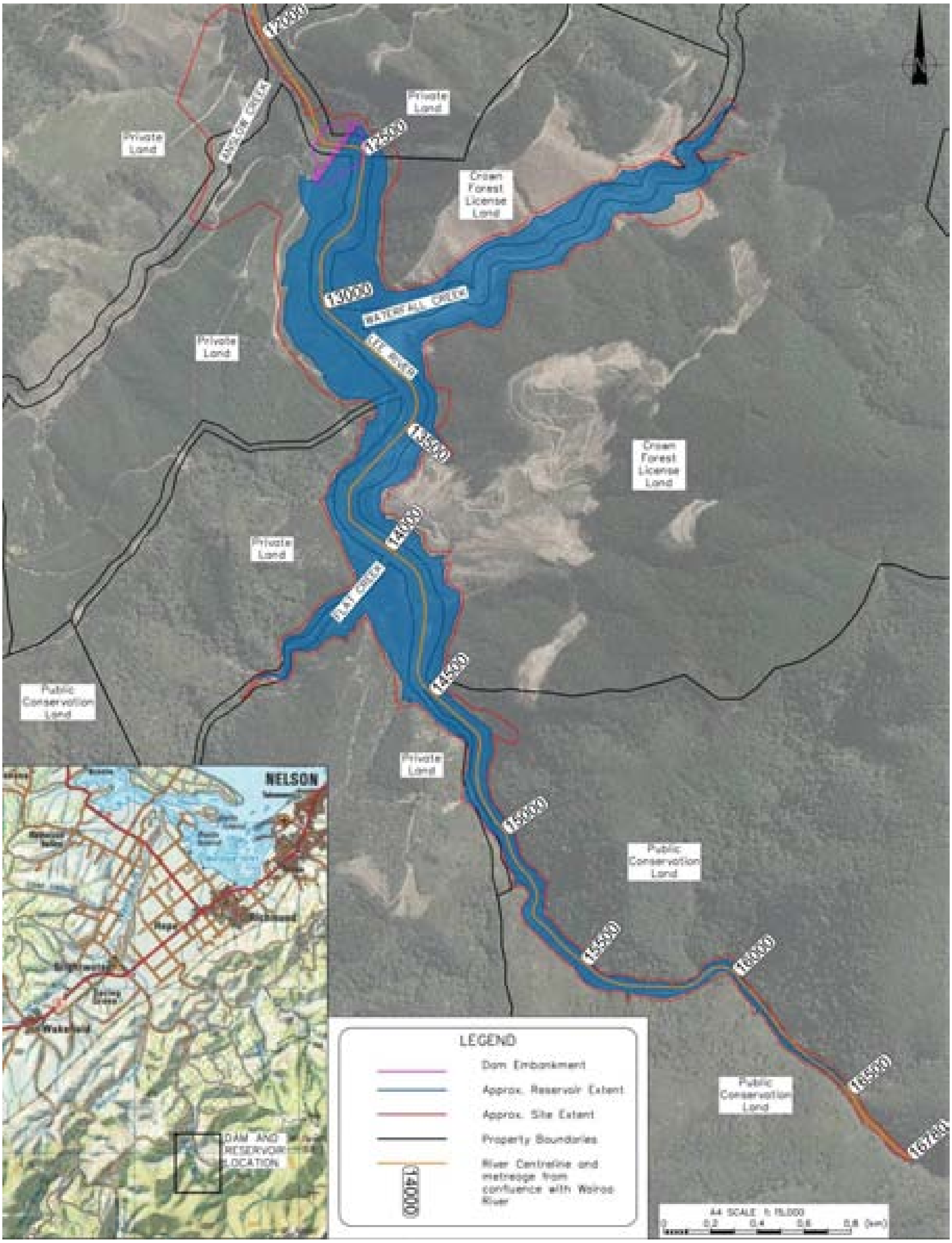


Figure 1-1 Site location plan

2 Potential Impact Classification System

The Building (Dam Safety) Regulations 2008 adopt a potential impact classification system to determine the appropriate design standards for a dam (for earthquake loading and safe flood passage) and the level of rigour that needs to be applied to site investigations, construction, commissioning and on-going maintenance and surveillance.

The consequences of failure, specifically the downstream harm and damage potential, are the main determinant for assessing the potential impact classification. Table 2-1 below shows the definitions of Potential Impact Category (PIC) adopted by the regulations.

Table 2-1 Determination of dam classification (Building (Dam Safety) Regulations 2008)

Assessed damage level	Population at risk (PAR)			
	0	**1-10**	**11-100**	**100+**
Catastrophic	High PIC	High PIC	High PIC	High PIC
Major	Medium PIC (see note 4)	Med/High PIC (see note 4)	High PIC	High PIC
Moderate	Low PIC	Low/Med/High PIC (see notes 3 and 4)	Med/High PIC (see note 4)	Med/High PIC (see notes 2 and 4)
Minimal	Low PIC	Low/Med/High PIC (see notes 1, 3 and 4)	Low/Med/High PIC (see notes 1, 3 and 4)	Low/Med/High PIC (see notes 1, 3 and 4)

Notes:

1. With a PAR of 5 or more people, it is unlikely that the potential impact will be low
2. With a PAR of more than 100 people, it is unlikely that the potential impact will be medium
3. Use a medium classification if it is highly likely that a life will be lost
4. Use a high classification if it is highly likely that 2 or more lives will be lost

The population at risk (PAR) is defined as all those people who would be affected by flood depths in excess of 0.5 metres.

Interpretative details on the assessed damage level are provided in the Building (Dam Safety) Regulations 2008. Table 2-2 reproduces the interpretation of 'catastrophic', 'major', 'moderate' and 'minimal' damages from the regulations.

Table 2-2 Determination of assessed damage level

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

In relation to residential houses, "destroyed" means rendered uninhabitable.

Critical or major infrastructure includes:

1. Lifelines e.g. power supply, water supply, gas supply, transportation systems, wastewater treatment.
2. Emergency facilities e.g. hospitals, police, fire services.
3. Large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community.
4. The dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.

3 Scope of Assessment

It is usual practice in a dam break hazard assessment to consider the incremental damages for a "sunny day" failure and in some cases, a flood induced failure.

Incremental damages are those that occur directly as a result of the dam failure. For example if 10 houses would be flooded during a natural 0.01% Annual Exceedance Probability (AEP) (or 10,000 year return period) event and 15 houses were shown to be flooded after a dam failure during the 0.01% AEP event, then the incremental damage is the flooding of 5 additional houses.

In terms of incremental damages from dam failure, "sunny day" failures typically have greater potential consequences because all the damage is directly caused by the dam failure.

For this particular study only a sunny day failure scenario has been assessed. It was considered unnecessary to also model a flood-induced failure as a decision was taken in the feasibility design to provide sufficient spillway capacity at the dam to cope with the Probable Maximum Flood (the largest flood that could conceivably occur at that location) without relying on mechanically controlled spillway gates.

The dam break assessment essentially consists of three parts:

i. **Dam breach outflow:** estimating the breach geometry and breach development rate; and then the rate at which water will flow out of the dam in the event of a dam break,
ii. **Downstream floodwave:** modelling the downstream flow path and character of water flows from the breach in the dam to the sea, and mapping the potentially inundated area,
iii. **Categorisation of potential impact:** assessment of the likely damage caused by the inundation and the potential for loss of life and environmental damage.

Results from the dam break modelling have been used in the development of an outline Emergency Action Plan for the Lee River Dam (attached in Appendix 1). This Plan will be further developed and finalised during the detailed design phase of the project.

4 Dam Breach Outflow

4.1 Background

The discharge hydrograph from the Lee River Dam in the event of a dam break largely determines the downstream floodwave. The discharge hydrograph is determined from a combination of the dam break parameters (e.g. shape of the breach, breach progression time) and the reservoir characteristics (e.g. storage versus elevation curve, water level at the start of the breach).

4.2 Dam breach parameters

A potential breach location centred over the highest part of the dam has been considered. The highest part of the dam is the worst case position for the dam breach to occur since it corresponds with the greatest water depth.

Wahl (1988) provides a comprehensive summary of the formulae by various researchers for predicting the breach parameters for embankment dams. These parameters include ultimate breach width, breach side slopes and the breach progression (formation) time.

While there was a wide range of predicted breach widths, the majority of the formulae indicated that the fully developed breach would occupy the entire valley bottom (typically less than about 50 m at river level). A dam break would likely result in erosion to bedrock at one of the dam's abutments. For this assessment, it is assumed that the right side of the fully formed breach would correspond with the exposed right abutment of the dam, which has an approximate slope of 1.3H : 1V (which is considerably steeper than the left abutment), while the left side of the breach would be a free-standing slope with a gradient 1H:1V. This gives a breach width at mid-height of the dam of about 110 m, which is close to the median (120 m) and average (130 m) of the predicted breach widths. Figure 4-1 shows the assumed location and shape of the fully formed breach superimposed on an elevation of the embankment dam.

With regard to breach formation time, the approach proposed by MacDonald et al. (1984), which used empirical data to relate the failure duration to the amount of material removed, indicated a period of 1.4 hours for the full breach to form. However, other empirical formulae indicate breach formation times as short as 0.4 hours. Because of the uncertainty in the estimate of the breach formation time, three scenarios have been modelled. These correspond with the "best" estimate plus an upper (0.5 hours) bound and a lower bound (1.5 hours) case. Note that a shorter breach formation time is more severe in terms of downstream hazard because there will be a greater depth (and hence higher discharge) of water over the fully formed breach compared with a longer failure.

The modelled scenarios are shown in Table 4-1.

Table 4-1 Dam breach parameters

	Water Level	Bottom Elevation	Left Slope	Right Slope	Bottom Width	Breach formation time
Scenario A	197 m RL	150 m RL	1(H) : 1(V)	1.3(H) :1(V)	50 m	0.5 hour
Scenario B	197 m RL	150 m RL	1(H) : 1(V)	1.3(H) : 1(V)	50 m	0.9 hour
Scenario C	197 m RL	150 m RL	1(H) : 1(V)	1.3(H) : 1(V)	50 m	1.5 hours

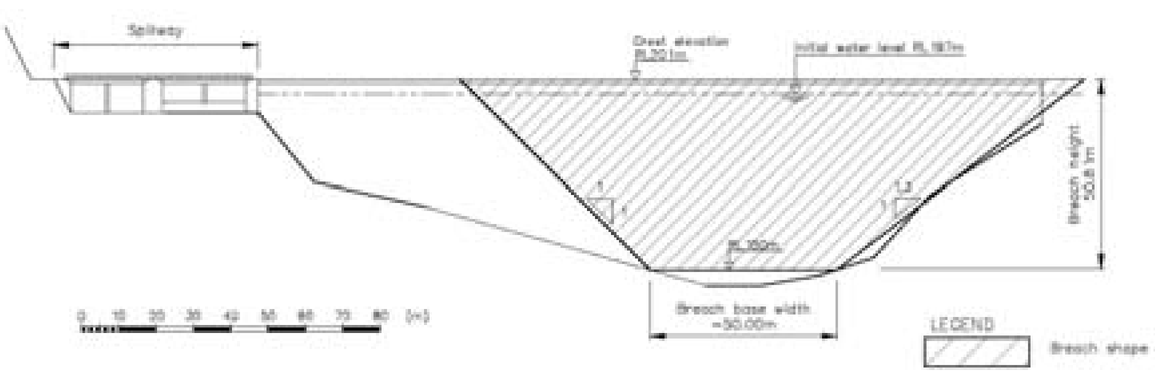


Figure 4-1 Assumed breach location and geometry of fully formed breach

4.3 Dam breach hydrograph

The discharge hydrograph for the different breach scenarios was determined using the HEC-HMS modelling package. HEC-HMS is produced by the U.S Army Corps of Engineers (USACE) Hydrologic Engineering Centre (HEC).

Within HEC-HMS, the dam break feature was represented using the breach parameters from Table 4-1 and the storage versus elevation relationship shown in Figure 4-2. The reservoir level was assumed to be at the normal top water level (197 m RL) at the start of the dam break.

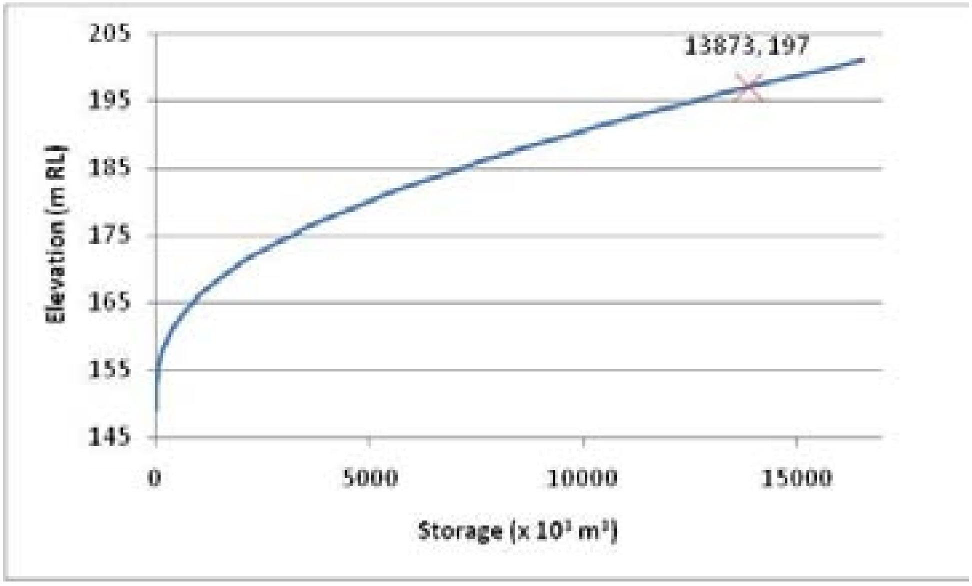


Figure 4-2 Storage versus elevation relationship for the proposed reservoir

4.3.1 Results

Figure 4-3 shows the results from the HEC-HMS model for the three different breach scenarios in Table 4-1.

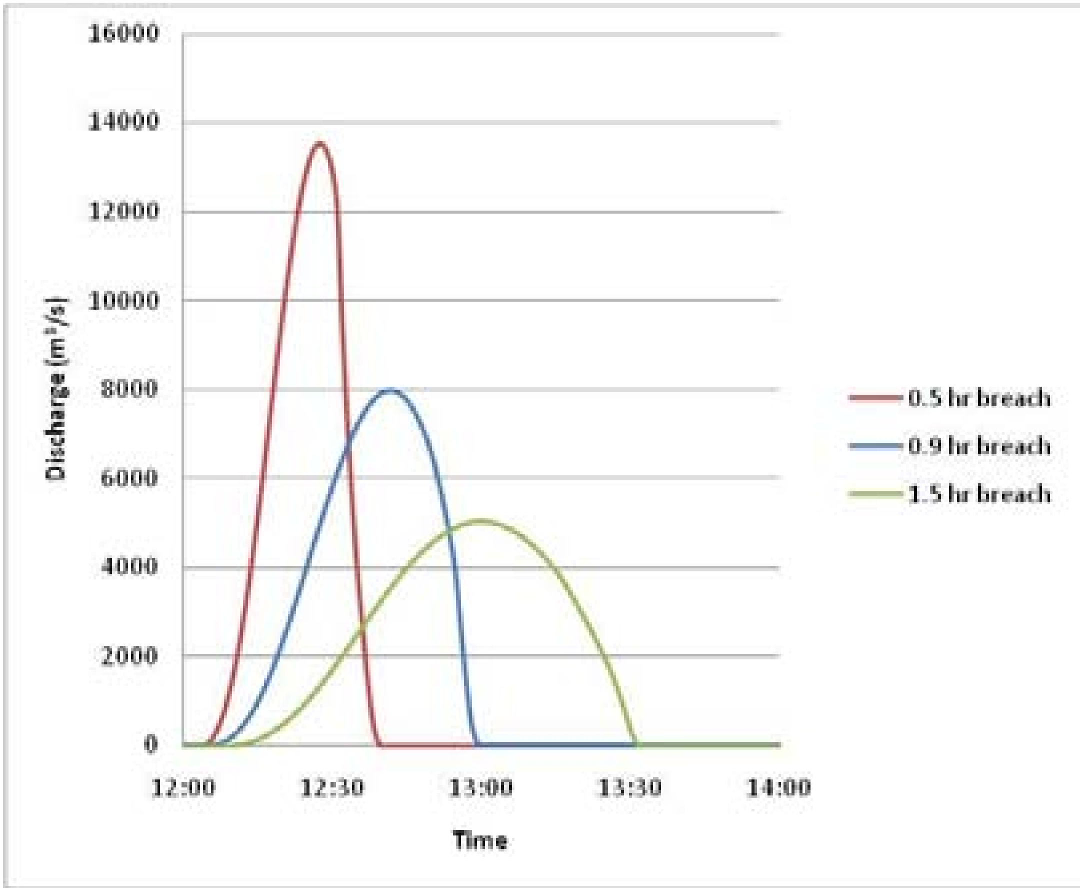


Figure 4-3 Dam breach hydrograph for three breach formation times

As an independent check of the modelled peak flows, a comparison between the HEC-HMS predicted flows and a number of peak outflow relationships published by the Dam Safety Office (1998) have been made. The HEC-HMS results have been compared with empirical relationships and observed data that relate peak flow to:

1. Height of dam (see Figure 4-4)
2. Storage volume of the dam (see Figure 4-5)
3. Storage volume and height of dam (see Figure 4-6).

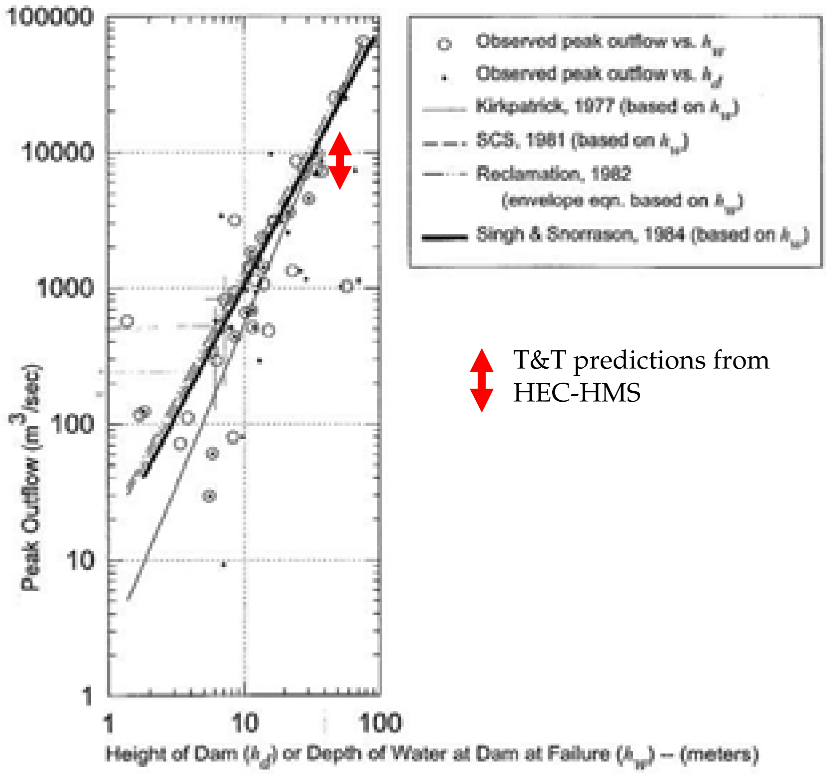


Figure 4-4 Peak discharge versus height parameters

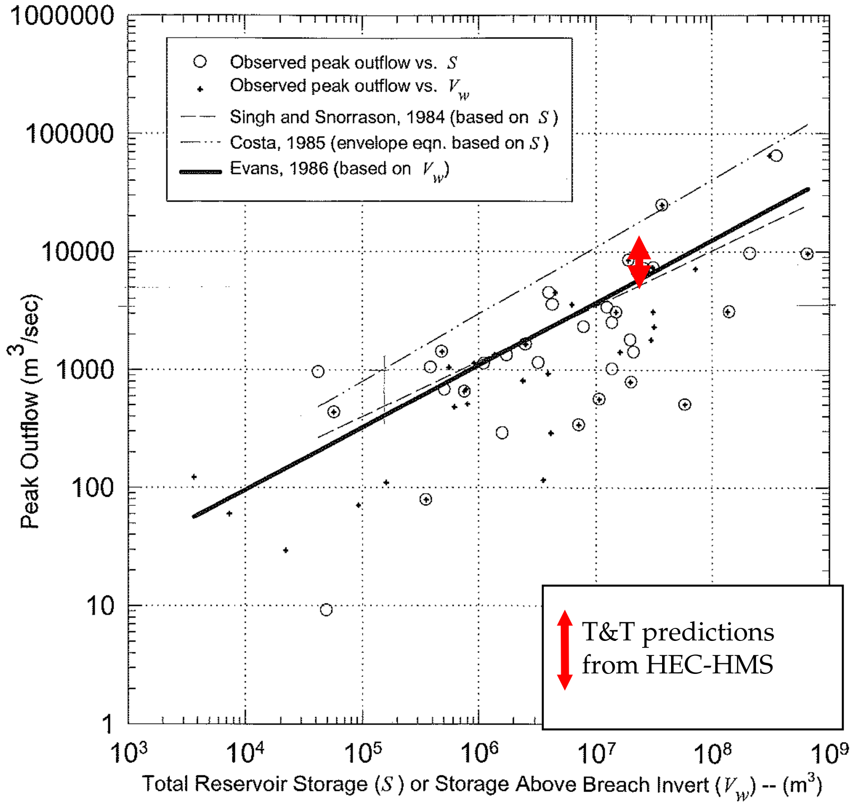


Figure 4-5 Peak discharge versus storage parameters

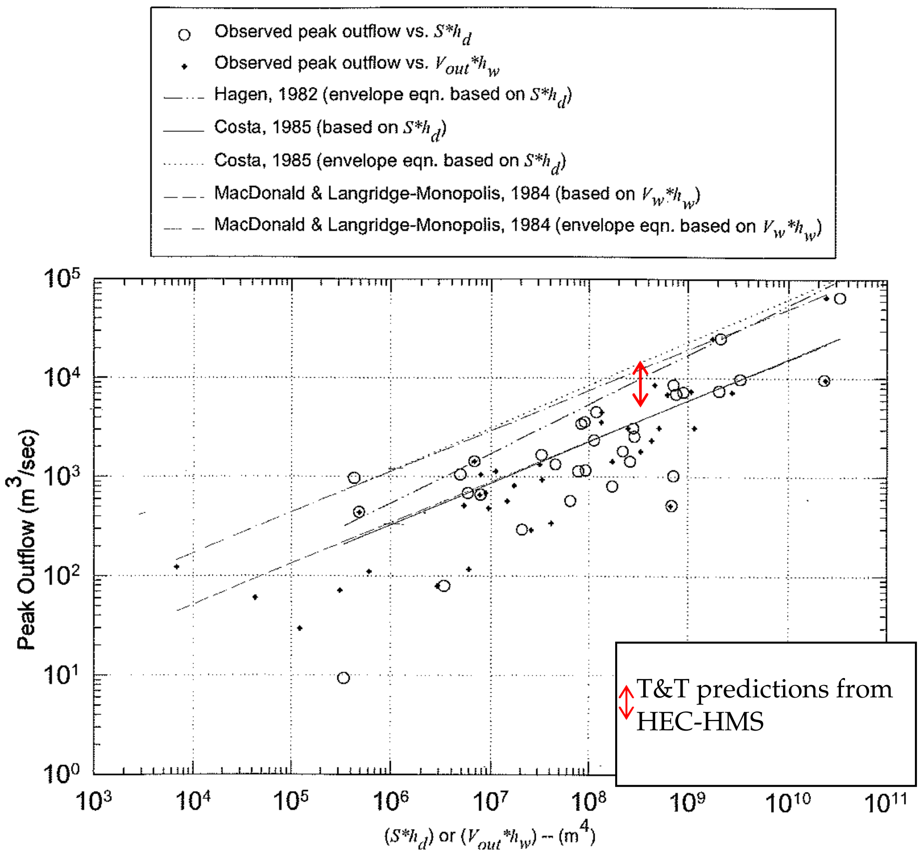


Figure 4-6 Peak discharge versus (volume x height) parameters

The relationships between peak flow and storage, and between peak flow and the product of storage and dam height both suggest that the HEC-HMS predicted peak outflows exceed historically observed flows and empirical relationships. However, the HEC-HMS modelled flows are lower than the peak flow relationship based on dam height alone.

Since the breach progression time has been shown to be an important parameter for breach hydrograph generation, the impacts of the resultant floodwave for each of the three scenarios have been assessed.

The other breach parameter that can govern the outflow hydrograph is the breach width, and there is a similarly wide range of estimates for this parameter based on available empirical formulae. However, as noted earlier, the breach width at the dam site, particularly the bottom width, is physically constrained by the narrow and steep river valley such that its practical limits are much more constrained.

5 Downstream Floodwave

5.1 Introduction

The DHI Mike Flood modelling suite was used to represent the Waimea River and its Lee River tributary from the proposed dam site to the mouth of the Waimea River near Rabbit Island. The modelling approach taken combines a 1 dimensional representation (Mike 11) of the river channel and confined steep-sided valley regions with a 2 dimensional representation (Mike 21) of the floodplain downstream from the Wairoa Gorge. This ensures optimal representation of the channel geometry and floodplain topography.

A site visit was undertaken in June 2009 and confirmed that the selected modelling approach was the most appropriate and accurate method for dam break flood assessment for the Lee River/Waimea River setting. Information was gained on the potential hydraulic roughness of the river and floodplain surfaces, as well as information on land use, terrain and features of importance for the hazard assessment.

The model was used to determine the flow characteristics (depth, velocity, flow, and lateral extent) of the dam break flood wave from the proposed dam location to the mouth of the Waimea River.

5.2 Model build

5.2.1 Channel geometry

The Mike 11 component of the Mike Flood model was developed from cross sections at approximately 250 m intervals along the length of the river. The cross section information was developed from LIDAR data provided by Tasman District Council (TDC).

The floodbanks in the rivers were incorporated directly from the LIDAR data. The LIDAR data is recorded at approximately 1 m intervals, and checks of the data were made to ensure that the floodbanks were well represented.

The steep- sided valleys of the Wairoa River and Lee River where the floodwave would be confined laterally were represented in the Mike 11 model (1D). Approximately 5 km south-east of Brightwater the topography becomes flatter and the flow velocity slows. A combination of Mike 11 and Mike 21 models was applied from this location downstream to the mouth of the Waimea River. The topography for the Mike 21 model was generated from TDC's LIDAR data at 10 m grid spacing.

The model chainages from the Mike 11 model are shown in Table 5-1. As noted earlier, the Lee River flows into the Wairoa River about 16 km downstream of the dam site, and is called the Wairoa River from this point. Some 7.5 km further downstream the Wai-iti River joins the Wairoa River, and downstream of their confluence the river is known as the Waimea River.

Table 5-1 Model chainages and locations

Lee/Wairoa/Waimea River Chainage (m)	Location	River
0	Lee Dam	Lee River
2910	Lucy Creek confluence	Lee River cont.
8220	Fairdale	
8970	Upstream extent of Mike 21 model	
12700	Wairoa River confluence	Wairoa River
16470	State Highway 6 bridge	
20330	Wai-iti River confluence	Waimea River
24220	Coastal Highway Bridge (SH60)	

5.2.2 Boundary conditions

The modelled dam break hydrograph from the HEC-HMS model was used as an upstream inflow boundary into the model.

While stopbanks on the Waimea River and lower Wairoa River have been represented as part of the modelled terrain, potential failure of these stopbanks as a result of overtopping has not been modelled. Depending on the depth of overtopping, these stopbanks may themselves fail and breach, resulting in a secondary but lesser dam break effect on the floodplain. It is anticipated that this secondary release would result in an increased area of inundation in the upstream part of the floodplain but cause a reduction in the flooded area further downstream as flow is shed from the main river channel as breakout flow.

A more detailed assessment of likely locations of stopbank breaches and potential breakout flows requires a systematic consideration of overtopping depth and duration along the full length of the river protection works. Extension of the assessment to map secondary inundation areas would require modelling of a range of stopbank breakout scenarios. However, this is a refinement which is considered unnecessary within the context of the current workscope because, as described in subsequent sections, failure of the Lee River Dam would result in damages which clearly fall within a high PIC classification. Such an assessment may be worthwhile for completeness for inclusion in the final Emergency Action Plan for the dam.

A constant tide level has been assumed for the downstream boundary of the hydraulic model. Ground levels (bathymetry) for this area, between the mouth of the Waimea River and Rabbit Island, have been derived from available hydrographic charts, and is thus less accurate than in the floodplain. Mainly for the same reason as cited above (that the Lee River Dam is clearly a high PIC dam), it was considered unnecessary to undertake an additional sensitivity analysis to a range of coincident tidal conditions.

5.2.3 Resistance

A literature review of roughness coefficients (Chanson, 2004; Chow, 1973; CIRIA, 1990; USACE (HEC-RAS help, undated); NZIE, 1977) was carried out to determine suitable roughness coefficients for the channel and floodplain roughness in the Mike 11 and Mike 21 models.

Table 5-2 presents a summary of the findings.

Table 5-2 Roughness (Manning n) coefficients for different river and vegetation cover types

		Descriptor	Min	Normal	Typical
RIVER	Mountain streams	Gravels, cobbles and few boulders	0.03	0.05	0.04
	Main channels	Clean, straight, full, no rifts or deep pools	0.03	0.025	0.03
	Main channels	Clean, winding, some pools and shoals	0.033	0.045	0.04
FLOODPLAIN	Hedge	Heavy stand of timber, few downed trees, little undergrowth, flow below branches	0.08	0.12	0.1
		Heavy stand of timber, few down trees, little undergrowth, flow into branches	0.1	0.16	0.12
	Road	Rough asphalt	0.015	0.016	0.016
	Vineyard	Light brush and trees, in summer	0.04	0.08	0.06
	Orchard	Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.08	0.15	0.1
	Native bush	Medium to dense brush, in summer	0.07	0.16	0.1
	Pasture	Short grass	0.025	0.04	0.03
	Brush	Light (summer and winter)	0.035	0.08	0.05-0.06
		Medium (summer and winter)	0.045	0.16	0.07-0.1
	Grass	<50mm			0.024-0.0275
		50-150mm			0.031
		150-250mm			0.034
		250mm-750mm			0.045
		>750mm			0.06-0.065

It is important to note that roughness changes with water depth (Chow, 1959; Chanson 2004; CIRIA, 1990). As water depth increases there is a reduction in the effect that bed resistance has on flow. Therefore under a dam break scenario where flood depths can be very deep, it may be appropriate to use lower resistance values.

The land use and/or vegetation type around much of the Wairoa/Waimea floodplain changes on a seasonal basis. Hence roughness coefficients appropriate for summer may be inappropriate for winter. Similarly, due to crop rotation and changes in land use the land cover can change from year to year.

Therefore, a sensitivity assessment for the 0.9 hour breach formation time for two different roughness approaches has been undertaken. The first assessment was based on using Manning's n = 0.03 for the river channel and a variable roughness across the floodplain. To simplify the land use and vegetation issues raised earlier, a Manning's n = 0.06 was used as a general figure, which was reduced to n = 0.016 around roads.

The results of the dam break assessment are presented in Figure 5-1.

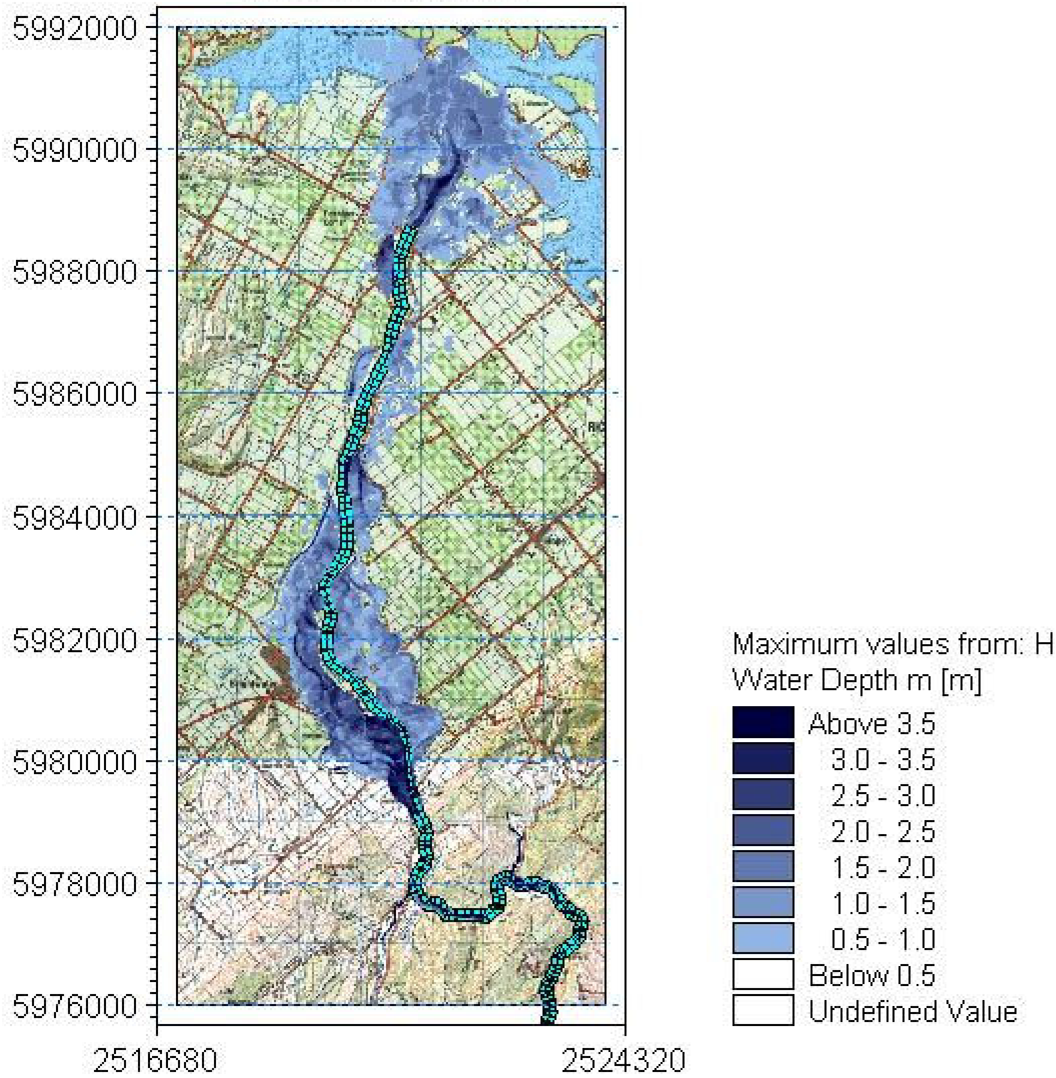


Figure 5-1 Maximum flooding extent for variable roughness coefficient

The second assessment was carried out using a constant roughness coefficient (Manning n = 0.033) in the floodplain. The same roughness as in the previous assessment (n = 0.03) was used for the channel. The modelled floodplain inundation is presented in Figure 5-2.

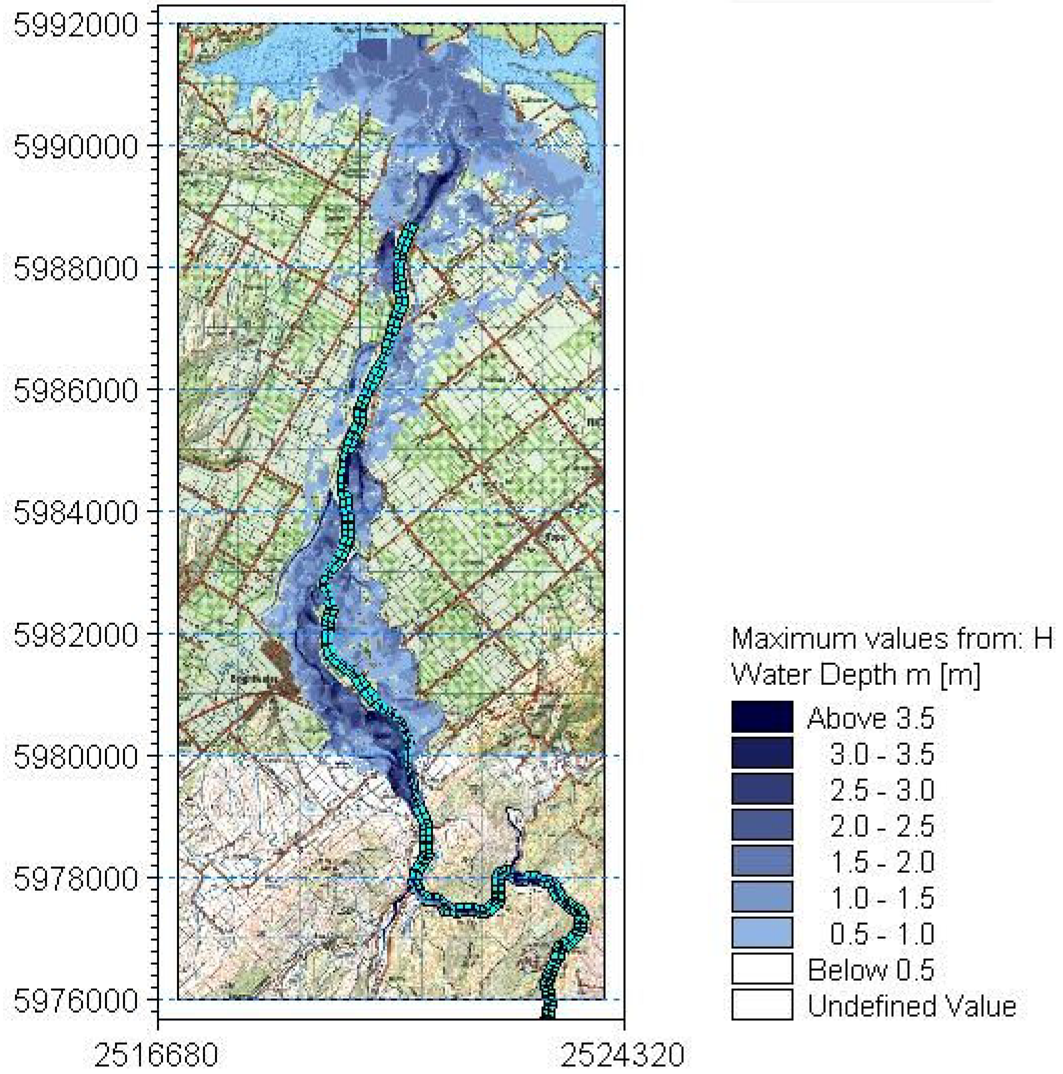


Figure 5-2 Maximum flooding extent for constant roughness coefficient

In order to determine the PAR for the PIC assessment, one is primarily concerned with the overall flooding extent where water depth exceeds 0.5 m, rather than the variation in water depth. A comparison of the flooding extent (where the water depth exceeds 0.5 m) between these two modelled scenarios is shown in Figure 5-3.

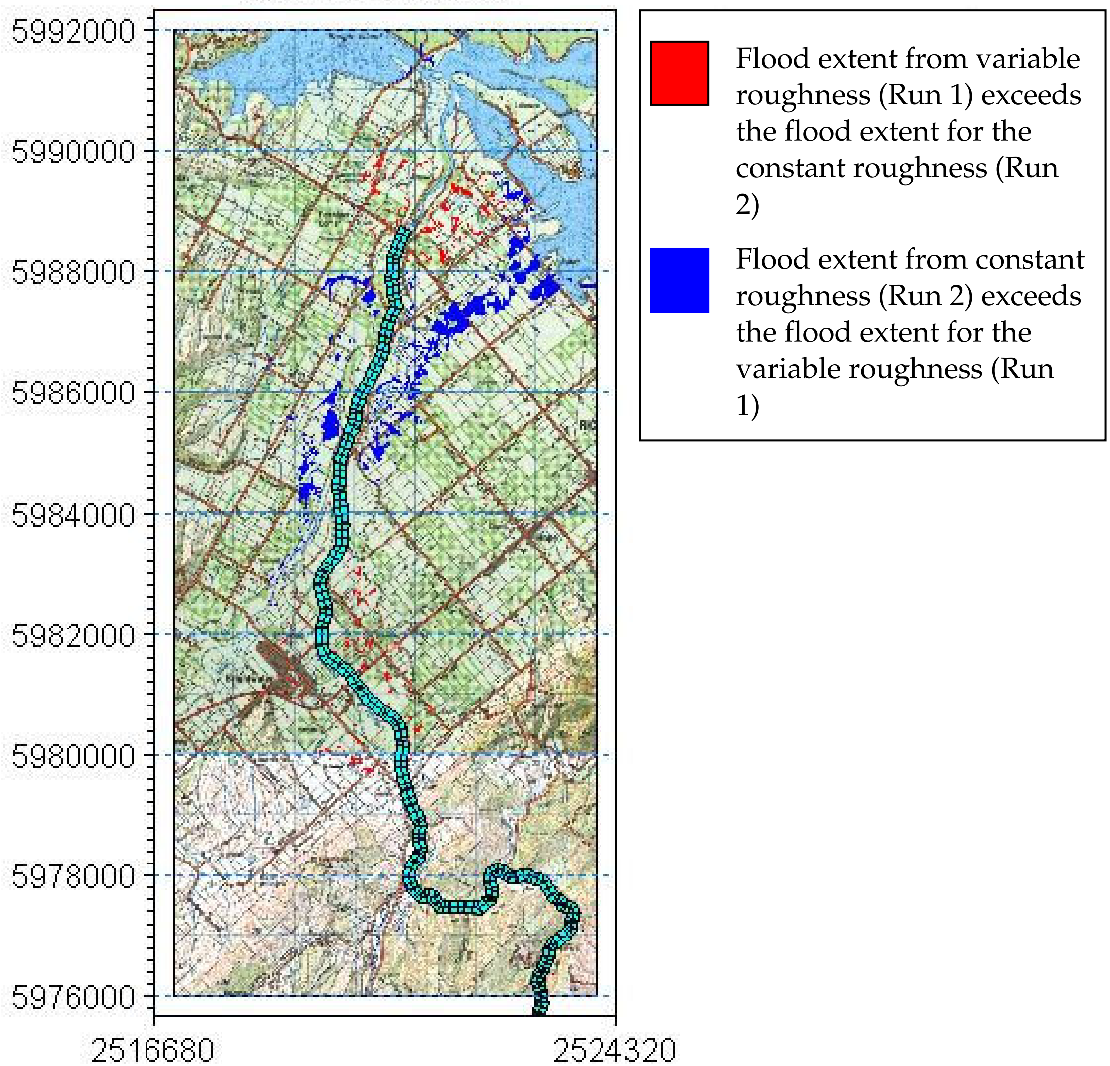


Figure 5-3 Difference in flooding extent between variable and constant roughness assumptions

The results show that the extent of flooding is generally greater downstream when the lower, constant floodplain resistance (n = 0.033) is used (see areas in blue in Figure 5-3). This occurs because there is less flood attenuation further upstream in the floodplain, thereby increasing the flow downstream. Since bed resistance decreases with depth, a low resistance value may be appropriate for a dam break assessment where large water depths may occur. The constant roughness value used is typical for short grass conditions.

Furthermore, because of the uncertainty in land use and vegetation cover it is considered appropriate to use a conservative value for catchment roughness (viz. the constant floodplain resistance case n = 0.033) which would result in the greater inundation area and thus more conservative assessment of PAR (see Section 6).

5.2.4 Model calibration

Figure 5-4 shows the assessed actual flood extents for the large flood of January 1986. This flood has an assessed return period of approximately 65 years and is the largest recorded event since continuous monitoring of the river flows began in November 1957. The flood extents were estimated by (TDC). The peak flow recorded during the flood event at the Wairoa Gorge flow gauging station was 1466 m^3/s.

Figure 5-4 January 1986 Flood event (source: TDC)

Model predictions of the flooding extent resulting from a hypothetical dam break are made in Section 5.3. The model predictions for the dam break event indicate reasonably similar patterns of flooding to the 1986 event which provides confidence in the model's ability to reproduce flood inundation extents.

The peak flows as a result of a hypothetical dam failure are anticipated to be much larger than any floods experienced in the Lee River and Wairoa River above the Wairoa Gorge. Therefore, a detailed model calibration in this reach is not practical. Apart from the assessed inundation extents for a number of significant historical floods by TDC (such as the January 1986 flood shown in Figure 5-4), there is limited detailed information on observed water levels and flows to enable a reliable calibration of the model for the lower reaches of the river and floodplain below the Wairoa Gorge.

5.3 Results

The Mike Flood model was used to determine the inundation extent and floodwave characteristics following a hypothetical dam break at the Lee River Dam. Breach parameters corresponding with the three modelled scenarios are presented in Table 4-1.

The flow and flood depth hydrographs for the three dam break scenarios are shown for a number of locations along the river in Figure 5-5 to Figure 5-10. The flood depth relates to water depth in the river, and therefore does not reflect the water depth in the floodplain.

A description of the river chainage locations is provided in Table 5-1.

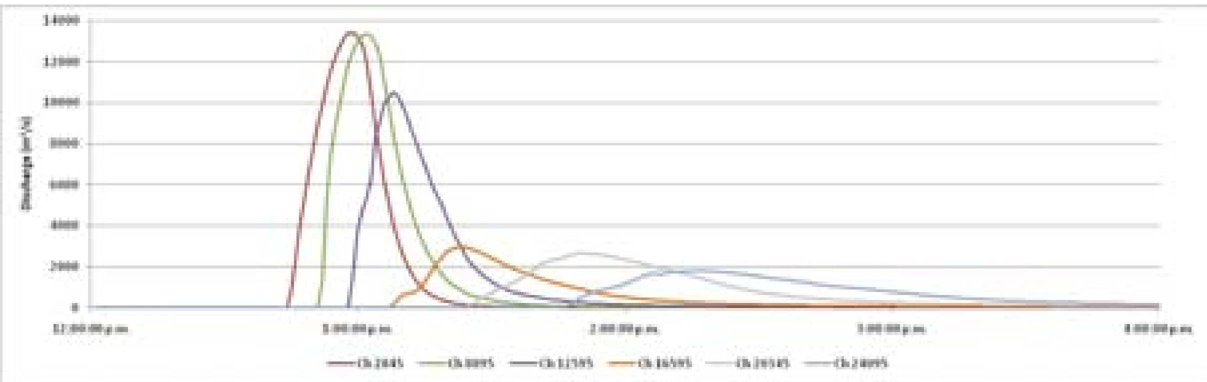


Figure 5-5 Flow hydrograph at selected river locations for 0.5 hour breach formation time

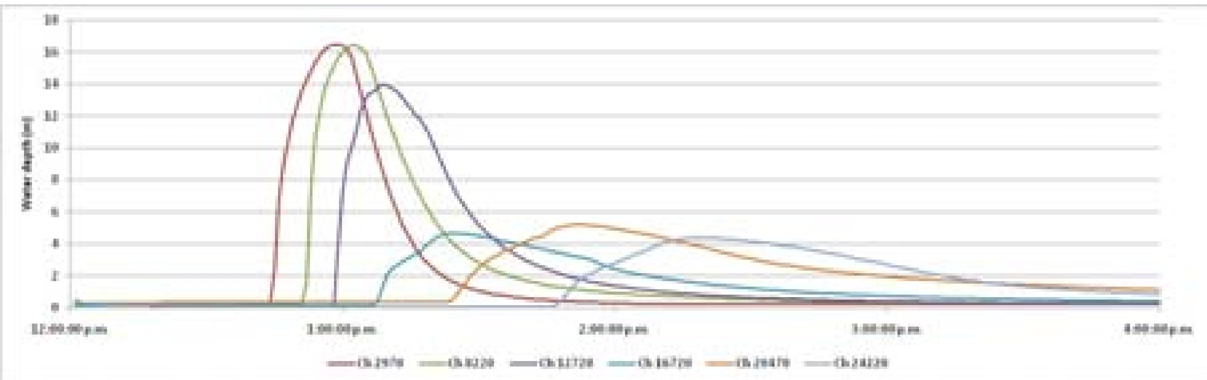


Figure 5-6 Water depth at selected river locations for 0.5 hour breach formation time

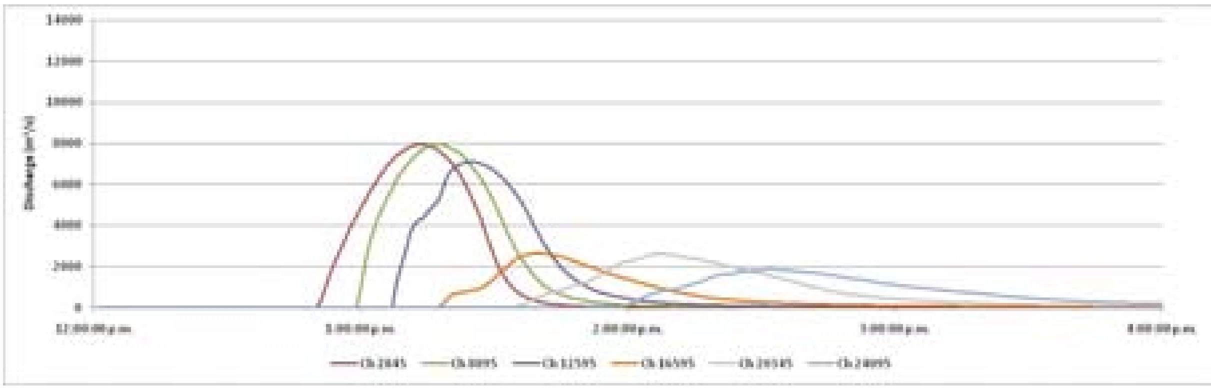


Figure 5-7 Flow hydrograph at selected river locations for 0.9 hour breach formation time

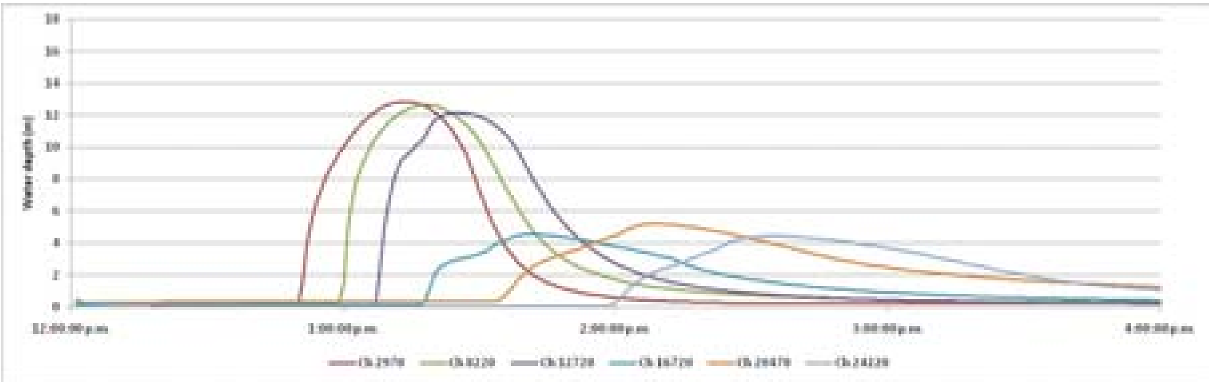


Figure 5-8 Water depth at selected river locations for 0.9 hour breach formation time

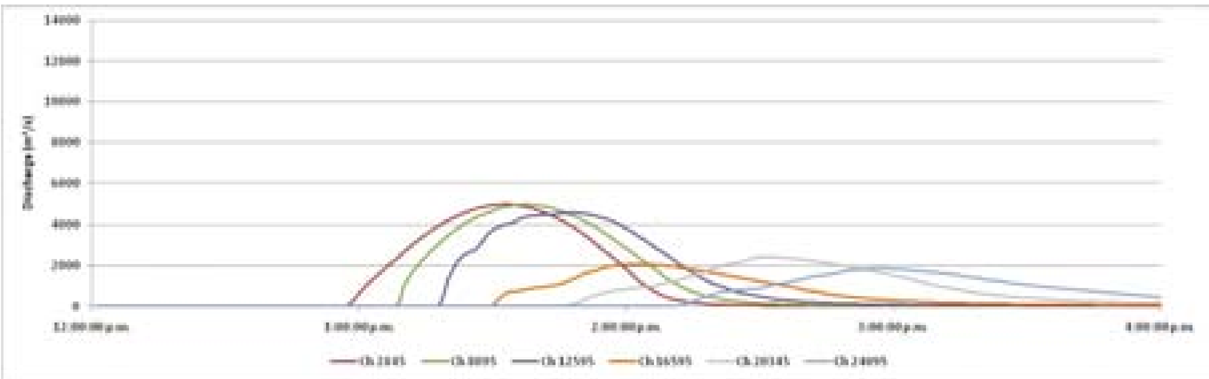


Figure 5-9 Flow hydrograph at selected river locations for 1.5 hour breach formation time

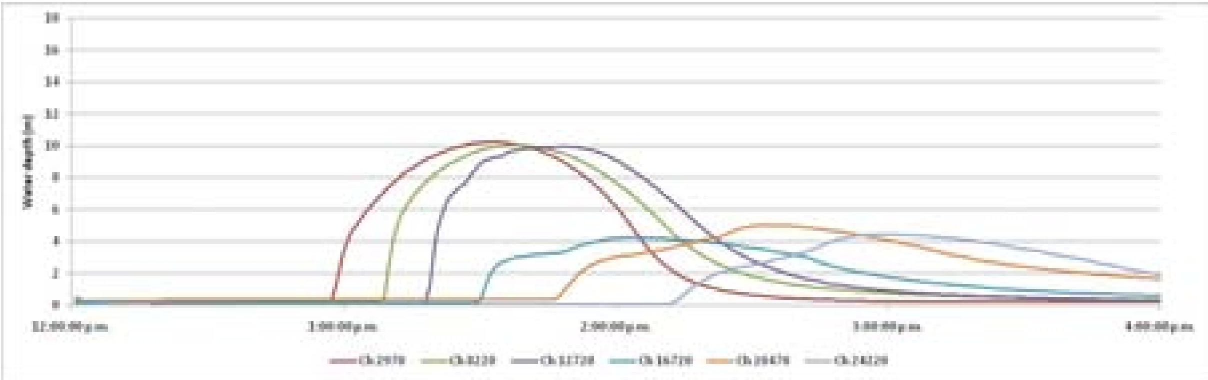


Figure 5-10 Water depth at selected river locations for 1.5 hour breach formation time

The flow hydrograph sensitivity results indicate that the choice of dam breach progression time makes a significant difference to the peak flow in the first 10 km downstream of the proposed Lee Dam site. There is very little flow attenuation in this stretch of the river and therefore peak flows along the river would be similar to the peak flows at the proposed dam site.

By the time the floodwave reached the SH6 bridge near Brightwater the peak flow would have been significantly attenuated and the peak flow is not significantly affected by the choice of dam breach progression time.

A lower water depth around chainage 16,720 m would occur because of the significant amount of breakout flow from the river that occurs upstream. The water depth downstream would increase as flow is returned to the Waimea River via the Wai-iti River.

At chainage 23,500 m (confluence of Wai-iti River) the highest maximum flow, which corresponds with the 0.5 hour breach formation would be approximately 2,590 m^3/s, while the lowest maximum flow would be 2415 m^3/s (1.5 hour breach formation time). The difference of 175 m^3/s (7%) is relatively small. Similar trends are observed for water depth, where water depth range would be only 0.17 m between the 0.5 hour and the 1.5 hour breach scenarios.

The Building (Dam Safety) Regulations 2008 necessitates identification of the population at risk. The ‘population at risk’ is defined as those people affected by flood water depths greater than 0.5 m. The inundation extents with maximum flooding depth greater than 0.5 m for the three dam break scenarios are shown in Figure 5-11 to Figure 5-13.

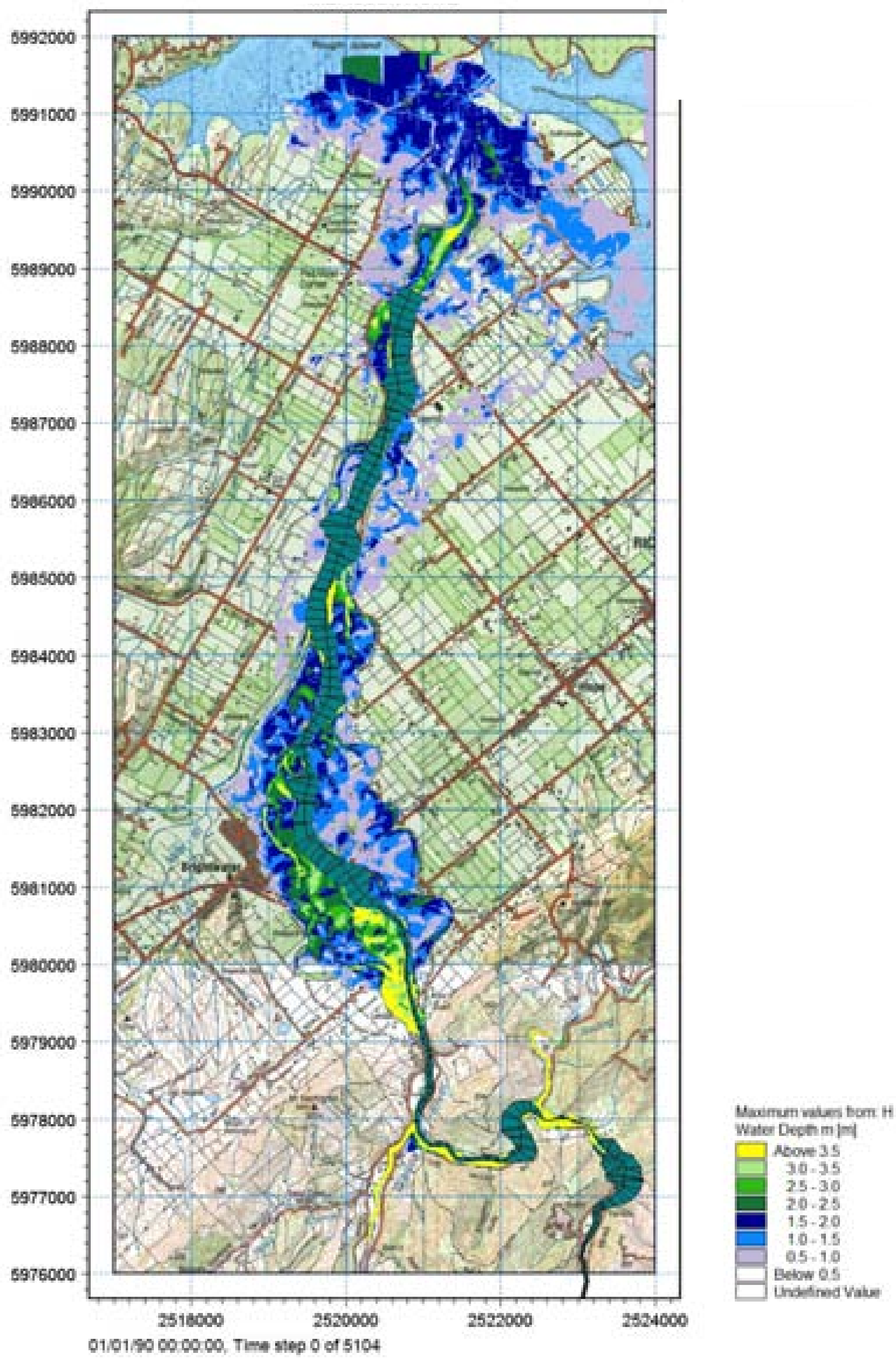


Figure 5-11 Maximum flood extent (>0.5m depth) for 0.5 hour breach formation time[2]

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[2] The water depths in the river channel are not displayed. The hatched green sections in the river channel display the approximate Mike 11 model extents.

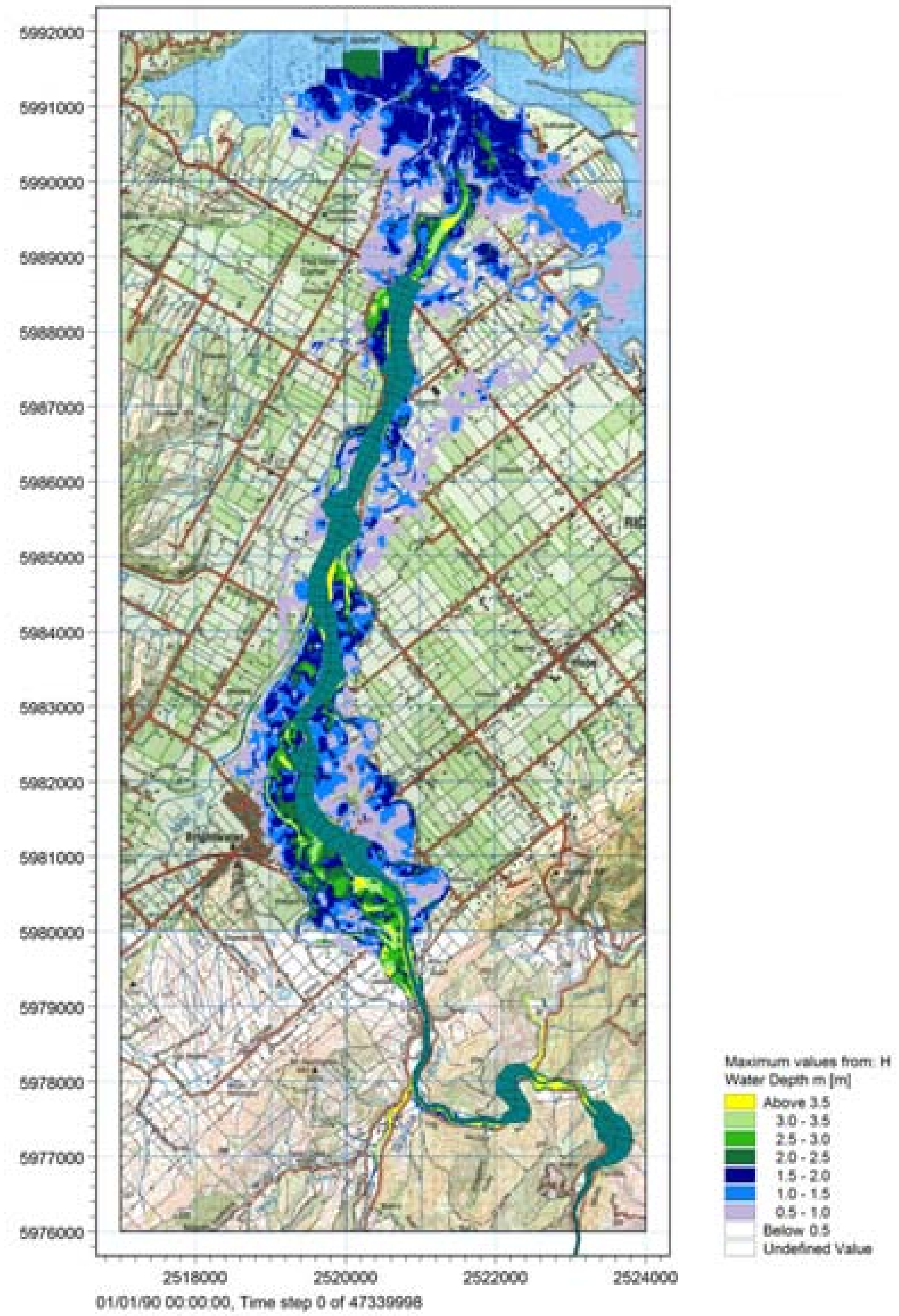

Figure 5-12 Maximum flood extent (>0.5m depth) for 0.9 hour breach progression time[3]

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[3] The water depths in the river channel are not displayed. The hatched green sections in the river channel display the approximate Mike 11 model extents.

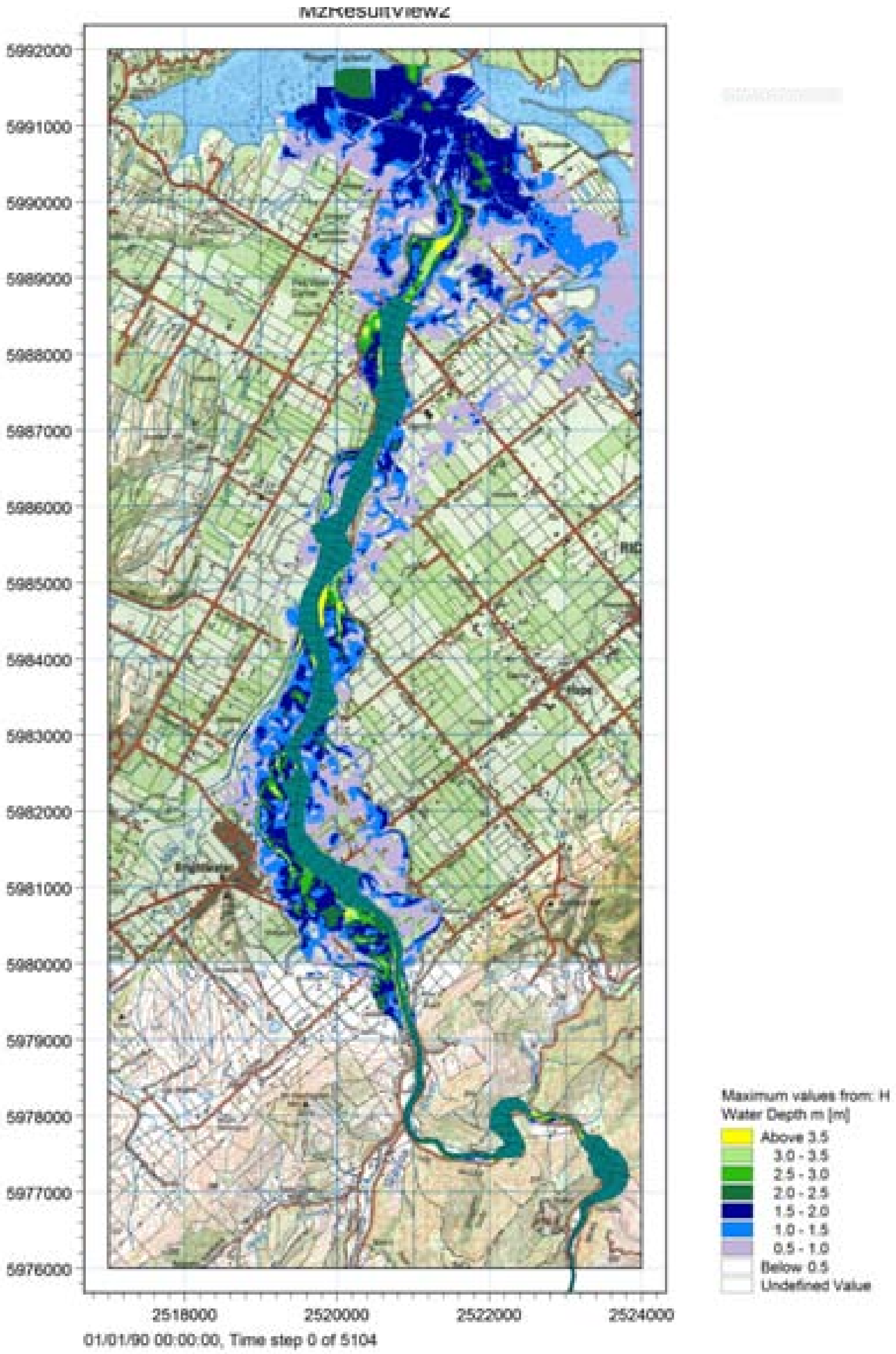

Figure 5-13 Maximum flood extent (>0.5m depth) for 1.5 hour breach formation time[4]

[4] The water depths in the river channel are not displayed. The hatched green sections in the river channel display the approximate Mike 11 model extents.

The results of the three modelled scenarios show that the dam break flow would first overtop the river banks approximately 5 km south-east of Brightwater, near Leedale and Max's Bush. In the area between Leedale and SH6 bridge there would be significant break out flows in excess of 3 m deep along the true left bank, and to a lesser extent along the right bank.

The current velocity at the break out flow along the left bank is likely to be in excess of 4 m/s in isolated places for short periods of time.

The inundation flood flows would pass the eastern and northern extents of Brightwater and they are constrained to the southern side of the Wai-iti River. The Wai-iti River would divert flood flows back into the Waimea River.

On the true right bank, the largest break-out flows would occur near Edens Road. Floodwater that breaks out from the river near this location would be conveyed parallel to the river before diverting towards the coast to the north side of Bartlett Road.

The flood inundation extent would increase as breach progression time decreases. The smallest flood extent corresponds with the 1.5 hour breach formation scenario, and the largest flood extent corresponds with the 0.5 hour breach.

Figure 5-14 shows that the inundation area is influenced by the choice of the breach formation time i.e. the difference between the 0.5 hour and 1.5 hour breach scenarios.

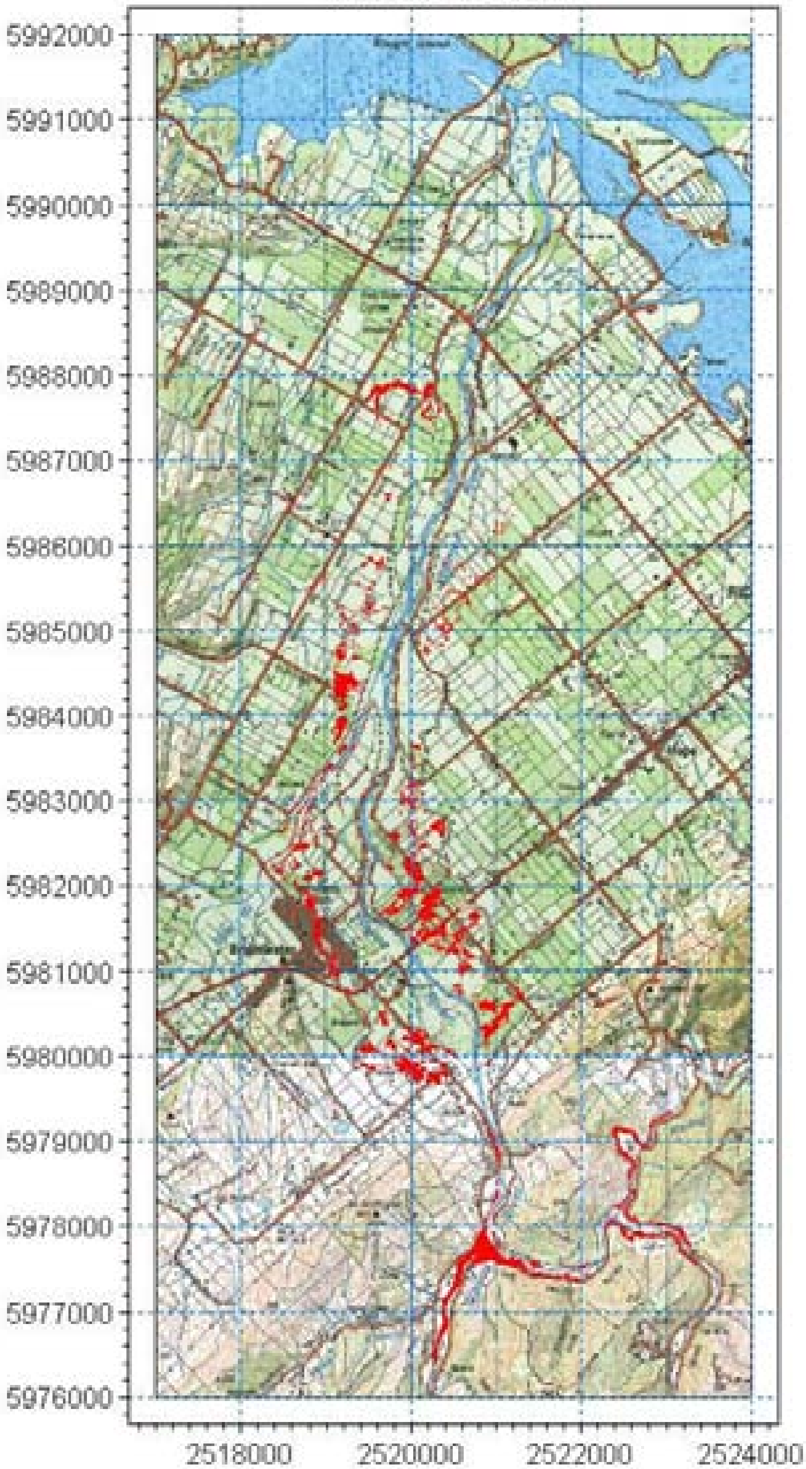

Figure 5-14 Difference in inundation extent between the 0.5 hour and 1.5 hour breach scenario

The areas in red highlight the areas that may or may not flood depending upon the selected breach formation time. The inundation extent is not significantly affected by the choice of this breach parameter. However, the small increases in area could result in a large increase in PAR (particularly the identified area in Brightwater). Therefore, the results from all three scenarios will be carried through to the PIC assessment (Section 6). This will determine whether the PIC rating is sensitive to assumptions in certain breach parameters.

6 Categorisation of Potential Impact

6.1 Description of the incremental hazard

The following discusses and assesses the downstream hazard potential if the proposed dam failed. To put this discussion into proper context, it is essential to draw the distinction between hazard potential (that is, the effects of the dam breach were it to occur) and the risk or probability of the dam breach actually occurring. The risk of failure occurring for a dam engineered, built, maintained and monitored to appropriate standards, as would be the case for the Lee dam, would be extremely low.

The maximum number of properties at risk from dam break for three breach scenarios has been determined. Three breach scenarios have been evaluated because of the uncertainty in the breach formation time.

The number of properties at risk has been identified from the following model results:

- The Mike Flood extent mapping shown in Figure 5-11, *Figure 5-12* and Figure 5-13
- Mike 11 model results for locations upstream of the Mike 21 model.

The numbers of properties at risk under the three dam break scenarios are shown in Table 6.1. Aerial photography and land boundary information was primarily used to identify buildings that may be flooded as a result of a dam break. The estimated number of residential properties that would be inundated is likely to be an over-estimate since it was not possible to ascertain building use from aerial photography. However, to avoid being overly conservative, the assumption was made that there is only one residential property per building lot.

Table 6-1 Property at Risk from Inundation

Location	Scenario 1 **0.5 hr breach progression time**	Scenario 2 **0.9 hr breach progression time**	Scenario 3 **1.5 hr breach progression time**
Number of flooded residential properties downstream of Wairoa Gorge (>0.5m flood depth)	Approximately 300	Approximately 290	Approximately 260
	(Approximately 2/3rds of the flooded properties are located in Brightwater)		
Number of flooded residential properties upstream of Wairoa Gorge (>0.5m flood depth)	14	10	9
Additional facilities of interest that would be flooded to greater than 0.5m (✓=flooded, x = no flooding, or less than 0.5m):			
Brightwater School - 106 Ellis Street	✓	✓	✓
Saint Peter & Paul Catholic Church at Waimea West Road, Richmond	Marginal	Marginal	X
Speedway grounds at 122 Lansdowne Road (marginal)	Marginal	X	X
Appleby School - 19 Moutere Highway (marginal)	Marginal	Marginal	X
Girl Guide regional camp at Paretai, 129 Lee Valley Road	✓	✓	✓

Other potential environmental and economic damages arising from a dam break event include the following:

- destruction or damage of some vineyards and orchards located within the inundation area,
- livestock losses and loss of topsoil,
- damage to road infrastructure including potential damage to the SH6 bridge,
- deposition of silt in downstream areas as the flood recedes derived from the dam embankment material eroded and entrained into the dam breach outflow,
- economic loss to the dam owner, including loss of the asset and operating revenue,
- damage to existing river protection/training works/stopbanks,
- other potential environmental and economic damages arising from a dam break event.

6.2 Recommended potential impact category

From the results shown in Table 6-1, and with reference to Table 2-1 for use in dam classification, the incremental population-at-risk (PAR) is expected to lie in the range greater than 100.

Excluding the economic losses suffered by the dam owner, and with reference to Table 2-2 for use in classifying incremental damage, the following damage descriptors are indicated:

- 'Catastrophic' for the number of residential houses that would be destroyed
- 'Major' for damage to critical or major infrastructure and the time to restore to operation
- 'Major' for Natural Environment damage
- 'Major' for community recovery time (years).

Because the population at risk is greater than 100, and the assessed damage level is either 'Major' or 'Catastrophic' the dam is classified as a "High" potential impact category (PIC) dam per Table 2-1.

The assessment of the High PIC rating is not sensitive to the uncertainty in the selection of the dam breach formation time nor to the precise assessed damage level.

7 Dam Break Mitigation

This section provides information regarding the potentially inundated area, warning time and a discussion of water depth and velocity and damage/risk to life to assist in developing an Emergency Action Plan / Civil Defence Plan.

7.1 Inundation area

The maximum inundation area occurs under the 0.5 hour dam break scenario. The inundation extent for this breach scenario is shown in Figure 7-1. Note that the extent is larger than in Figure 5-11 because Figure 5-11 only shows the extent of inundation greater than 0.5 m. (The difference in flooded area between the two figures is therefore the inundation extent that is less than 0.5 m in depth.)

7.2 Warning time

Warning time and evacuation can dramatically influence the loss of life in a dam break event.

Case history based procedures developed by the US Bureau of Reclamation indicate that the loss of life can vary from 0.02% of the "population-at-risk"[5] when the warning time is 90 minutes to 50% of the population-at-risk when the warning time is less than 15 minutes.

Table 7-1 identifies the duration from the start of the dam break to the time that the flood wave would first arrive at specific locations downstream. The table also shows the time that it would take (from the start of the dam break) for the peak water depth (and flow) to occur.

The warning time is not relevant for the PIC assessment under New Zealand's Dam Safety Regulations, but it is important for emergency planning.

[5] The "population-at-risk" in the USBR report is different to the definition of "population at risk" under the NZ Building (dam safety) Regulations 2008. The USBR definition includes all people living in the area inundated by a flood, unlike the NZ regulations which include only those people affected by flood depths greater than 0.5 m.

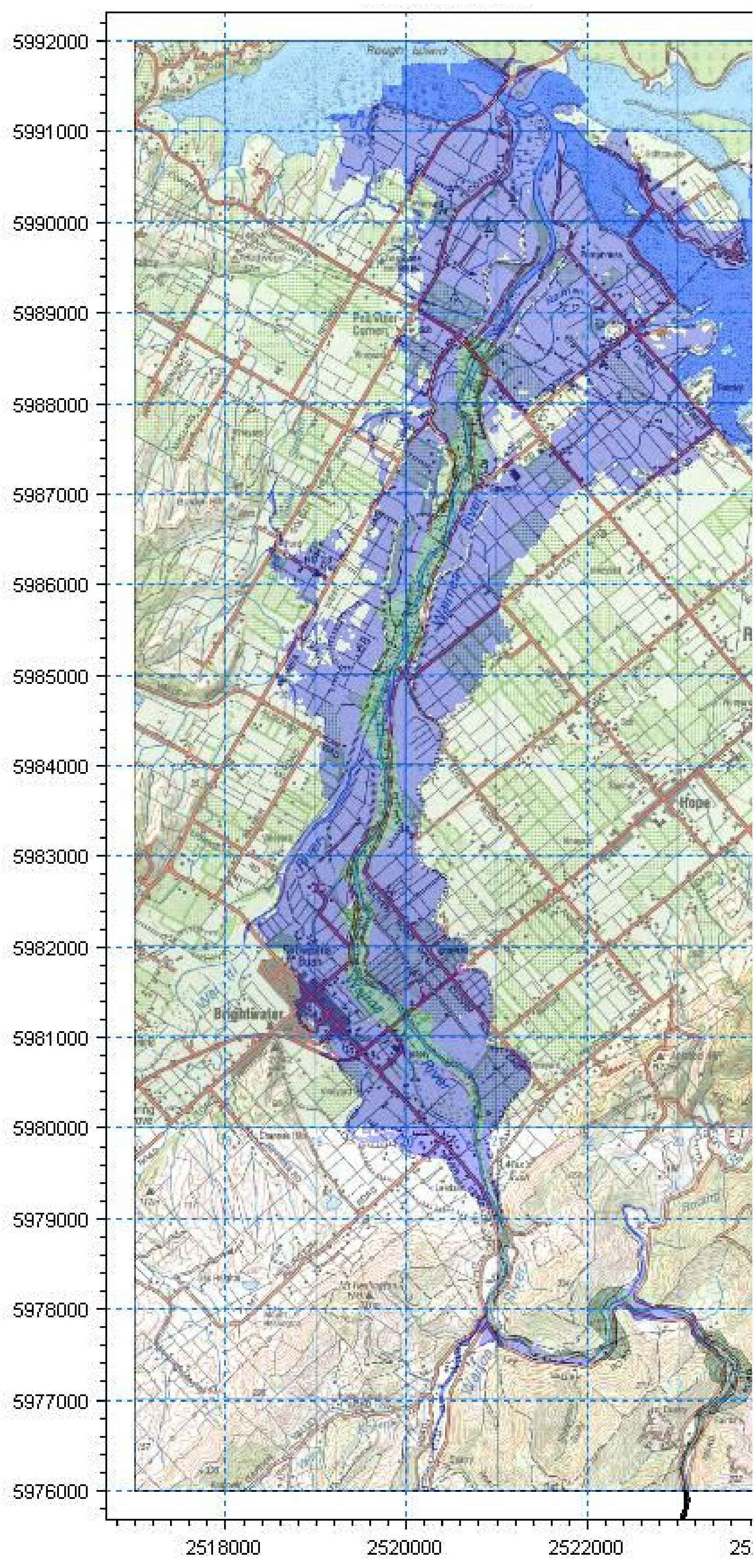

Figure 7-1 Maximum inundation extent from dam break

Table 7-1 Peak flow during sunny day failure

River Chainage (m)	Location	Scenario A (0.5 hr breach formation time)		Scenario B (0.9 hr breach formation time)		Scenario C (1.5 hr breach formation time)	
		Time for flood-wave to first arrive	Time for peak water depth to occur	Time for flood-wave to first arrive	Time for peak water depth to occur	Time for flood-wave to first arrive	Time for peak water depth to occur
0	Lee Dam	**T=0 min**	-	**T=0 min**	-	**T=0 min**	-
2910	Lucy Creek confluence	+8 min	+23 min	+10 min	+36 min	+13 min	+52 min
8220	Fairdale	+16 min	+27 min	+22 min	+40 min	+28 min	+57 min
12720	Wairoa River confluence	+19 min	+33 min	+26 min	+38 min	+37 min	+66 min
16470	State Highway 6 bridge at Brightwater	+31 min	+47 min	+40 min	+62 min	+48 min	+80 min
20330	Wai-iti River confluence	+46 min	+74 min	+54 min	+89 min	+63 min	+110min
24220	Coastal Highway Bridge (SH60)	+68 min	+102min	+78 min	+116min	+88 min	+135min

Figure 7-2 to Figure 7-5 show the flood inundation at various times after dam break commencement for the 0.9 hour reach formation scenario (i.e. the intermediate case).

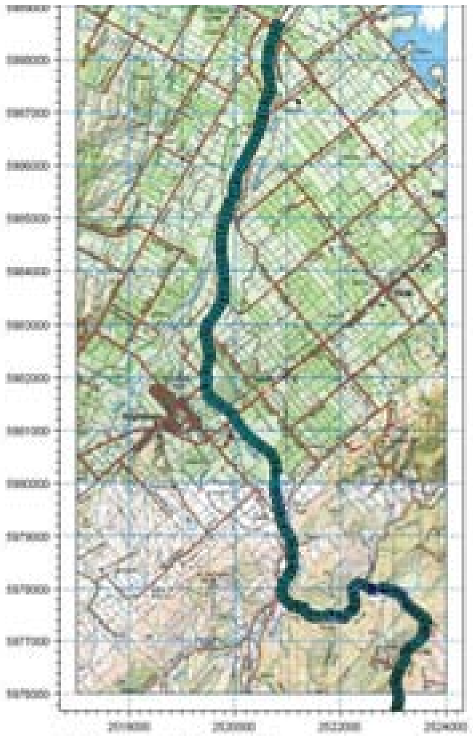

Figure 7-2 Time = DB +40mins(DB = Dam break initiation time)

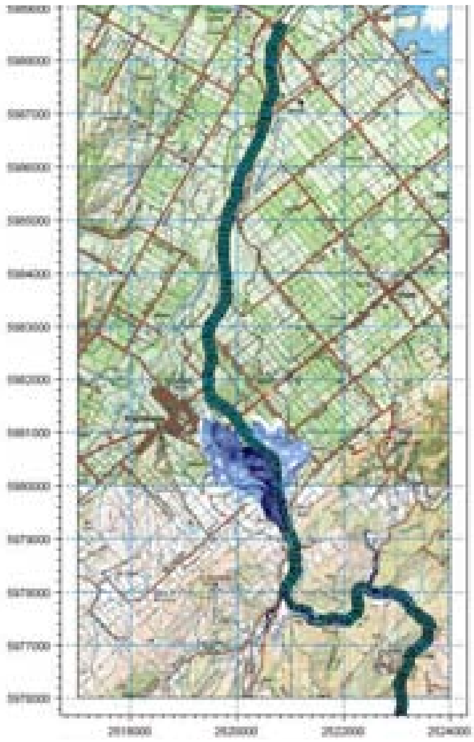

Figure 7-3 Time = DB + 55 minutes

Figure 7-4 Time: DB + 85 minutes

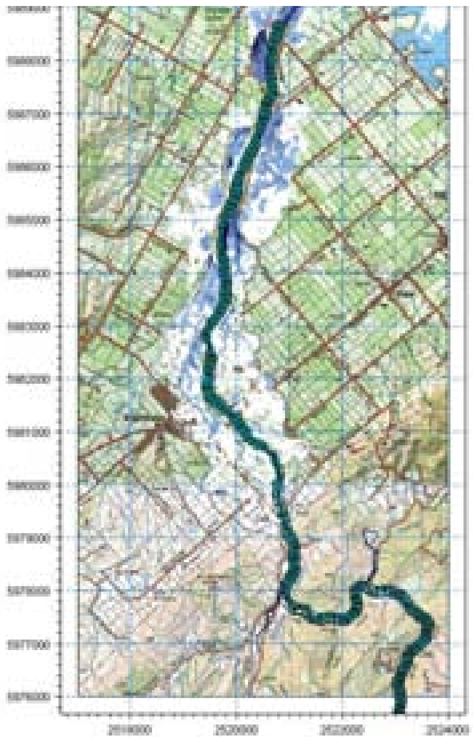

Figure 7-5 Time: DB + 115 minutes

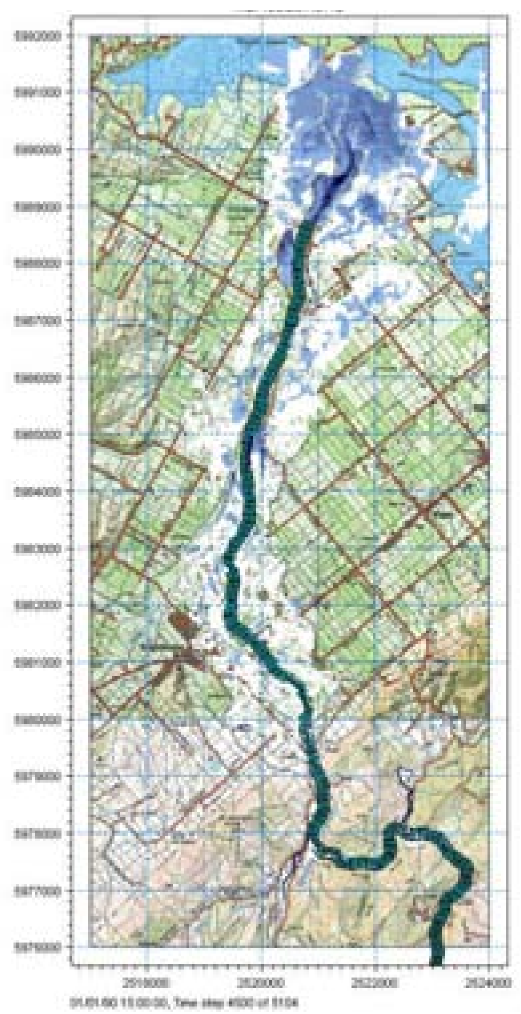

Figure 7-6 Time: DB +145 minutes

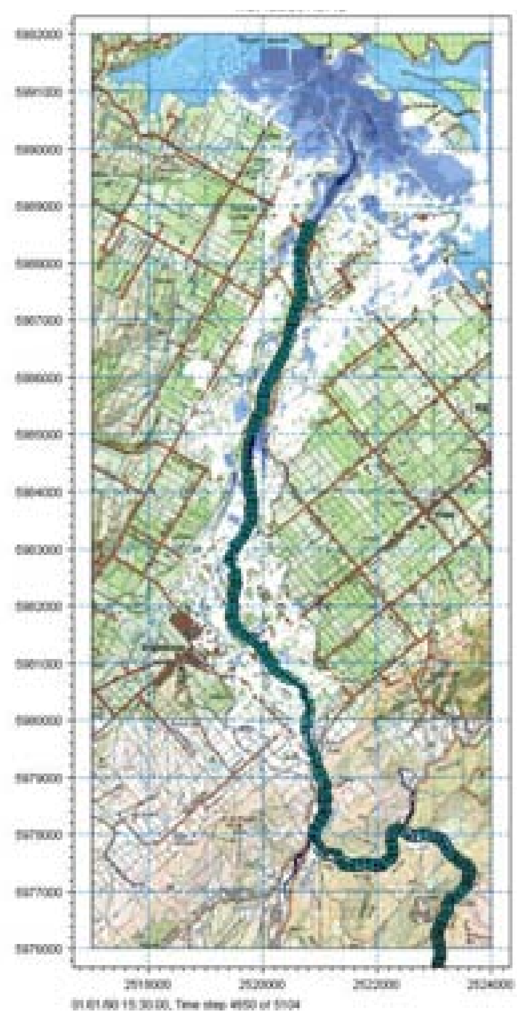

Figure 7-7 Time: DB + 175 minutes

7.3 Flow depth and velocity

In order to assess the effect of dam break flows on structures and people, the parameter given by the product of depth (*d*) and velocity (*v*), i.e. *dv*, can assist with providing a basis for assessment.

Reiter (2000) provided an indication of risk for loss of life and damage classes for houses based on *dv*. This has been reproduced in Table 7.2.

Table 7-2 Risk to life and houses using *dv* parameter (Reiter, 2000)

Risk for loss of life classes, damage classes of cars and houses	Damage parameter *dv* (m^2/s)		
	Small damages, small danger	**Medium damages, medium danger**	**Total damages, very high danger**
Lightly constructed detached one family houses	< 1.5	1.30 - 2.50	> 2.50
Well constructed wooden houses	< 2.0 (v>2.0 m/s)	2.0-5.0 (v>2.0m/s)	> 5.0
Brick houses, concrete structures	< 3.0, (v>3.0 m/s)	3.0-7.0 (v>2.0m/s)	> 7.0

A paper by Amos *et al* presented at the 2004 ANCOLD / NZSOLD conference discussed the application of the *dv* parameter to determine the potential hazard, citing Project Aqua as an example. The paper referred to work at Helsinki University (Hut, 2000) in which hazard criteria developed by Hut and in other papers associated with the research project were extensive. The criteria were simplified by Amos *et al* into the following description:

$dv < 0.5$	No danger to life
$0.5 < dv < 1.0$	Some danger to life exists
$dv > 1.0$	Danger to life significant

The distribution of *dv* across the potentially inundated area corresponding with a 0.9 hour dam breach is mapped in Figure 7-8.

The results show that the highest risk to life and buildings in the unlikely event of a dam failure is the true left bank of the floodplain area generally to the east of Brightwater.

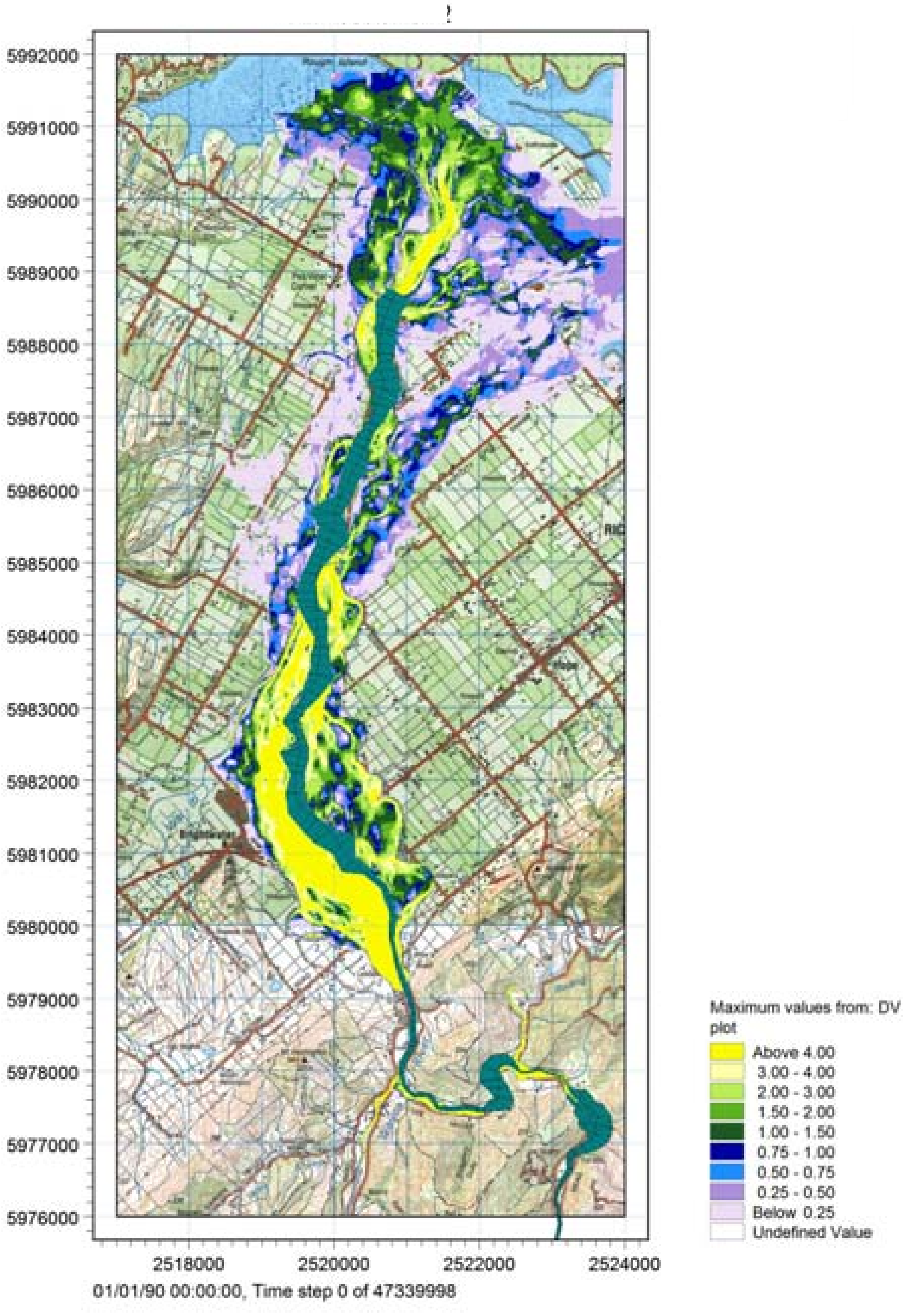

Figure 7-8 Depth times velocity (dv) assessment of inundation area

8 Conclusion

The results of the dam break assessment show that the Lee River Dam should be categorised as High PIC. The categorisation was determined largely from the high PAR, where modelling shows that approximately 260-300 properties would be at risk of flooding from water depths in excess of 0.5 m.

In the unlikely event of dam break of the Lee River Dam, the northern and eastern areas of Brightwater township are the most densely populated areas that would significantly affected. Following dam breach initiation, flood waters near Brightwater would be likely to start rapidly rising within a period of 35 to 45 minutes, and peak flood depths would be likely to occur between 45 and 75 minutes after breach initiation.

It is essential to draw the distinction between hazard potential (that is the effects of the dam breach were it to occur) and the risk or probability of the dam breach actually occurring. The risk of failure occurring for a dam engineered, built, maintained and monitored to appropriate standards, as would be the case for the Lee dam, would be extremely low.

9 References

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10 Applicability

This report has been prepared for the benefit of the Waimea Water Augmentation Committee with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

TONKIN & TAYLOR LTD

Environmental and Engineering Consultants

Report prepared by: Authorised for Tonkin & Taylor by:

Jon Rix — Water Resources Engineer

David Leong — Project Director

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Appendix 1 Outline Emergency Action Plan

Report

WAIMEA WATER AUGMENTATION COMMITTEE

LEE DAM: OUTLINE EMERGENCY ACTION PLAN

Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE

Report prepared by:

TONKIN & TAYLOR LTD

Distribution:

WAIMEA WATER AUGMENTATION COMMITTEE	1 copy
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December 2009

T&T Ref: 24727.304

Amendment Record Sheet

Amendment Number	Effective Date	Subject	Pages	Signed
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Summary

This document provides an outline of a possible Emergency Action Plan (EAP) for the operational phase (as distinct from construction phase) of the proposed Lee River Dam. It is intended to provide an indication of the envisaged content for the operative EAP for the completed dam. Details will need to be completed and particular descriptions in this document will need to be modified and sections added or deleted to match the final physical and organisational arrangements following detailed design and construction.

This Emergency Action Plan (EAP) provides a systematic means of:

a. Defining and identifying Emergency Situations and/or Unusual Occurrences which may threaten the integrity of the dam and require immediate action.

b. Documenting the procedure for declaring an Emergency Situation.

c. Ensuring effective actions are taken to prevent dam failure.

d. Avoiding loss of life and minimising property damage in the event of a failure by providing timely warnings in a systematic way to the appropriate emergency management agencies for their implementation.

The responsibilities and actions of each organisation are outlined in the plan. It is intended that each organisation will keep this EAP readily available to assist staff in rapid decision making.

Preliminary warnings to Civil Defence and NZ Police are to be utilised wherever possible.

The priority of the plan is identified as requiring that, in any Emergency Situation, where a condition of a serious nature has developed that may endanger the integrity of the dam and/or downstream property or life, then the Future Dam Owner or Operator (FDOO) or its Agent must immediately notify:

Organisation	Name	Contact Numbers	
		Work	**Home**
NZ Police	Duty Supervisor	TBC	TBC
Civil Defence Controller	Headquarters	TBC	TBC

Special Note

If an Emergency Situation is declared and the NZ Police and Civil Defence cannot be contacted immediately on the above numbers, ring 111 and report the incident to NZ Police.

Table of contents

1 PURPOSE of EMERGENCY ACTION PLAN

This document provides an outline of a possible Emergency Action Plan (EAP) for the operational phase (as distinct from construction phase) of the proposed Lee River Dam. It is intended to provide an indication of the envisaged content for the operative EAP for the completed dam. Details will need to be completed and particular descriptions in this document will need to be modified and sections added or deleted to match the final physical and organisational arrangements following detailed design and construction.

1.1 Purpose

This Emergency Action Plan has been issued by the Future Dam Owner or Operator (FDOO) to provide a systematic means of:

a. Defining and identifying Emergency Situations and/or Unusual Occurrences which may threaten the integrity of the dam and require immediate action.

b. Documenting the procedure for declaring an Emergency Situation.

c. Ensuring effective actions are taken to prevent dam failure.

d. Avoiding loss of life and minimising property damage in the event of a failure by providing timely warnings in a systematic way to the appropriate emergency management agencies for their implementation.

This Emergency Action Plan sets out the responsibilities of all organisations associated with assessing potential emergency situations or unusual occurrences and implementing preventative actions.

1.2 Operative Period

This is ***an outline draft*** of the Emergency Action Plan. This Emergency Action Plan remains in force throughout the Operational Phase of the dam. It will be updated regularly with formal written notification of any amendments being circulated to each holder of a controlled copy.

The dam is considered to be in operation whilst its structure is being used to store water or regulate the flow of water in the stream.

2 RESPONSIBILITIES

2.1 General

This section specifies the organisations and persons responsible for the surveillance, maintenance and operation of the dam and the organisations and people responsible for implementing the Preventative and Emergency Actions detailed in Section 4.

2.2 Future Dam Owner or Operator

FDOO as the owner has a responsibility to operate the dam in a manner that is considered to meet sound engineering and professional standards, to meet all relevant legislative guidelines and in accordance with the Lee River Dam (LRD) Operating Procedures. These procedures are based on the New Zealand Society on Large Dams (NZSOLD) Guidelines and the LRD Resource Consent conditions.

From an emergency planning perspective FDOO is responsible for:

a. Providing advice in the preparation of this plan.

b. Complying with the detail of this emergency plan.

c. Ensuring that all the staff involved in the operation of the LRD are familiar with this plan, and the company obligations in it.

d. Ensuring that suitably trained and authorised staff are available to competently assess potential emergency situations and/or unusual occurrences. The staff must be familiar with this Emergency Action Plan. If in accordance with the guidelines contained in Section 3 and their professional judgement they identify an Emergency Situation or Unusual Occurrence they must discharge the responsibilities of FDOO throughout the period of the Emergency Situation or Unusual Occurrence.

e. Suitably trained staff are deemed to be those that can:

   - Recognise the Emergency Situations and/or Unusual Occurrences as listed in this plan, and understand their possible effects on the integrity and safety of the dam.

   - Understand that Emergency Situations and Unusual Occurrences in Section 3 are not an exhaustive list of every possible condition that could arise, and that judgements must be judiciously applied when assessing situations.

   - Acknowledge the importance of providing early warning to NZ Police and Civil Defence of any Emergency Situation and/or Unusual Occurrence that may develop at the dam site.

   - Accurately monitor, record and report on reservoir levels in relation to reservoir staff gauge and dam crest.

   - Accurately complete the Notification Form as shown in Appendix 1.

   - Readily access the emergency service numbers required to notify the NZ Police and Civil Defence (see contact list Appendix 3).

- Operate communication equipment used to convey emergency messages (e.g. telephone, fax and cellphone).
- Correctly interpret and manage the implementation of the preventative actions set out in Section 4 of this plan.
- Liaise with Tonkin and Taylor Ltd or other suitably experienced organisations where specialist advice is required. The acquisition of such advice must not delay the notification of an Emergency Situation or Unusual Occurrence.
- Safely supervise any of the operational tasks that may be necessary to remedy conditions which may be hazardous to the dam and/or downstream residents and properties.

f. Having facilities and procedures in place to receive alarm signals from monitoring equipment at the dam.

g. Having facilities and procedures in place to give warnings to Civil Defence and NZ Police in the event of emergency situations and unusual occurrences that may arise at the dam site.

h. Maintaining a schedule of the expertise, staff, materials and equipment to counter threats to the integrity of the dam.

i. Testing and maintaining the effectiveness of this Emergency Action Plan.

2.3 Civil Defence

The Tasman District Council (TDC) Civil Defence Organisation is affected by any emergency or unusual event that involves the LRD.

Civil Defence responsibilities, in relation to planning for emergencies at the dam, are those which pertain to local situations that could give rise to the need to declare an Emergency under the Civil Defence Act or require a coordinated multi-agency response to an emergency not declared under the CDEM Act.

The most important requirement of this plan is that it dovetails in to the Civil Defence plan currently in force for the Tasman District.

Civil Defence planning responsibilities can be summarised as:

a. Providing advice in the preparation of this plan.

b. Ensuring that this plan is compatible with the Civil Defence plan of the Tasman District Council.

c. Maintaining an easily accessed contact system to ensure they can receive early warnings, and keeping FDOO informed of any external events and/or information which may assist in assessing Emergency Situations and/or Unusual Occurrences at the dam site.

Maintaining their own plan for the handling of emergencies that may arise out of a sudden release of water from the LRD.

2.4 NZ Police

The NZ Police have responsibility for protecting the life and property of residents of this country. This responsibility extends across the spectrum of emergencies, from those that may affect only one individual, to major emergencies.

NZ Police responsibilities in relation to planning for Emergency Situations and/or Unusual Occurrences affecting the LRD are as follows:

a. Providing advice in the preparation of this plan.

b. Ensuring this plan is compatible with other Police plans and procedures in the Tasman District.

c. Having systems in place to allow receipt of reports of Emergency Situations and/or Unusual Occurrences, thus allowing the early implementation of Police procedures.

d. Liaising with Civil Defence on plans for the district relating to the handling of emergencies involving the LRD, in particular warning and/or evacuation procedures.

e. Maintaining a current contact list of all residents downstream from the LRD that may be immediately affected by a sudden release of water from the dam.

f. Establishing and maintaining a notification system for warning downstream residents, as well as Fire and Ambulance Services, in the event of an Emergency Situation and/or Unusual Occurrence developing at the LRD site.

2.5 NZ Fire Service and St John Ambulance

Both services will be informed by the NZ Police of any Emergency Situation and/or Unusual Occurrence at the earliest opportunity. Both services will in turn keep FDOO informed of any external event of which they have knowledge, which may affect the safety of the LRD.

The NZ Fire Service and St John will develop and maintain their own specific procedures relating to emergency situations and/or unusual occurrences which may result in sudden release of water from the LRD. Such plans and procedures will be known to, and be compatible with, all other agencies involved with emergencies pertaining to the LRD.

2.6 Tonkin & Taylor

As designer of the dam, Tonkin & Taylor is best placed to provide advice to FDOO in any Emergency Situation and/or Unusual Occurrence. Tonkin & Taylor will endeavour to provide advice regarding preventative actions that may need to be undertaken. If such advice is not available then other suitably qualified organisations will be utilised.

3 EMERGENCY IDENTIFICATION and EVALUATION

3.1 Notification of Potential Emergency Situations

The Operational Phase does not require permanent staff to be located at the dam site (TBC).

Therefore, apart from warnings derived from automatic sensors and control equipment at the dam, it is possible that if a potential Emergency Situation and/or Unusual Occurrence occurs it will be reported by a local resident or member of the public. The report is most likely to be received by FDOO, Civil Defence or the NZ Police. It is most important that any organisation receiving a report carries out its duties as set out in the Section 5 of this Emergency Action Plan.

FDOO staff on receipt of a message indicating a potential problem with the dam will, without delay, investigate the cause, instigate the necessary actions and provide preliminary warning to Civil Defence and NZ Police as per the notification requirements in Section 5 of this plan. It is the responsibility of FDOO to identify any incident as an Emergency Situation and/or an Unusual Occurrence. In the event that FDOO cannot be contacted, Civil Defence will decide whether the emergency is likely to lead to the declaration of a Civil Defence Emergency and declare an Emergency Situation or not. Emergency Situations and Unusual Occurrences at the dam are defined below.

3.2 Definitions of Emergency Situations

An **Emergency Situation** is a condition of a serious nature developing suddenly or unexpectedly that may endanger the integrity of the dam or downstream property and/or life, and requires immediate action. Emergency Situations include, but are not limited to:

a. Impending failure of the dam - this is the most serious emergency which will require immediate notification of the NZ Police and Civil Defence;

b. Imminent overtopping of the dam, for example due to excessive flood inflow, impaired spillway capacity, slope failure into reservoir;

c. Imminent overtopping or fusing of the auxiliary spillway fuse plug;

d. Failure or impending failure of the dam spillway;

e. Excessive seepage at new locations, or highly coloured seepage from the embankment, foundations, abutments or adjacent to the conduit;

f. Occurrence of an earthquake greater than the design earthquake;

g. Blockage of the spillway with the lake level rising.

3.3 Definitions of Unusual Occurrences

An **Unusual Occurrence** is an event which takes place, or a condition which develops, that is not normally encountered in the routine operation of the dam and may endanger its structure. These include but are not limited to:

1. High inflows to the reservoir or rainfall in the catchment area that exceed the rain gauge alarm levels;
2. Lake level rising above *x* m RL (*x* m above normal top water level (NTWL));
3. Slumping, cracking or erosion of the dam or its abutments;
4. New springs, seeps, boggy areas or increased drainage;
5. An increase in, or a murky appearance of, the seepage from the dam;
6. If significant material is being eroded by water a breach may ultimately develop;
7. Erosion of or damage to riprap due to storm wave action;
8. Loss of freeboard;
9. Failure or impending failure of the downstream weir structures;
10. Total loss of remote indications and alarm communications with the dam;
11. Operating occurrence outside specific guidelines in the current version of the Lee River Dam Operating Procedures;
12. Incident associated with the dam operation likely to have a significant impact on the environment downstream of the dam;
13. Unusually high winds;
14. Blockage of the primary or auxiliary spillway at any time (other than an Emergency Situation as defined in Section 3.2);
15. Any sudden change in the reservoir and its immediate surrounds.

4 ACTIONS DURING AN EMERGENCY SITUATION OR UNUSUAL OCCURENCE

4.1 Emergency Action Plans - General

Each organisation involved in the LRD emergency planning will have their own internal policies and procedures. These will determine their own actions in the event of an emergency.

The intent of this plan is to outline the actions to be taken by FDOO when dealing with Emergency Situations and/or Unusual Occurrences at the dam site. These action plans are not exhaustive of every possible condition, and judgements by FDOO and design advice from Tonkin & Taylor or other suitably qualified organisations will be necessary in other situations.

The primary action is to notify the NZ Police and Civil Defence if there is <u>the potential</u> for an uncontrolled release of water from the LRD.

4.2 Emergency Action Plans - Specific

In preparation for possible remedial action FDOO will at all times operate the dam in accordance with approved operating procedures and this Emergency Action Plan.

Adequate, suitably trained staff will also be available on a 24-hour basis to respond quickly in the event of any emergency situation or unusual occurrence being reported and to supervise necessary remedial work.

In response to the following listed Emergency Situations and/or Unusual Occurrences, the following actions will be carried out by FDOO. Additional actions may be undertaken if deemed necessary or appropriate.

4.2.1 Impending breach of the dam

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Arrange for the placement of sandbags and/or riprap material in the breach if safe to do so.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.2 Imminent overtopping of dam

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.
- Consider breaching the auxiliary spillway fuse plug if not already breached by rising water.

- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.3 Excessive seepage, or excessively discoloured seepage

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Attempt to plug the reservoir side of leak with whatever suitable material is available (e.g. hay bales, plastic sheeting, gravel, etc.) if it is safe and practical to do so.
- If practical and safe, place protective sand and gravel filter over the exit area to hold materials in place.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.4 Failure or impending failure of the dam spillway

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.5 Primary spillway blockage with lake level rising

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Investigate methods of safely removing part or all of the blockage.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.6 Overtopping or fusing of the auxiliary spillway (imminent or actual)

- Immediately warn Civil Defence and NZ Police of the emergency situation in accordance with the Priority Notification Plan in Section 5 of this plan.

- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.2.7 Earthquake

- Ensure Civil Defence has been notified of the earthquake and issue a preliminary warning to them and NZ Police in accordance with the Priority Notification Plan in Section 5 of this plan.
- Observe reservoir levels and river flows below the dam for indications of a potential dam breach. If such a condition is indicated then immediately implement the Emergency Actions for Impending Dam Breach.
- Contact Tonkin & Taylor.
- Immediately inspect dam.
- If dam is damaged to the extent that there are increased flows or new flows passing downstream, then immediately implement action plans for Impending Breach of the Dam.
- Warn NZ Police and Civil Defence in accordance with the Priority Notification requirements shown in Section 5 of this plan.
- If visible damage has occurred, but is not serious enough to cause failure of the dam, then monitor the location and nature of damage.
- Provide preliminary warning to Civil Defence and NZ Police as per Priority Notification Plan in Section 5 of this plan.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.3 Unusual Occurrence Action Plans

These situations do not represent an immediate danger to the dam and therefore will not endanger property or lives downstream of the dam. Nevertheless, wherever possible or necessary, a preliminary warning should be issued to the Civil Defence and NZ Police in accordance with the Notification requirements shown in Section 5 of this plan. The timely provision of early warnings can avert disasters.

4.3.1 Lake level rising above x m above NTWL

- Provide a suitably trained observer at the dam site who can accurately monitor and report on the situation.
- Test communication systems in place and advise Civil Defence and NZ Police of the situation at the dam site.
- Monitor rainfall and reservoir level gauge.
- Consult with TDC regarding data from TDC monitoring gauges and Meteorological Services information.
- Alert operations staff to the situation.

If the spillways block or overtopping is imminent, then follow the appropriate specific Emergency Action Plans in Section 4.2.

4.3.2 Slumping, cracking or erosion of the dam or its abutments

- If the slumping is on the downstream face of the dam or a breach, or overtopping of the dam is imminent, implement the Emergency Situation action plan and immediately warn Civil Defence and NZ Police of the Emergency Situation and/or Unusual Occurrence as per the Priority Notification Plan shown in Section 5 of this plan.
- If the slumping is on the upstream face of the dam and a breach or overtopping are not imminent, then provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Contact Tonkin & Taylor to determine possible remedial actions.

4.3.3 New springs, seeps, boggy areas or increased drainage

- Provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Investigate the reason for the change in seepage from the dam. If a specific area is suspected then plug the reservoir side with whatever suitable material is available (e.g. hay bales, plastic sheeting gravel, etc) and place a protective sand and gravel filter over the exit area to hold materials in place.
- Contact Tonkin & Taylor for advice on possible further remedial action.

4.3.4 An increase or a murky appearance to the seepage from the dam

- Provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
- Commence lowering the water level in the reservoir by fully opening the outlet valve.
- Investigate the reason for the change in seepage from the dam. If a specific area is suspected then plug the reservoir side with whatever suitable material is available (e.g. hay bales, plastic sheeting gravel, etc) and place a protective sand and gravel filter over the exit area to hold materials in place.
- Contact Tonkin & Taylor for advice on possible further remedial action.

4.3.5 Erosion of or damage to riprap due to storm wave action / loss of freeboard

- Warn NZ Police and Civil Defence in accordance with the Priority Notification Plan in Section 5 of this plan.
- Place additional riprap or sandbags in damaged area to prevent further embankment erosion.
- Commence lowering the water level in the reservoir by fully opening the outlet valves.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Restore freeboard with sandbags or rockfill.

4.3.6 Blockage of the auxiliary spillway with the lake level below z m RL

- Warn NZ Police and Civil Defence in accordance with the Priority Notification Plan in Section 5 of this plan.
- Prepare machinery/equipment and unblock auxiliary spillway.
- Monitor reservoir level closely for possible rise of water and if rising then fully open the outlet valve.
- If unable to clear the blockage, prepare to counter overtopping of dam.
- If overtopping of the dam is foreseeable, implement the Emergency Situation action plan for overtopping.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.3.7 Total loss of remote indications and alarm communications with dam

- Warn NZ Police and Civil Defence in accordance with the Priority Notification Plan in Section 5 of this plan.
- Locate a suitably experienced person with communications equipment at the dam site to carry out local operation of the dam.
- Restore controls.
- Contact Tonkin & Taylor for advice on possible further remedial action.
- Continue to liaise with, and provide information to, Civil Defence and NZ Police.

4.3.8 Operating outside specific guidelines in the Dam Operating Procedures

- A suitably experienced person is to carry out an inspection of the situation.
- Provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.

- If the Plan can continue to be operated safely then a written revision to the operating procedures will be approved by FDOO and issued. Without such a written authorisation the plan will not be operated except as required to maintain its safety.
- If necessary, seek advice from Tonkin & Taylor.

4.3.9 Incident likely to have significant impact on downstream environment

- A suitably experienced person is to carry out an inspection of the situation.
- Provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
- If the Plan can continue to be operated safely then a written revision to the operating procedures will be approved by FDOO and issued. Without such a written authorisation the plan will not be operated except as required to maintain its safety.
- If necessary, seek advice from Tonkin & Taylor.

4.3.10 Unusually high winds

- Inspect dam for storm wave damage to the riprap or auxiliary fuse plug.
- If damage is apparent then provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan and proceed as per the appropriate unusual occurrence.
- Inspect the rest of the dam structures for damage.
- If necessary, seek advice from Tonkin & Taylor.
- Continue to liaise with and provide information to Civil Defence and NZ Police.

4.3.11 Blockage of spillways at any time (other than emergency)

- A suitably experienced person is to carry out an inspection of the situation.
- If the blockage cannot be immediately cleared then:
  - provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
  - Commence lowering the water level in the reservoir by fully opening the outlet valve located in the valve chamber.
  - Water level lowering may be increased by the use of high capacity diesel powered pumps if these are available.
  - Contact Tonkin & Taylor on possible further remedial action.
  - Contact a suitable Contractor to clear the blockage subject to advice from Tonkin & Taylor.

4.3.12 Sudden change in reservoir and surrounds

- A suitably experienced person is to carry out an inspection of the situation.
- Provide preliminary warning to Civil Defence and NZ Police as per the Notification Plan.
- If the Plan can continue to be operated safely then a written revision to the operating procedures will be approved by FDOO and issued. Without such a written authorisation the Plan will not be operated except as required to maintain its safety.
- If necessary, seek advice from Tonkin & Taylor.

5 COMMUNICATION PLANS and PROCEDURES

5.1 Communication Responsibilities

5.1.1 The future dam owner or operator (FDOO)

The future dam owner or operator has primary responsibility for the following communications requirements:

a. As a priority, to immediately warn Civil Defence and the NZ Police of any Emergency Situation and/or Unusual Occurrence that develops at the dam site in line with the Priority Notification requirements of this plan.

b. Such advice to be given verbally, then confirm by faxing the completed Notification Form as shown in Appendix 1.

c. To assist this communication requirement, FDOO will maintain effective 24-hour telephone, fax and cellphone contact capability as shown in the Contact List in Appendix 3.

d. FDOO will also need to consult with Tonkin & Taylor as designers of the dam, in the event of an Emergency Situation and/or Unusual Occurrence.

e. In the event of any incident at the dam site FDOO are responsible for advising the insurers of incident details.

f. FDOO will maintain an arrangement with the Meteorological Service to be provided with heavy rainfall warnings.

5.1.2 Civil Defence

24-hour Civil Defence contacts are maintained by Tasman District Council.

Civil Defence staff from the Council will advise FDOO of any external event known to them, and considered to pose a possible threat to the LRD (e.g. information from the Meteorological Service, is a prime example of data made available to Civil Defence, that could assist FDOO in planning and decision-making).

The Civil Defence Organisation will have systems in place to allow easy contact from FDOO, or any other agency or individual wishing to advise of emergency situations or unusual occurrences relating to the LRD.

5.1.3 NZ Police

The NZ Police are an essential service who will quickly become involved in any emergency, which threatens life or property. FDOO will therefore keep the Police informed of Emergency Situations and/or Unusual Occurrences relating to the LRD.

The Police will keep FDOO informed of any external events known to them, which may create an emergency situation at the dam site. Contact details for FDOO are shown in Appendix 3 of this plan.

On receipt of notification of any Emergency Situation and/or Unusual Occurrence the Police will immediately inform the NZ Fire Service and St John Ambulance Service of the details of the incident and ensure that Civil Defence have been notified. Contact details for these organisations are shown in the Contact List in Appendix 3.

The Police will maintain their own up to date contact list of all properties/persons downstream of the LRD and liable to be immediately affected by any sudden release of water from the dam. This list is shown in Appendix 4. *<This process may need to be modified/changed due to the large number of residents that would need to be contacted.>*

It is the responsibility of the Police to advise all such persons of any danger resulting from an Emergency Situation and/or Unusual Occurrence at the LRD site.

5.1.4 NZ Fire Service and St John Ambulance

Both organisations will maintain easily accessible contact systems to allow receipt of warnings from the NZ Police.

Contact details for these organisations are shown on Section 5 of this plan.

5.1.5 Tonkin and Taylor Ltd

As designers of the Lee River Dam, Tonkin & Taylor will need to be available for consultation, or to give advice to FDOO in relation to any Emergency Situations and/or Unusual Occurrences involving the dam.

They must therefore maintain a contact list of persons capable of giving such advice. Details of these contacts are shown in Appendix 3.

5.2 Communication Systems

The following communication systems are available to be utilised throughout any Emergency Situation and/or Unusual Occurrence.

- A Telecom fixed landline system with a fax and a phone at the FDOO manager's office and connected to all other telephone networks in New Zealand.
- Any personnel sent to the site under this EAP will have access to a mobile telephone network with connections to all other telephone networks. *<Check that the dam site has cellphone coverage, or if there will be coverage in the future?>*

FDOO will ensure that these communication systems are maintained and remain operable as far as is reasonably possible.

The relevant voice and fax communication telephone numbers are detailed in Appendix 3.

5.3 Notification Procedure

5.3.1 Priority notification

In the event of an Emergency Situation and/or Unusual Occurrence FDOO will, as a priority, give warnings to the following 2 agencies:

NZ Police

Duty Supervisor Ph *TBC*

Tasman Civil Defence

(Civil Defence Manager) Office Ph *TBC*

Special Notes

If dam collapse is imminent, or an Emergency Situation occurs, and the NZ Police and Civil Defence cannot immediately be contacted on the above numbers, ring 111 and report the incident to NZ Police.

The key information that needs to be supplied is given in the Notification Format below. Contacts will be made in accordance with the Notification Flow Chart in Appendix 4. An Emergency Contact List is provided in Appendix 3.

Notification Format

When reporting to other services (e.g. Civil Defence, NZ Police, FDOO or Tonkin & Taylor), the report will convey the following information:

a. Name of person making report and the organisation they represent.

b. Name of dam and location.

c. Description of problem.

d. Location of problem:

  - in relation to the embankment (e.g. halfway up from toe),
  - in relation to the dam crest (e.g. 60 metres left of the spillway),
  - in terms of what part of the dam is affected (e.g. upstream slope, downstream slope or crest).

e. An estimate of the quantity of any unusual flow, as well as a description of flow quality (e.g. clear, cloudy, muddy, etc).

f. A reading of the reservoir level.

g. An indication of whether the water level is rising, stable or falling.

h. The current weather conditions at the site.

i. An indication of whether the situation appears to be worsening, remaining stable, or improving.

j. An indication of whether the situation appears able to be contained or not.

k. Anything else that the caller feels is important.

This information must also be passed immediately to FDOO.

The message is to be confirmed by faxing a completed copy of the Notification Form shown in Appendix 1.

6 SITE ACCESS

6.1 Site Access by Road

This section will be developed when site specific details are available.

6.2 Site Access by Other Means

This section will be developed when site specific details are available.

Figure 6- LRD Regional Map

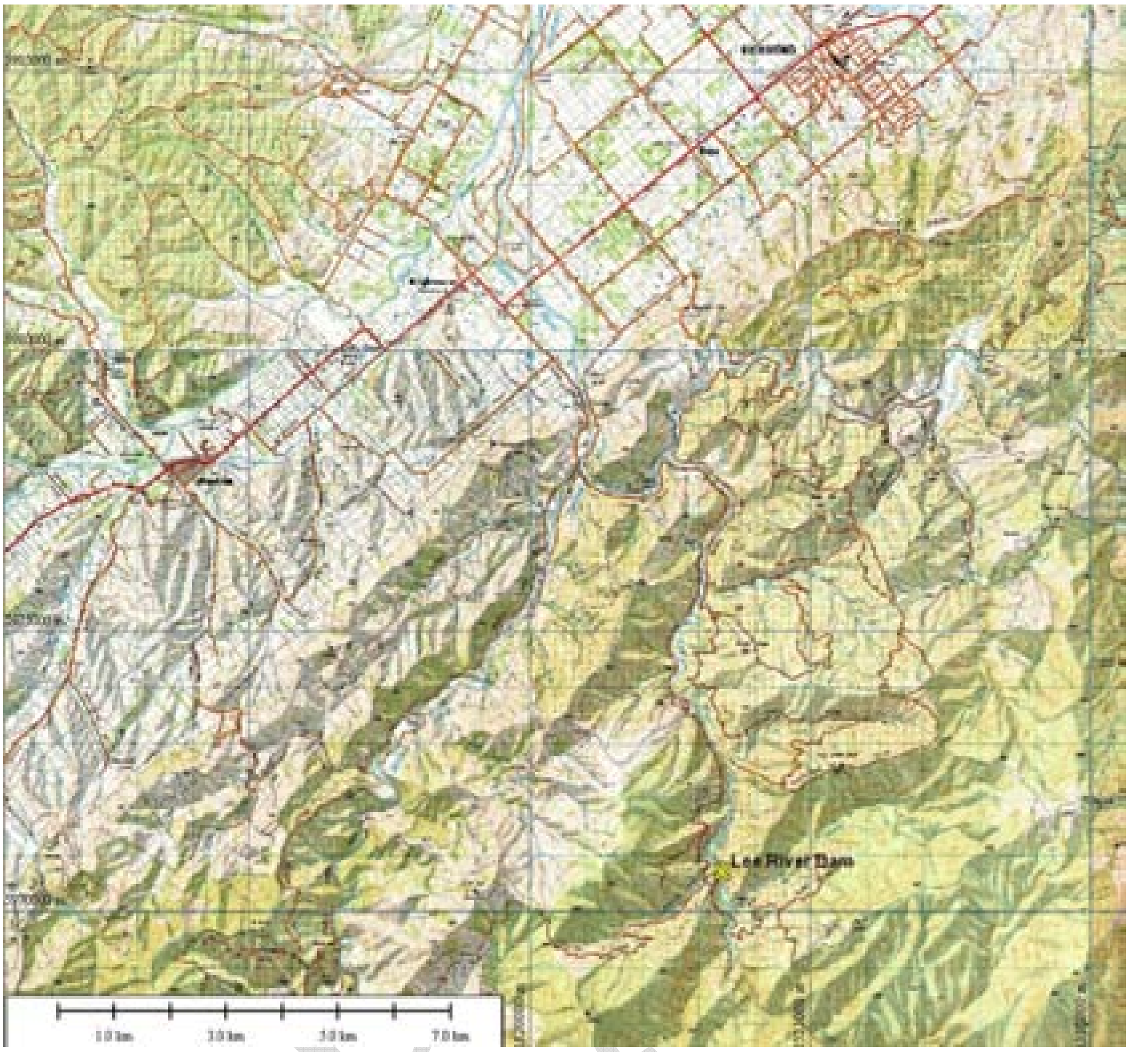


Figure 6- LRD Local Map

7 RESPONSE DURING DARKNESS OR ADVERSE WEATHER

7.1 Access

The relatively remote location of the LRD to urban centres means that it is probable that access to the site will be unavailable in the event of extreme weather (*TBC*).

In the event that access cannot be gained in a timely manner then contact should be made with residents close to the dam and their untrained services should be utilised if required.

7.2 Work at the site

The location of all staff involved in investigating or monitoring potential or actual Emergency Situations and/or Unusual Occurrences at the dam should wherever possible be known to another responsible person at all times.

Wherever possible two people should attend to an Emergency Situation and/or Unusual Occurrence at the site during periods of adverse weather. Contact must be established and regularly maintained with an external party at no more than one hourly intervals.

FDOO will ensure that suitably trained staff will be available to cope with all reasonable activities required under this Emergency Action Plan and the Operating Procedures under foreseeable weather conditions.

FDOO will ensure that suitable equipment and information (including a copy of this Emergency Action Plan) and the routine test and inspection records are kept at the site.

Site lighting may not be working at the dam, especially during or following a natural hazard event. Therefore, for action during periods of darkness staff should use vehicle headlights and take battery and vehicle operated spotlights to site.

Communications from the site could be significantly more difficult during periods of adverse weather. It is therefore important that all the systems are regularly checked throughout any Emergency Situation and/or Unusual Occurrence and that care is taken to ensure all messages are correctly received.

8 SOURCES OF SPECIAL EQUIPMENT and MATERIAL

8.1 Special Equipment

Special equipment in the form of earthmoving plant may be required under certain Emergency Situations and/or Unusual Occurrences. This plant is large, slow to move and therefore due allowance must be made for the time it will take to reach the site. Wherever possible equipment located in the vicinity of the dam should be utilised. Civil Defence has the right to commandeer equipment in the event of a civil emergency and therefore close cooperation should be maintained with the Tasman Civil Defence.

Earthmoving equipment required can be sourced from these suppliers:

Company: ……
Address: ……
Telephone: ……

Company: ……
Address: ……
Telephone: ……

8.2 Supplies and Materials

Riprap, sandbags and other construction materials can be sourced from the following local suppliers:

Company: ……
Address: ……
Telephone: ……

Company: ……
Address: ……
Telephone: ……

8.3 Emergency Power Supplies

Direct power supply is normally available at the dam site. However, if required, a generator can be sourced from:

Company: ……

Address: ……

Telephone: ……

Company: ……

Address: ……

Telephone: ……

9 SUPPORTING INFORMATION

9.1 Dam Details - General

Lee River Dam is a 52 m high concrete faced rock fill embankment with a crest length of 220 m. At NTWL of 197 m RL the reservoir stores approximately 13 million cubic metres of water.

Table 9-1 presents the key dam levels for the LRD.

Table 9-1 Key Dam Levels (Provisional)

Level (m RL)	Description
197.00	Normal Top Water Level (NTWL), operating level at spillway weir
201.077	Operating level at 1 in 200 year flood
201.577	Operating level for the probable maximum flood (PMF)
202.00	Effective crest level (including parapet wall of 1.0 m)

Figure 9-1 below presents the reservoir water storage versus water elevation relationship.

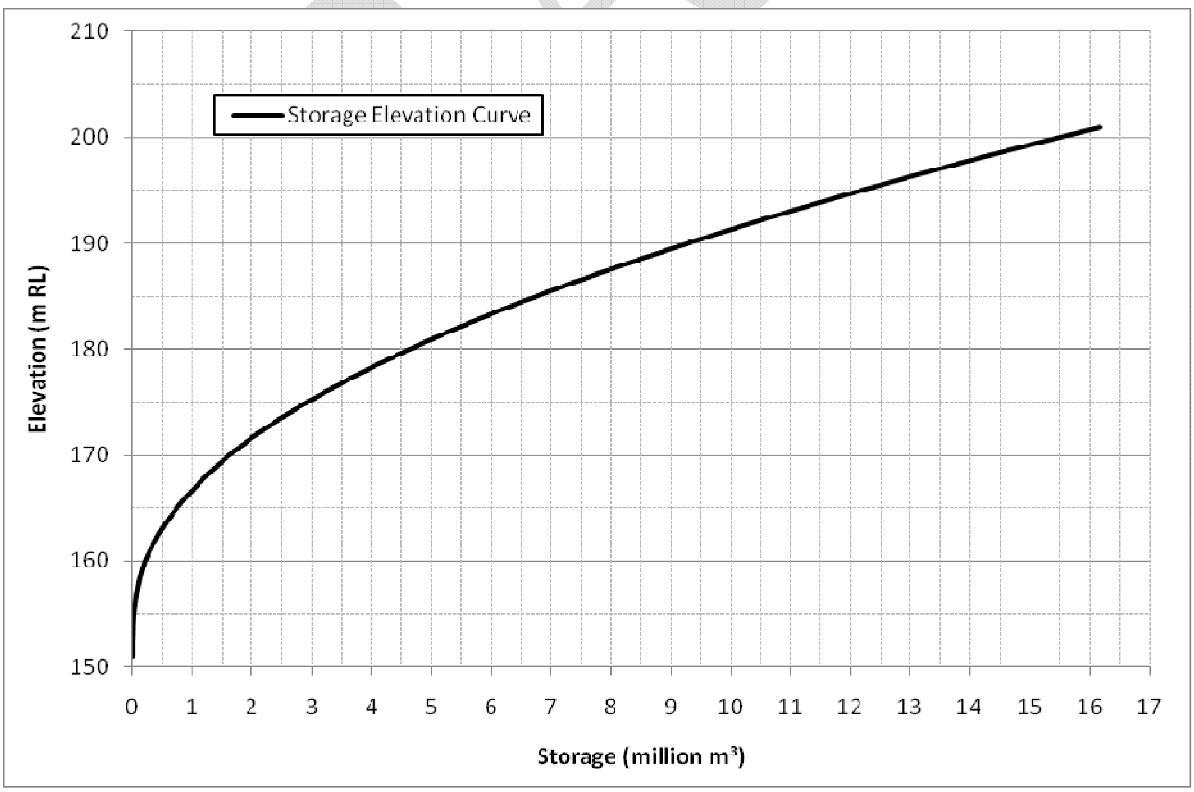


Figure 9-1 Reservoir Storage versus Elevation Relationship Curve

Figure 9-2 shows the rating curve (flow rate versus reservoir level) for the primary spillway (ogee weir).

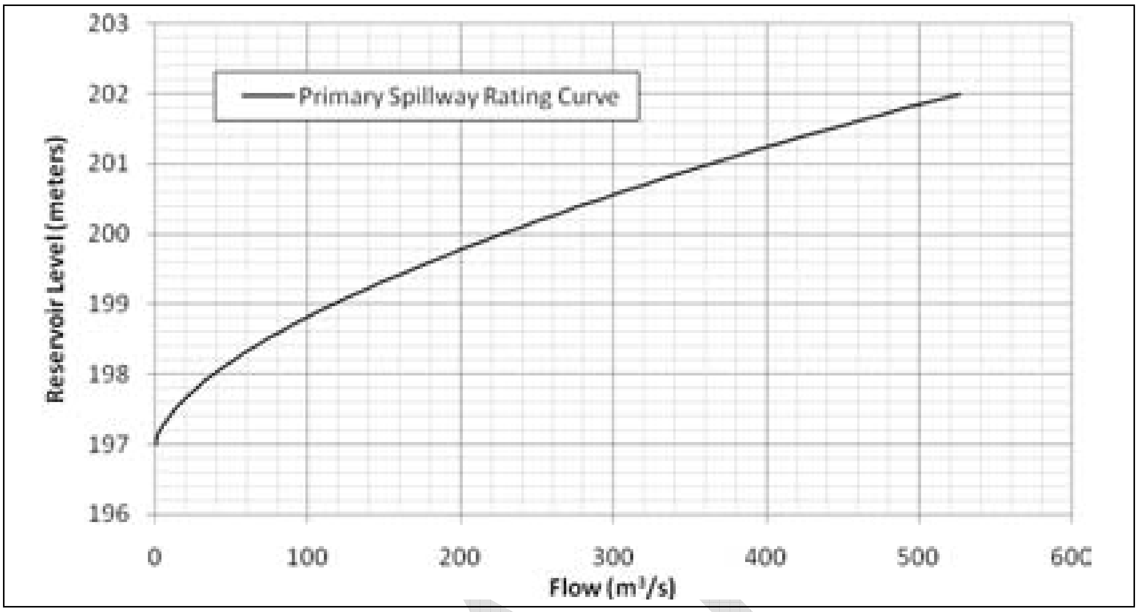


Figure 9-2 Spillway Rating Curve

Figure 9-4 shows the rating curve (flow rate versus reservoir level) for the auxiliary spillway.

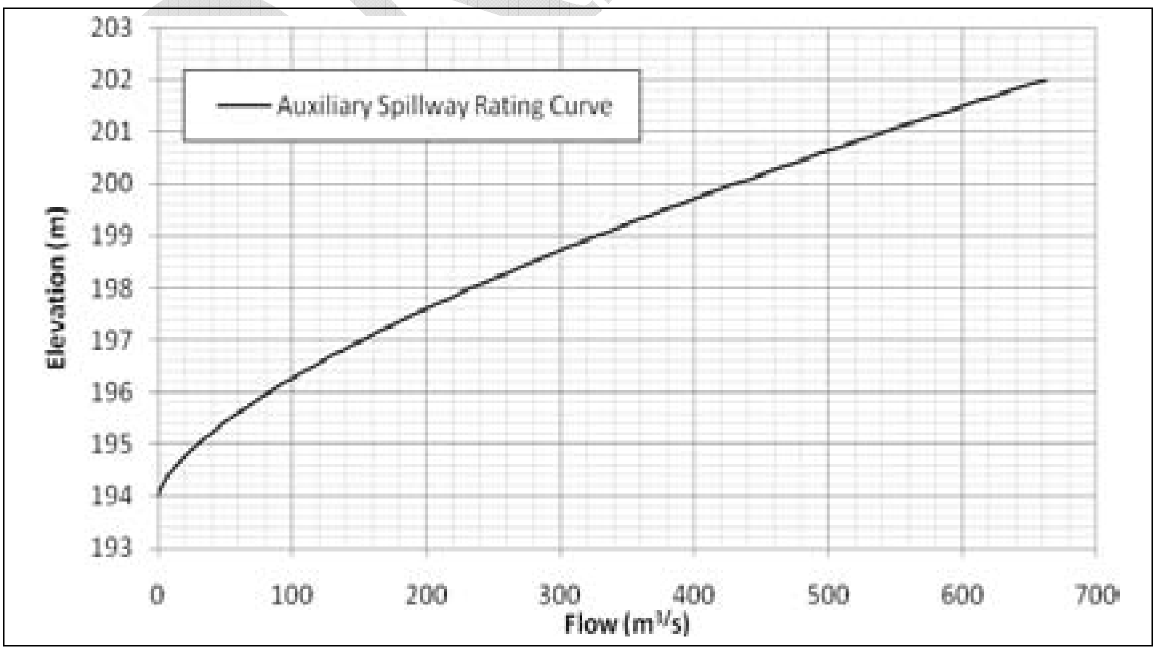


Figure 9-3 Spillway Rating Curve

Figure 9-4 shows the location of the monitoring instrumentation for the dam. The instrumentation comprises:

- *List to be developed and figure to be inserted when details available*

[Figure 9-4 to be inserted]

Figure 9-4 Instrumentation locations

Table 9-2 presents the expected spillway flows and maximum reservoir levels for the 100, 1,000 and 10,000 year return period rainfall events.

Table 9-2 Predicted Spillway Flows and Reservoir Levels (Provisional)

Rainfall event	Maximum spillway flow rate (m^3/sec)	Maximum reservoir level (m RL)
200 year return period	372	201.077
PMF	1055 (including auxiliary spillway flow of 606 m^3/s)	201.577

9.2 Flood Inundation Mapping

A dam break analysis has been undertaken to map the potential inundation area in the event of a dam break event. Such studies are hypothetical in nature, and entirely divorced from the remote chances of a dam failure ever occurring. The results are used to review downstream hazard potential and provide information for emergency management planning purposes.

The results of the simulated dam break scenarios and assumptions made in the analysis are detailed in the Dam Break Analysis and Hazard Assessment report (Tonkin & Taylor 2009).

The elapsed time from dam breach initiation until the first wave arrives (warning time) and the elapsed time to the peak water depth are given at specific locations downstream from the LRD in Table 9-3.

Two figures below represent the inundation area. Figure 9-5 shows the maximum predicted flood extent (flooding >0.5 m depth) based on a 0.9 hour breach formation time. Figure 9-6 is a plot of the "dv" parameter which is explained later.

Table 9-3 Peak flow during dry weather failure

River Chainage (m)	Location	Scenario A (0.5 hr breach formation time)		Scenario B (0.9 hr breach formation time)		Scenario C (1.5 hr breach formation time)	
		Time for flood-wave to first arrive	Time for peak water depth to occur	Time for flood-wave to first arrive	Time for peak water depth to occur	Time for flood-wave to first arrive	Time for peak water depth to occur
0	Lee Dam	**T=0 min**	-	**T=0 min**	-	**T=0 min**	-
2910	Lucy Creek confluence	+8 min	+23 min	+10 min	+36 min	+13 min	+52 min
8220	Fairdale	+16 min	+27 min	+22 min	+40 min	+28 min	+57 min
12720	Wairoa River confluence	+19 min	+33 min	+26 min	+38 min	+37 min	+66 min
16470	State Highway 6 bridge at Brightwater	+31 min	+47 min	+40 min	+62 min	+48 min	+80 min
20330	Wai-iti River confluence	+46 min	+74 min	+54 min	+89 min	+63 min	+110min
24220	Coastal Highway (SH60) bridge	+68 min	+102min	+78 min	+116min	+88 min	+135min

The 'population at risk' is defined as those people affected by flood water depths greater than 0.5 m. The flood extents with maximum flooding depth greater than 0.5 m for the 0.9 hour breach scenario is shown in Figure 9-5.

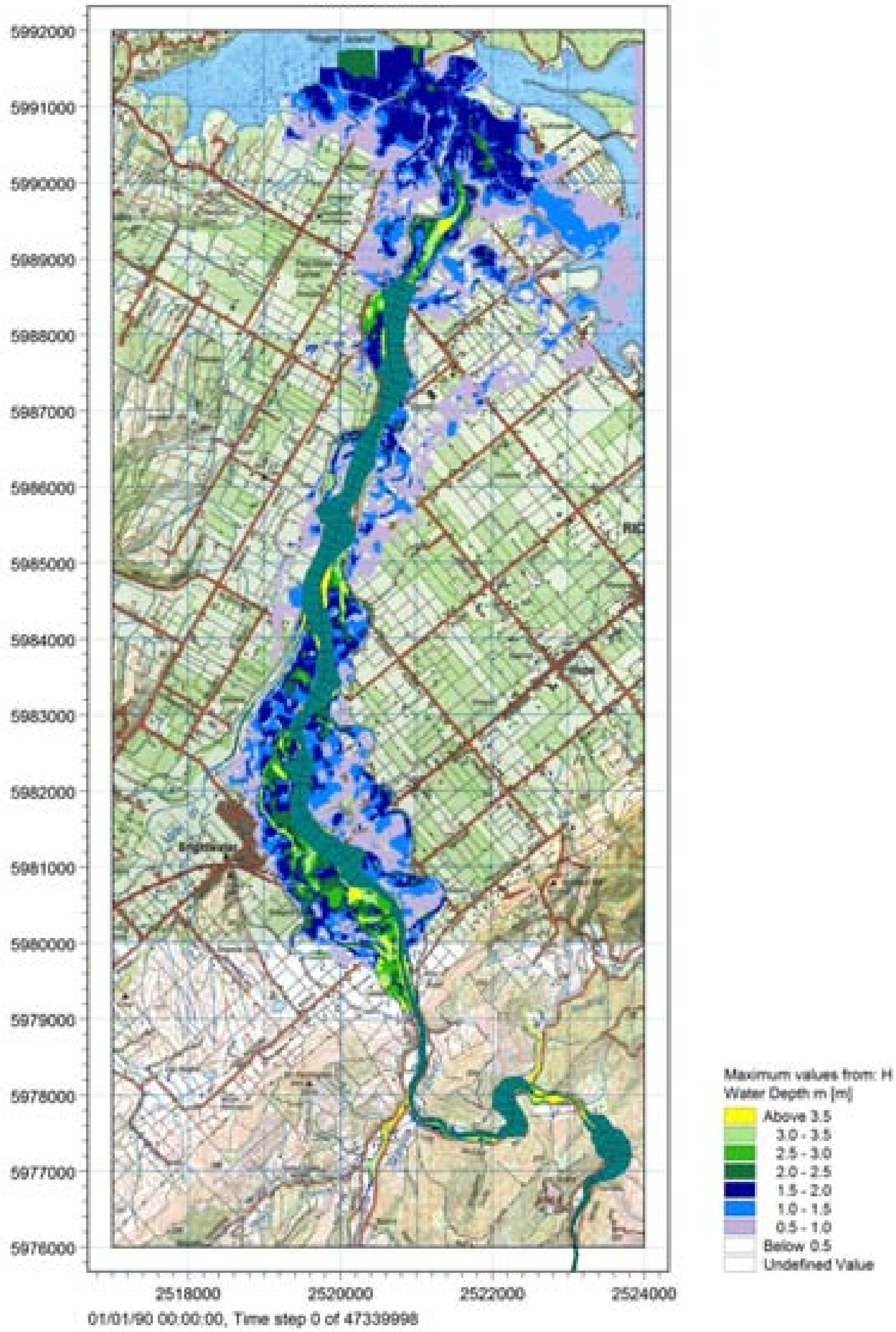


Figure 9-5 Maximum flood extent (>0.5m depth) for 0.9 hour breach progression time

A paper by Amos *et. al* presented at the 2004 ANCOLD/NZSOLD conference discussed the application of depth and velocity dam break parameters in assessing the "danger to life" due to a dam failure. It refers to a parameter known as "*dv*" (depth multiplied by velocity).

This parameter is used to assess the risk to life according to the following:

$dv < 0.5$	No danger to life
$0.5 < dv < 1.0$	Some danger to life exists
$dv > 1.0$	Danger to life significant

The inundation area and the distribution of *dv* is shown in Figure 9-6. The highest risk to life and buildings in the unlikely event of a failure of the LRD is the area to the east and north of Brightwater.

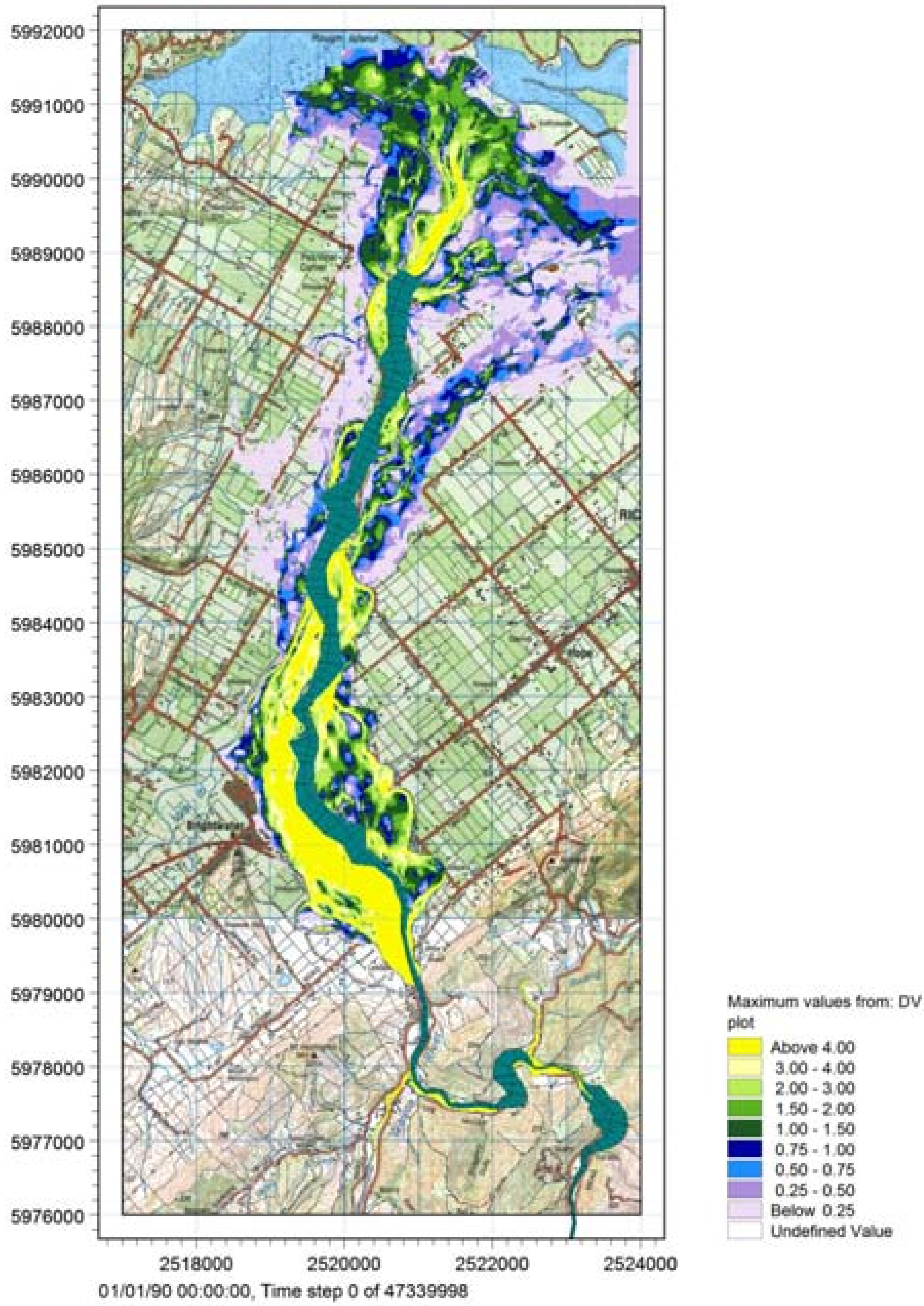


Figure 9-6 Assessment of inundation area based on dv (depth multiplied by velocity)

10 PROCEDURE FLOW CHARTS

Provided on the following pages are flow charts outlining procedures to take in the event of an Emergency Situation and/or Unusual Occurrence, namely:

i. Steps to take in a storm event – *to be developed as appropriate*

ii. Steps to take in an earthquake or a report of unusual activity – *to be developed as appropriate*

iii. Steps to take for damage control – *to be developed as appropriate*

EMERGENCY NOTIFICATION FORM

1. NAME/Organisation……………………………. 2. DATE……… 3. TIME………..

4. DAM NAME: LEE RIVER DAM 5. LOCATION: NZMG Map ref. 5970700 mN, 2523431 mE

6. PROBLEM

6.1. Description ………………………………………………………………………………..

6.2. Area of dam ……………………………………………………………………………….

7. UNUSUAL FLOW

7.1 Quantity ………………………………………………………………………………

7.2 Quality ………………………………………………………………………………………..

7.3 Colour …………………………………………………………………………………………

8. WATER LEVEL

8.1 Reservoir level ………………………………………………………………………………

8.2 Rising/stable/falling ………………………………………………………………………..

9. GENERAL COMMENTS

9.1 Weather conditions …………………………………………………………………………

……………………………………………………………………………………………………….

9.2 Situation – improving/stable/worsening ……………………………………………..

……………………………………………………………………………………………………….

9.3 Other comments : …………………………………………………………………………

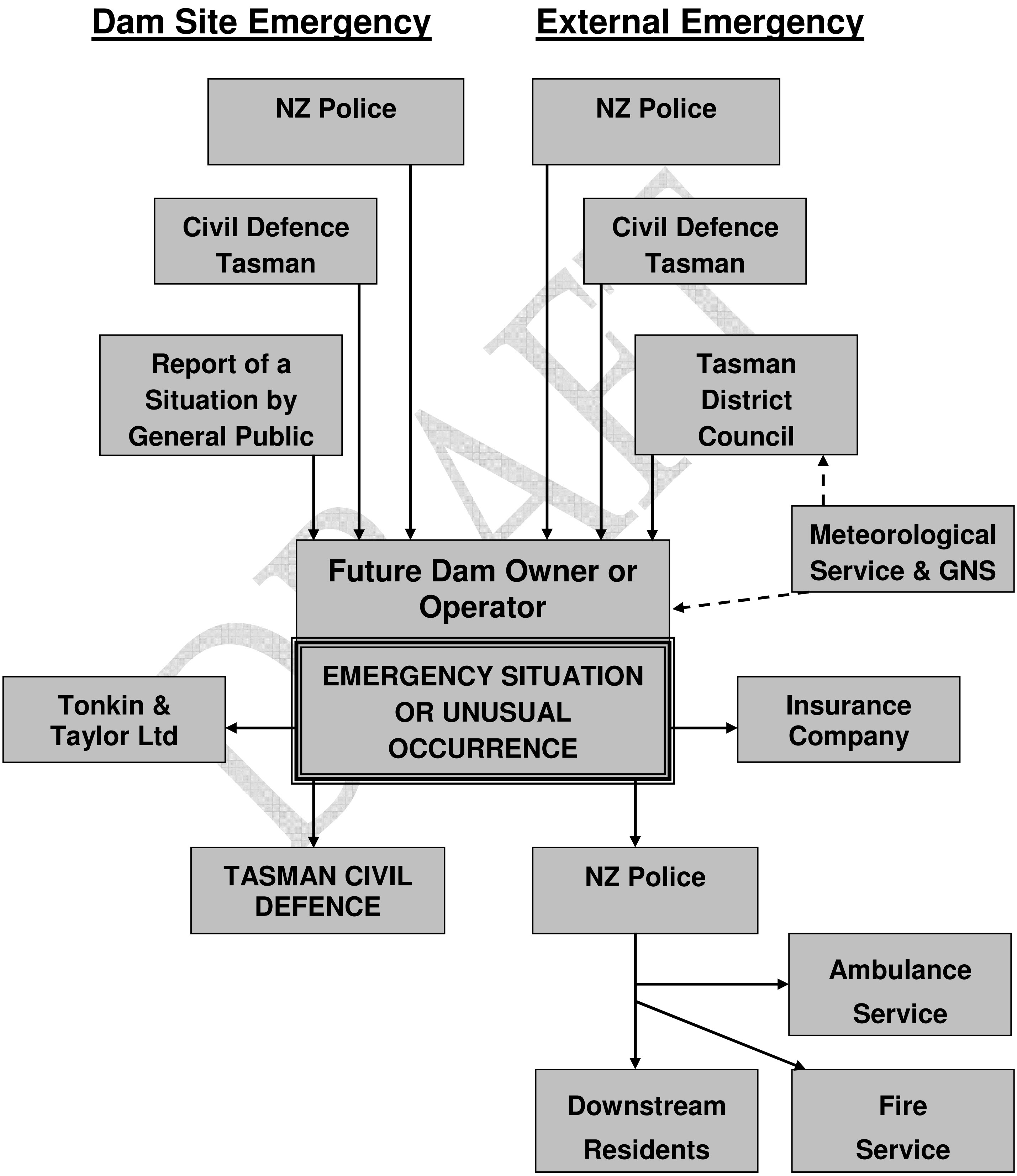
……………………………………………………………………………………………………….

10. NOTIFICATION CHECKLIST (tick when done, record time and name)

Future dam owner or operator	Time ……………. ☐	Name …………………..……
Tasman Civil Defence	Time ……………. ☐	Name ……………………..…
NZ Police	Time ………….. ☐	Name ………………………..
Tonkin and Taylor Ltd	Time ………..…. ☐	Name ………………………..

SIGNATURE: ……………………………………………………………………….

Appendix 2 - Notification Flow Chart



Appendix 3 - Contact List

ORGANISATION	LOCATION	NAME	OFFICE PH	OFFICE FAX	OUTSIDE OFFICE	PAGER/ MOBILE
Future Dam Owner or Operator						
Tasman Civil Defence						
NZ Police						
St John Ambulance						
NZ Fire Service						
Tonkin & Taylor						

Appendix 4 - Contact List – Downstream Residents

Initial Contact	Address	Rapid No.	Phone	Fax	Mobile Ph No.

Note:

The Police are responsible for updating and amending this list. Any amendments to the List will be circulated as received from the Police.

Appendix F: Peer Review Reports

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

17 December 2007

Tonkin and Taylor Ltd
P O Box 5271
AUCKLAND

RECEIVED
13 MAR 2008
BY: SLM

Attention: Robin Dawson

Dear Robin,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.1

Engineering Geology Ltd (EGL) have been engaged as a sub-consultant by Tonkin and Taylor Ltd (T&T) to provide external peer review of the Phase 2 geotechnical investigations and design of the proposed dam that will create a water storage reservoir on the Lee River. The water storage reservoir is being developed for the Waimea Water Augmentation Committee (WWAC). The scope of external peer review is outlined in Appendix A of the Services Agreement.

The objects of the peer review are to:

i) carry out peer review of preliminary geotechnical mapping and engineering design work associated with selecting the most appropriate location for the dam within the subject reach of river, and the selection of the most appropriate type of dam (including site visit)

ii) carry out peer review of the proposed second and third stage geotechnical investigations

iii) carry out peer review of the Stage 2 geotechnical report

iv) carry out peer review of the Stage 3 geotechnical report

v) carry out peer review of the preliminary dam design, cost estimate and associated reporting.

This is the first external peer review report by EGL, and covers the first objective listed above. Peer review has consisted of a site inspection by Trevor Matuschka on 28 November 2007 and review of the following documents:

- section 5 Methodology from the T&T proposal for Phase 2 studies
- Geology of the Lee River Catchment in the Environs of Potential Dam Sites (T&T Ref. 24727.200, 10 December 2007 draft).
- Project Feasibility Study Optimisation of Dam Location and Type (T&T Ref. 24727.301, 10 December 2007 draft)

Peer Review comments on the work to date follow:

1. Site Inspection

An inspection of the seven potential dam sites identified between CH10500 and CH12400 was undertaken by Trevor Matuschka on 28 November 2007. The inspection was undertaken in company with Mark Foley (Project Manager – T&T). Prior to the site inspection a briefing was provided on the geology of the area, the scope of and interpretation of geological investigations to date and implications for dam design. The various dam sites that are being considered and potential borrow areas and materials were inspected during the site visit.

2. Geological Investigations

The results of geological and engineering geological studies and a preliminary assessment of the geotechnical risks are documented in the T&T report 'Geology of the Lee River Catchment in the Environs of Potential Dam Sites' (Ref. 24727.200, 10 December 2007). Some investigations were undertaken as part of the Phase 1 (pre-feasibility) investigation, but they have been extended upstream as part of the Phase 2 study. This work constitutes Stage 1 of a three stage geotechnical investigation for the Lee Valley Dam.

We consider that the scope of investigations are appropriate for this stage of the study. The T&T report provides a good summary of the regional and local geology and seismic hazard of the site. It also includes assessment of geotechnical issues affecting the selection of the preferred dam site and the design of the various types of dams that are being considered. The geotechnical issues include slope stability, foundation conditions, assessment of the construction materials and seismic risk.

A summary of the geotechnical conditions as they are currently understood, at the seven potential dam locations, is provided. The studies to date indicate that the depth of weathering is shallow and bedrock would be expected to have acceptably low permeability, except at shallow depth where some grouting maybe required where rock relaxation may have occurred. Landslides are evident at two locations but they are not considered fatal flaws and could be mitigated as part of the dam design. No active faults have been identified at the potential dam sites. The availability of low permeability fill material for core material in an earth/rockfill embankment dam is limited. This has been recognized and further studies will be necessary to confirm the location and properties of such material.

3. Optimisation of Dam Location and Type

A study to determine the optimum dam location and type has been undertaken and is documented in the T&T report 'Feasibility Study Optimisation of Dam Location and Type' (T&T Ref.24727.301, 10 December 2007). This study considers the possibility of two different sized storages (13 and $16MM^3$). The optimisation study has compared seven potential dam sites and three different dam types: earth/rockfill embankment, concrete faced rockfill dam (CFRD) and roller compacted concrete (RCC).

We consider that the seven potential dam sites selected by T&T are appropriate and cover the range of realistic possibilities. We are also in agreement with the three types of dams that have been considered. For each type of dam a concept design has been prepared as the basis for preparing construction cost estimates. Quantities have been determined from computer generated design models using ground contour data that was obtained from a recent survey. Generally the design concepts for the different dam types, including assumptions for diversion and spillway design are reasonable for the purposes of assessing options. Modifications to these designs can be expected as design is advanced for the selected option. We consider that it may be necessary to widen the core of the earth/rockfill embankment, and add an upstream filter zone, but there details should not add significant cost. Also the slopes of the shoulder associated with the CFRD options may require flattening to achieve acceptable performance under seismic loading conditions. A conservative approach has been adopted with respect to diversion requirements and this is appropriate. Unit rates for the key items used to determine the construction cost estimates are generally considered realistic. The rate for core material in an earth embankment involves some uncertainty at this time because locations and haul distances have not yet been confirmed. Analyses have been conducted to assess the sensitivity of the construction costs of the various dam types to variations in the unit rates, and they indicate a low sensitivity to the cost of core material.

The optimisation study concludes that an earth/rockfill embankment at CH11010 is the optimal solution from a cost perspective. The conclusion is the same for both the 13 and 16Mm^3 storages. Two other options (CH10540 and CH10880) are only about 6-8 percent more expensive, which is not that significant given the level of uncertainty associated with studies to date. However, these options are less favourable for diversion and spillway construction. For most sites, including the preferred (CH11010) the CFRD option is only marginally more expensive than the earth/rockfill embankment. Taking into account the uncertainties associated with the assumptions adopted in the optimisation study the difference in cost between the earth/rockfill embankment and the CFRD option is not significant.

We agree with the T&T recommendations that CH11010 is the preferred site. T&T conclude that an earth/rockfill embankment is the preferred dam type. Due to the uncertainty associated with suitable core material geotechnical investigations are recommended that focus on locating suitable core material, to confirm dam type suitability, prior to investigating the dam foundations. This is appropriate, however, we recommend that due to the very close cost difference between an earth/rockfill embankment the CFRD option, consideration could be given to also advancing studies for the CFRD option in parallel, particularly if there is an imperative to have the dam commissioned as quickly as possible. The CFRD option is a simpler design and has cost/time advantages with respect to diversion arrangements and should have a shorter construction period.

4. Summary and Conclusions

We consider that the geotechnical and engineering studies undertaken by T&T are appropriate for optimising the location and type of dam for the proposed water storage reservoir on the Lee River. We agree that the preferred site is CH11010. An earth/rockfill embankment is identified as the preferred dam type and recommendations for advancing consideration of this option are provided. We agree, however, we note that the cost difference between an earth/rockfill embankment and a CFRD is not significant and consideration could be given to also advancing studies for a CFRD in parallel, particularly if there is an imperative to commission the dam as quickly as possible.

Yours faithfully
ENGINEERING GEOLOGY LTD

Trevor Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON

RECEIVED
28 FEB 2008
BY: SLM

26 February 2008

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.2

This report covers review of the Tonkin and Taylor Ltd memo dated (ref: 24721.203) 20 February 2008. That memo reports on the results of additional sampling, testing and geological mapping undertaken to assess the characteristics of potential borrow material (earthfill and rockfill) for an earthfill (clay core) dam and a concrete faced rockfill dam (CFRD). The memo was accompanied by the laboratory transcripts of the test results on the different materials. I have reviewed both the test results and the memo.

The results indicate limited sources of plastic, low permeability material. Greater quantities of low plasticity earthfill were identified, but the study indicates that the available volumes of these materials are not great. It is possible to construct an earthfill dam with low plasticity earthfill, but particular attention is required to the design of filters and a high standard of construction would be required to provide assurance that the constructed dam met design specifications. The study has identified significant volumes of rock suitable for construction of a CFRD dam. Consequently Tonkin and Taylor has recommended that priority should be given to assessing the feasibility of a CFRD. I agree with this recommendation and the recommendation to develop a staged dam foundation investigation programme. I note that a CFRD has some advantages over an earthfill type dam including:

- it should be able to be investigated and designed quicker
- construction is less dependent on weather
- it can be constructed faster
- there are potential savings with flood diversion costs during construction because the design standards for the diversion conduit can be less onerous as a CFRD can sustain some overtopping during construction.

In summary, I consider that the approach that has been adopted to investigate potential sources of borrow is adequate for this stage of the project. The results indicate that priority should be given to proceeding with evaluating the feasibility of a CFRD. I consider the recommendations provided by Tonkin and Taylor for progressing with the feasibility of a CFRD are appropriate.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref6387

Tonkin and Taylor Ltd
P O Box 5271
AUCKLAND

20 June 2008

Attention: Robin Dawson

Dear Robin,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.3

Tonkin and Taylor (T&T) have recently undertaken Phase 2 and 3 investigations for a potential dam on the Lee River. The results are presented in an interim report (Ref:24727.201, June 2008). Phase 2, geotechnical investigations covered assessment of:

i) the stability of the landslide on the left bank between 11,700 and 11,900m
ii) the engineering geology of a dam site at 11,010m
iii) a potential hard rock borrow site in the Putaki Melange, on the left bank at 10,200m

Phase 3 investigations comprised:

i) review of aerial photographs
ii) mapping bedrock geology exposures in the vicinity of the landslide, proposed dam and borrow area
iii) excavation and logging of 18 test pits at the landslide and proposed dam locations
iv) drilling and logging of four triple-tube rotary boreholes.

We previously reviewed and commented on the proposed scope of work for the Phase 2 and 3 investigations.

In addition to the interim report covering the Phase 2 and 3 investigations T&T have prepared a memo (Ref:24727.301, 17 June 2008) that provides interpretation the Phase 2 and 3 geotechnical investigations, comments on the implications of the results of the investigations and provides recommendations for advancing studies for the Lee River Dam. This memo identifies some issues that potentially affect the performance of the dam and will increase previous construction cost estimates for the dam at 11,010m. The issues are:

- the rock mass quality on the right abutment is poor to a large depth (>35m) and will likely result in the need for extensive foundation remedial works (additional excavation to remove relaxed rock and extensive grouting). The poor quality rock on the right abutment is believed to be due to the presence of a splay fault associated with the Anslow Fault

- up to approximately 14m of alluvial and colluvial materials blanket the left abutment which most likely will require removal

- active slope movement is evident upstream of the dam on the left bank and will likely require buttressing (T&T estimate the volume of buttress fill to be about 200,000m^3).

As a consequence of the above issues, T&T have undertaken a further site reconnaissance and have reviewed the geological ranking of the sites reported in the Project Feasibility Study Optimisation of Dam Location and Type report (December 2007). It is concluded that many of the dam sites considered in 2007 will be affected by the features identified by the Phase 2 and 3 investigations for a dam at 11,010m. The recent site reconnaissance and air photo interpretation indicates that higher quality rock would be expected between 12,100m and 13,000m. Consequently T&T now recommend that the Phase 2 Feasibility Study be extended to undertake a staged investigation of foundation condition at sites between 12,100m and 13,000m.

We have reviewed the data from the Phase 2 and 3 investigations and agree with the interpretation of the results and the assessment of the implications for the dam at 11,010m. The additional work that will be necessary at this site is such that the previous economic advantage with this site compared to between 12,100m and 13,000m is not longer valid. Consequently, we also agree with the recommendation to undertake a staged investigation of foundation conditions at sites between 12,100m and 13,000m. In addition, we note that as a result of the Phase 2 and 3 investigations, no suitable core material in the vicinity of the dam has been found. The closest source is approximately 18km downstream on private property. This has implications on the cost for a zoned earth/rockfill embankment, and supports the adoption of a concrete faced rockfill dam (CFRD).

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON

9 April 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
ALTERNATIVE SITE (CH 12,000m - 13,000m)
EXTERNAL PEER REVIEW REPORT NO.4

Since July 2008 Tonkin and Taylor (T&T) have planned and undertaken geotechnical investigations of an alternative dam site located between CH12,000m and 13,000m. Investigations of this site were instigated because of limitations identified with the previously preferred site at CH11,010m.

Two stages of investigation have been conducted at the CH12,000m - 13,000m site as summarised below:

1. Stage 1

This comprised surface mapping of track and river bank exposures, interpretation of the data, initial assessment of suitability of the site for a dam and recommendations for more detailed (Stage 2) investigations. The Stage 1 investigations are summarised in a T&T memo dated 28 October 2008.

2. Stage 2

This comprised the drilling of five drillholes with packer permeability tests, further mapping of exposures in tracks formed to access the drillhole locations and interpretation of the results of the investigations. The Stage 2 investigations are summarised in a memo dated 25 February 2009. The initial scope of work for Stage 2 was for three drillholes. However, we recommended an additional drillhole in the spillway. A further drillhole was drilled as a result of encountering poor quality rock in the south abutment. In total, 5 drillholes were drilled.

The Stage 2 investigations have been undertaken by T&T in a staged manner, with on-going reporting of the results by the Engineering Geologist (Mark Foley) to the Internal Engineering Geology Peer Reviewer (Bernard Hegan), the Dam Designer (Robin Dawson), and the Internal Dam Design Peer Reviewer (Alan Pickens) as drilling progressed. T&T's Project Director for this particular work package, Senior Engineering Geologist Gary Smith, was involved in an overview capacity. We have participated in regular tele-conferences with the team to discuss the results, to confirm the depth of drilling and the locations of subsequent drillholes, and the need for additional drillholes to fill in gaps of knowledge.

The investigations indicate that a potential dam site exists between CH12,420m and 12,600m. T&T have reported that the rock mass quality is fair, but permeabilities are acceptably low for a dam allowing for some grouting of the near-surface rock. Defects are not generally unfavourably orientated.

We consider that T&T's memo dated 25 February 2009 on the Stage 2 investigations provides a good summary of the results and conclusions arising from the investigations to date. We confirm our agreement with T&T's conclusions that a potential dam site exists between CH12,420m and CH12,600m. The site is suitable for a CFRD or embankment dam, and possibly a RCC dam if the rock mass modulus can be proven to be consistently high across the site. We consider a CFRD is most suited, as low permeability core materials for an embankment dam are scarce. Additional investigations and analyses will be necessary to confirm the depth of excavation for the plinth associated with a CFRD, extent of foundation excavation, foundation treatment (grouting, rock anchors), stability issues during construction, rockfill properties and diversion structures. However, these are more matters of detail.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

9 September 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE VALLEY IRRIGATION STORAGE DAM
ENGINEERING FEASIBILITY REPORT
EXTERNAL PEER REVIEW REPORT NO.5

This report summarises the conclusions arising from our review of the engineering feasibility study of the proposed Lee Valley Irrigation Storage Dam undertaken by Tonkin and Taylor (T&T) and presented in a report titled “Lee Valley Storage Dam Engineering Feasibility Report” (Ref: 24727.303, September 2009). A supporting report “Geotechnical Investigations Report” (Ref: 28727.204, September 2009) has also been prepared and reviewed. The feasibility study is for a concrete faced rockfill dam located at CH12,430m. This site was selected following review of potential dam sites and preliminary geotechnical investigations. The original preferred site was at Chainage CH11,010m. However, geotechnical investigations revealed issues that had an adverse effect on potential construction costs and programme. We reviewed the outcome of those investigations and endorsed the conclusions and recommendations (see External Peer Review Report No 3 20 June 2008).

The engineering feasibility report is focussed on the preliminary design of the proposed dam. Sections 1-9 cover site selection, design standards and inputs. Section 10 covers the evaluation of options at the selected dam site (embankment type, flood diversion and routing and optimisation of spillway and dam crest parameters). Section 11 presents the arrangements of the embankment, spillway and outlet works. Section 12 Construction Methodology and Section 13 Capital Construction Cost Estimate are excluded from the review. We understand that the commencement of these sections is dependent on our endorsement of the preliminary design presented in the current issue of the engineering feasibility report. They will be completed in October. The engineering feasibility study report makes reference to accompanying technical reports, ‘Dambreak Hazard Assessment’ and ‘Hydrological Assessment’. The results from a dambreak assessment are used to assess the potential impact category (PIC) of the design. This can be Low, Medium or High. The PIC guides the design standards for the dam. The design standards for a High PIC dam have been adopted by T&T in their preliminary design and we understand that early indications from the dambreak assessment support this assumption. We also consider this is the most likely outcome and endorse the assumption of the High PIC. We understand the dambreak assessment report will be made available to us shortly. Relevant hydrological information has been developed and used in the feasibility study. The interpretation of the hydrological data and development of design assumptions presented in the Feasibility Report appear reasonable. We understand the hydrological report will also be made available to us shortly.

We consider that engineering studies have been undertaken to a level that is appropriate to establish and confirm the feasibility of the proposed dam at CH 12,430m. We agree that geotechnical investigation studies indicate the site is suitable for a concrete faced rockfill dam. No issues affecting the feasibility of the proposed dam have been identified, and we consider that the geotechnical investigations undertaken are appropriate for this stage of feasibility assessment. There are some issues that need to be considered at the detailed design phase and they are summarised in 'Section 7 Conclusions and Recommendations' of the Geotechnical Investigation Report. The more significant issues include:

- further mapping, drilling and test pitting to confirm the range of rock strengths, orientation and strength of defects in the rock, particularly in the spillway area, and to delineate potential permeable zones (seepage pathways) or acquitards
- further packer testing to assess the location and permeability of potential seepage pathways and the groutability of the rock
- assessment of the dispersion and erosion vulnerability of shear zone material
- confirmation of foundation stripping depths
- undertake a site-specific seismic hazard study to confirm design earthquake ground motions and the potential for ground movement
- excavation and compaction trials to assess the grading and permeability of the potential rockfill. This needs to take into consideration the likely mix of rock types (greywacke and argillite) that will result during excavation of the spillway. This may lead to re-evaluation of the quantities of the different types of rockfill for dam construction
- assessment of durability of the fill and filter material under expected service conditions that include freeze/thaw/wetting and earthquake shaking

We endorse these conclusions, and agree that they are appropriate to investigate at the detailed design phase.

The proposed dam design (i.e. concrete faced rockfill dam) is well proven overseas, including in areas of high seismic hazard. There are many dams of this type that are considerably larger than that proposed on the Lee River. T&T's preliminary design is generally in accordance with established guidelines. The intake structure is not typical in that it is not a free standing structure. It is located on the upstream face of the dam. However, we understand that there is some historical precedence for this.

Issues that we recommend giving further consideration to at the detailed design phase include:

- zoning of the embankment to ensure that the principal design objectives (to minimise deformation under the face slab and to have a zone of high drainage capacity at low level) are achieved with the available construction materials. This may require some modification to the current zoning and use of alluvial materials to achieve drainage in critical areas
- confirm that sufficient freeboard has been allowed for in the maximum design flood event
- providing mesh reinforcement to the downstream shoulder so as to allow overtopping during construction which would allow a reduction in the size of the diversion culvert

In assessing the construction cost estimate we recommend allowing for:

- additional 100mm thickness of concrete in the face slab to allow for the difference between design and actual constructed thickness
- anchor bars and drainage holes beneath the ogee weir, spillway chute and flip bucket.

In summary, we agree that an appropriate level of geotechnical investigations and preliminary dam engineering design has been undertaken by T&T for this Feasibility Study, and that the results adequately demonstrate the feasibility of a concrete-faced rockfill dam at CH12,430m on the Lee River.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

10 November 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER DAM: CATCHMENT AND FLOOD HYDROLOGY
AND DAM BREAK ANALYSIS AND HAZARD ASSESSMENT
EXTERNAL PEER REVIEW REPORT NO.6

We have undertaken review of the following documents:

- Section 4 Catchment Hydrology from Waimea Water Augmentation Phase 2-Water Resource Investigations Report (T&T Ref:24727.100)
- Section 5 Flood Hydrology from Waimea Water Augmentation Phase 2-Water Resource Investigations Report (T&T Ref:24727.100)
- Lee River Dam: Dam Break Analysis and Hazard Assessment (T&T Ref:24727.304)

The catchment hydrology has been updated from the Phase 1 study. A flow recording station was installed on the Lee River upstream of Waterfall Creek on 20 April 2007 and this has assisted with improving the accuracy of flow estimates at the proposed dam site. In addition, the proposed dam site has moved upstream to CH12,430m and this has been accounted for in the update of catchment water balance, mean flows and low flow analysis.

Flood hydrology for the proposed dam site (CH12,430m) has also been updated as part of the Phase 2 studies. Three methods have been used to compute design floods for a range of return periods. Synthetic flood hydrographs were compared to the flood hydrograph computed using the conventional catchment rainfall-runoff model. They were comparable in terms of both peak flow and overall flow volume. The 48 hour duration storm is predicted to be the critical in terms of reservoir routing. An estimate of the probable maximum flood is also provided.

We consider that catchment and flood hydrology have been thoroughly assessed and will provide an adequate basis for final design.

A dam break analysis has been undertaken to assist with determining the potential impact classification (PIC) of the dam and to provide information for the emergency action plan (EAP). This information is required by the Building (Dam Safety) Regulations 2008. The dam break analysis has been conducted for a 'sunny day' failure as incremental damages have been assessed likely to be greater than for a flood-induced failure scenario. This is often the case and we consider it also likely to be the case for the Lee River Dam. A rigorous approach has been used to assess the effects of a dam break. The results show quite clearly that the Lee River Dam should be categorised as high PAR. This arises from the modelling that shows approximately 260-300 properties would be at risk of flooding from water depths in excess of 0.5m. We consider that the dam break analyses have been undertaken in accordance with current accepted practice and we concur with the conclusion that the dam should be categorised as high PIC.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

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Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
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Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

10 December 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE VALLEY STORAGE DAM - ENGINEERING FEASIBIITY - CONSTRUCTION METHODOLOGY AND CAPITAL CONSTRUCITON COST ESTIMATE
EXTERNAL PEER REVIEW REPORT NO.7

We have undertaken review of Section 12 'Outline Construction Methodology' and Section 13 'Capital Construction Cost Estimate' contained in the December 2009 (Rev 0.4) version of the Lee Valley Storage Dam Engineering Feasibility Report. Our comments follow:

1. Outline Construction Methodology

The proposed construction methodology is considered reasonable. The diversion for construction consists of box culverts located on the right-hand side of the river channel. The 50 year flood was selected as the design storms for diversion, in accordance with ICOLD Bulletin 48a (19986) recommendations. Analyses for either two or three box culverts are presented in the Engineering Feasibility report. Results indicate acceptable performance. There are lower risks of overtopping when three box culverts are used but this does add significantly to the cost (approximately $2.6 million). It is proposed to undertake a detailed risk assessment during detailed design. We endorse this approach. It will allow the Designer to make informed decisions about the diversion design that take into account risk and cost/benefit, and allow the design to be optimised. We understand that design issues such as the extent and details of the proposed sheet pile wall and the need for reinforcement of embankment fill to withstand overtopping will be considered at the final design stage, taking into account the results from the proposed risk assessment. We note that due to the shallow depth of rock in the streambed it will be difficult to drive the sheet piling far and so armouring maybe required to prevent erosion of the river gravels, overlying bedrock. The gravels will be supporting the toe of the sheet piling.

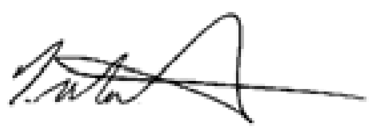
The construction programme indicates a two year period for construction. This is realistic assuming that construction is timed to take advantage of the seasonal variation in weather.

2. Capital Construction Cost Estimates

Cost estimates for the various components of work generally appear reasonable. We consider that the rates are appropriate for a situation where bids are obtained in a competitive environment, such as exists at the present time. In a busy period rates would be expected to be higher. We note that no separate allowance has been made for formation and maintenance of haul roads, establishment of borrows areas and sediment control. We understand that these items are included in earthworks rates, but this should be clarified. Rates for perimetric and face-slab joints are much lower than compared to a CFRD in Australia that we are familiar with and that is currently being designed, although it has copper water stops. No separate item is noted for establishment / disestablishment. We understand it is covered under the Contractors preliminary and general cost.

The cost estimate includes a 20 percent contingency. We consider this appropriate. The preliminary and general costs are assumed to be 15 precent of the base cost. The project is of a scale that these costs could be greater than assumed, although the 20 percent contingency would be expected to partly compensate for any under estimation in this item.

Yours faithfully
ENGINEERING GEOLOGY LTD



T Matuschka, CPEng

Appendix G: Estimated construction costs for reduced demand scenarios

Tonkin & Taylor

Memo

To:	Robin Dawson, Sally Marx, Greg Anderson	**T&T Ref**:	24727.303
From:	John Chesterton, David Leong	**Date:**	4 December 2009
cc:			
Subject:	Lee Valley Dam Re-Costing for Two Reduced Demand Scenarios		

1.0 Introduction

This memo summarises the reduction in quantities, costs and potential hydropower generation for two alternative scenarios with reduced abstractive demand. The original base case quantities and cost estimates were computed for a dam at CH 12430 m with a Normal Top Water Level (NTWL) of 197 m RL which corresponded with a gross reservoir storage capacity of 13.4 million cubic metres.

All things being equal, a reduction in the abstractive demand will translate to a reduction in the required storage capacity. The dam will thus be lower and smaller, and correspondingly cheaper. However, the potential hydro generation will also be less because of lower generating head.

2.0 Summary

Key results are summarised in Table 1 below. The demand scenarios, costing methodology and assumptions, and reduction in hydropower potential are outlined in subsequent sections.

Table 1: Cost and Output Summary for Reduced Demand Scenarios

	Base Case	**Scenario A** (Removal of future regional demand)	**Scenario B** (Removal of 1500 ha. and future regional demand)
Storage Base Case (cu. m)	13,400,000	-	-
Storage Reduction (cu. m)	-	2,120,000	4,220,000
Storage (cu. m)	13,400,000	11,300,000	9,200,000
NTWL (m RL)	197	193.6	189.9
Capital Cost (3 Culverts)	$ 38.1 million	$ 35.2 million	$ 33.2 million
Capital Cost (2 Culverts)	$ 35.5 million	$ 32.7 million	$ 30.7 million
Cost per m^3 *	2.84 (2.65)	3.12 (2.89)	3.16 (3.33)
Potential Hydro Generation	6.23 GWh p.a.	5.66 GWh p.a.	5.00 GWh p.a.
Installed Capacity/ Capital Cost	0.99 MW/ $4.25 million	0.91 MW/ $4.07 million	0.81 MW/ $3.85 million

*Brackets for two culvert case

3.0 Reduced Demand Scenarios

The two reduced demand scenarios that have been considered are as follows:

A. Eliminate the allowance for the Future Regional Demand (22,000 m^3/day constant surface water take);

B. In addition to A., reduce the allowance for new irrigation by up to 1500 hectares; this relates to Wai-iti (300 ha), Rabbit Island (250 ha), and part of the un-irrigated area on the Waimea Plains that needs expensive distribution (approximately 950 ha).

The storage simulation model was re-run for the two reduced demand scenarios, and the resulting reduction in live storage requirements were found to be 2.12 and 4.22 million cubic metres respectively for Scenarios A. and B. These volumes have been deducted from the base case and the corresponding revised NTWLs found using the storage-elevation curve for the dam site.

Note that if the 1500 hectare reduction were to be implemented first and the future regional need deletion second, the storage reduction for the Future Regional Demand component would be lower than indicated above (approximately 1.8 million m^3 compared with 2.12 million m^3 above). The point here is that the order in which demand components are removed has an effect on the live storage reduction attached to any particular component.

Methodology and Assumptions for Re-Costing

The methodology and assumptions used for estimating the quantities for the Lee Valley Dam are outlined below. A full redesign was not undertaken but rather a re-evaluation of the items that would have a significant impact on the cost and some targeted calculation of those quantities. Revised cost evaluation spreadsheets for both reduced demand scenarios are attached.

Determine NTWL

The NTWL was recalculated (see Table 1) using the existing base case storage elevation curve for a dam at CH 12430 m and crest level of 202 m RL.

Route flows through spillways and resize spillways

The routing spreadsheets used for the base case were utilised to re-route the flood flows for the OBF and MDF events. Key assumptions are as follows:

- Spillway widths are as per base case.
- Freeboard allowance of 5m from NTWL to crest level was assumed as per base case scenario.

Routing calculations showed that these assumptions could be applied to the reduced storage scenarios with only minor variations in flood rise and within the tolerances given in the base case. Although the spillways would be expected to widen due to the reduction in storage, the large flood volumes compared to the amount of live storage provided show there is very little attenuation during design floods.

Calculate new crest level.

Given the routing calculations undertaken new crest levels were calculated at 5m above NTWL.

Recalculate embankment zone volumes

As the embankment volumes account for a large proportion of the cost, these were re modelled using the 12D CAD package. The embankment was lowered and the embankment templates were adjusted to allow for the new crest height.

Recalculate spillway cut volumes.

Spillway cut volumes were also re modelled in 12D. The spillways were lowered to achieve the desired NTWL and the cut volumes for various rock types were calculated. The Lee Valley road was realigned as part of this process. The Power station access road was not remodelled.

Calculate revised cut and fill balance

Using the volumes calculated for the revised embankments and spillways, a new cut fill balance was derived and incorporated into the costing spreadsheets.

Update structures quantities

Following the remodelling and calculation of earthworks quantities, quantities for structures that had been significantly changed or that were thought to have a significant cost impact were calculated as discussed below.

- **Diversion** – Volumes for the diversion culverts were recalculated by scaling the base case volumes by the new length over base case length. All other structures including preliminary diversion works were assumed to remain the same.
- **Plinth** – The length of the perimetric joint was re-measured from the embankment model. Similar to the base case quantities, this length was used to recalculate the foundation work and volume of plinth concrete. The volume of the starter dam or how it affects the length of the perimetric joint has not been taken into account
- **Embankment Structures** – The quantity of concrete for the concrete face and parapet wall were re-measured from the embankment model to provide updated quantities. The vertical jointing was scaled with respect to the comparative areas of the base and reduced storage cases. Road aggregate for the crest was also re-measured.
- **Spillway Structures** – As the spillway widths were consistent through all three cases, the quantities for the fuseable embankment and primary spillway weir and approach slab were not changed. Quantities for the chute did not change significantly and were not recalculated. The flip bucket was not redesigned and it was assumed that these quantities would remain approximately the same. Excavation for the plunge pool was also assumed to be as per base case.
- **Other items** – Quantities for the outlet works fish pass, access and contingency items were not modified as they either did not change or were not a significant cost item.

5.0 Reduction in Potential Hydro Generation

The potential reduction in hydro generation is proportional to the reduction in the maximum generating head. In each case the base case design flow capacity and operating buffer storage has been retained, comprising:

- a residual flow unit (turbine and generator) with a flow capacity of 0.51 m^3/s matching the dam residual flow; plus
- a main unit (turbine and generator) with a flow capacity of 2 m^3/s; and
- an operational storage volume of 250,000 m^3 for hydropower regulation to enhance capture of inflows (that would otherwise be spilled) to generation.

There is a corresponding reduction in peak power output (installed capacity) to about 0.91 MW and 0.81 MW for Scenarios A. and B. respectively (compared with 0.99 MW in the base case). The cost of the hydro add-on for Scenarios A. and B. would also be correspondingly lower at $4.07 million and $3.85 million compared with $4.25 million for the base case.

At 5.66 GWh p.a., the potential hydro generation for Scenario A. is 9% lower than the base case, while for Scenario B., the potential generation is 20% lower than the base case at 5.00 GWh p.a.

04 December 2009
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Attachment 1

Cost Evaluation: **Reduction A - Removal of future regional demand**

LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES — Ver 01a

COMPONENT COMBINATIONS AND SUMMARY OF COSTS — 19-Nov-09

Item	Components	% Cost	Current Cost Estimate			Previous Cost Estimate			Percent Variation		
			Civil	E&M	Total	Civil	E&M	Total	Civil	E&M	Total
1	Site Preparation & Diversion Works	25.1%	$8,809,200	$0	$8,809,200	$10,316,730	$0	$10,316,730	-15%	100%	-15%
2	Plinth Work	10.4%	$3,645,525	$0	$3,645,525	$4,379,565	$0	$4,379,565	-17%	100%	-17%
3	Embankment	34.5%	$12,142,886	$0	$12,142,886	$14,916,046	$0	$14,916,046	-19%	100%	-19%
4	Spillway	22.7%	$7,990,635	$0	$7,990,635	$6,600,268	$0	$6,600,268	+21%	100%	+21%
5	Outlet Works	3.8%	$915,064	$434,868	$1,349,931	$941,034	$434,868	$1,375,902	-3%	0%	-2%
6	Fishpass	0.2%	$87,375	$0	$87,375	$87,375	$0	$87,375	+0%	100%	+0%
7	Access	3.2%	$1,127,258	$0	$1,127,258	$1,127,258	$0	$1,127,258	+0%	100%	+0%
	TOTALS	100.0%	$34,717,942	$434,868	$35,152,809	$38,368,275	$434,868	$38,803,143	-10%	0%	-9%

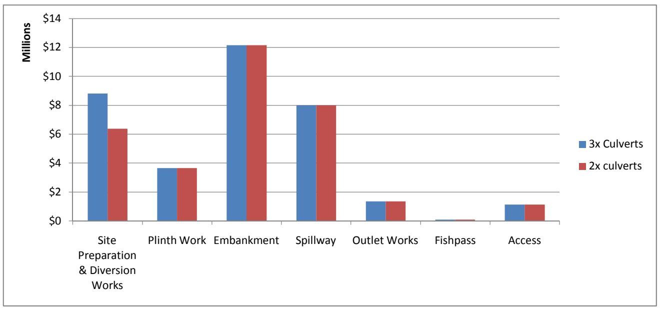
Base Cost Included Above	$ 23,145,294	$ 343,769	$ 23,489,063	Base cost
Contingency Allowance Included Above	$ 4,629,059	$ 34,377	$ 4,663,436	Contingency
Design Allowance Included Above	$ 2,777,435	$ 18,907	$ 2,796,343	Design allowance
P&G Allowance Included Above	$ 4,166,153	$ 37,815	$ 4,203,968	Preliminary and general
TOTALS	$ 34,717,942	$ 434,868	$ 35,152,809	

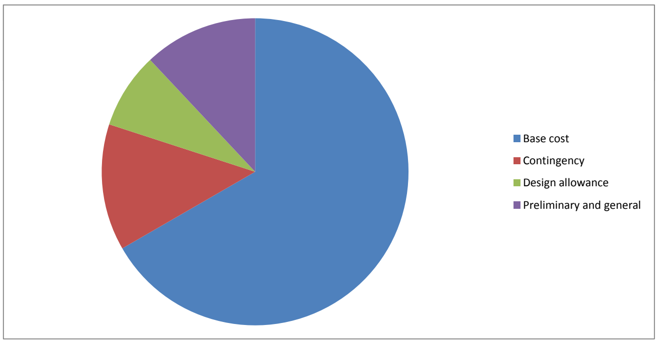
Contingency Allowance Calculated Explicitly $ 3,100,035.00

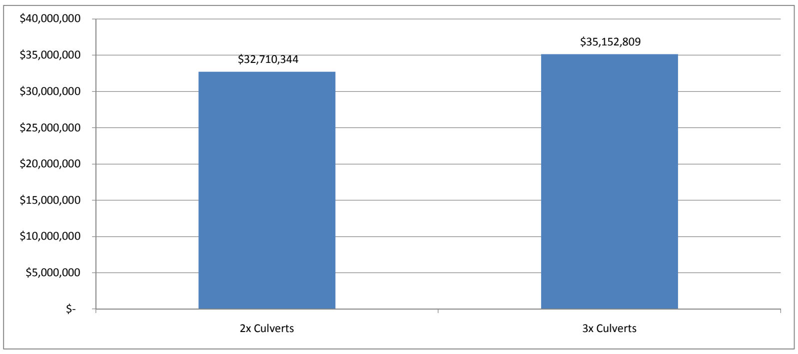
Costs with 2x Culverts

Item	Components	% Cost	Civil	E&M	Total
1	Site Preparation & Diversion Works	19.5%	$ 6,366,735.00	$ -	$6,366,735
2	Plinth Work	11.1%	$3,645,525	$0	$3,645,525
3	Embankment	37.1%	$12,142,886	$0	$12,142,886
4	Spillway	24.4%	$7,990,635	$0	$7,990,635
5	Outlet Works	4.1%	$915,064	$434,868	$1,349,931
6	Fishpass	0.3%	$87,375	$0	$87,375
7	Access	3.4%	$1,127,258	$0	$1,127,258
	TOTALS		$ 32,275,476.55	$ 434,867.53	$ 32,710,344.08

2x Culverts	$ 32,710,344
3x Culverts	$ 35,152,809







Description	Code	Unit	Rate
Earthworks & Related			
Soil cut to fill (Zone 1A)	cutToFill1A	cu.m	$ 5.00
Soil cut to fill (Zone 1B)	cutToFill1B	cu.m	$ 5.00
Soil cut to fill incl processing (Zone 2A)	cutToFill2A	cu.m	$ 20.00
Soil cut to fill incl processing (Zone 2B)	cutToFill2B	cu.m	$ 20.00
Rock cut to fill (Zone 3A)	cutToFill3A	cu.m	$ 10.00
Rock cut to fill (Zone 3B)	cutToFill3B	cu.m	$ 10.00
Rock cut to waste	cutToWasteRock	cu.m	$ 8.00
Soil cut to waste	cutToWasteSoil	cu.m	$ 4.00
Foundation cleanup for concrete placement	fndCleanConc	sq.m	$ 10.00
Road aggregate	roadAgg	cu.m	$ 60.00
Shotcrete slope protection	slopeProtect	sq.m	$ 100.00
100 dia HDPE pipe	pipeHDPE100	m	$ 20.00
Dowell anchor drilled into rock	anchorDowell	no	$ 200.00
Heavy rock armour	armRockHeavy	cu.m	$ 100.00
Treatment of foundation defects at plinth	defectTreat	no	$ 4,000.00
Drill and grout	drillGrout	m	$ 500.00
Geotextile	geoTex	sq.m	$ 10.00
Grouted rock	groutRock	cu.m	$ 150.00
750 dia concrete culvert	culvert750	m	$ 750.00
Sheet Piling	SteelSheetPile	t	$ 4,000.00
Bulk borrow to fill	borToFill	cu.m	$ 5.00
Liner placement	liner	cu.m	$ 12.00
Liner protection armour	linerPro	cu.m	$ 6.00
Wave armour	waveArm	cu.m	$ 12.00
Topsoil stripping	strip	cu.m	$ 2.00
Topsoiling and grassing	topsoil	sq.m	$ 2.00
Hydroseeding	hydroseed	sq.m	$ 2.00
Filter material	filter	cu.m	$ 60.00
Coarse filter	corFilter	cu.m	$ 60.00
Drainage material	drainage	cu.m	$ 60.00
Drainage pipe	drnPipe	m	$ 30.00
Steel sheet piling	SteelSheetPile	t	$ 4,000.00
Crest fence (farm type)	wireFence	m	$ 10.00
Heavy armour	armourHeavy	cu.m	$ 60.00
Coffer dam placement	cofferPlace	cu.m	$ 12.00
Coffer dam removal	cofferRemove	cu.m	$ 8.00
Piles	piles	m	$ 1,000.00
Heavy rubber bearings	heavyBearing	no	$ 4,000.00
Light rubber bearings	lightBearing	no	$ 2,000.00
Structural & Power Station Related			
Mass concrete	massConc	cu.m	$ 350.00
Structural concrete including reo	struConc	cu.m	$ 800.00
Formwork - straight	formStr	sq.m	$ 150.00
Formwork - curved	formCur	sq.m	$ 350.00
Formwork - slip formed	formSlip	sq.m	$ 100.00
Reinforcing steel	reoSteel	t	$ 3,200.00
Structural steel	strSteel	t	$ 5,000.00
Roller compacted concrete	RCC	cu.m	$ 150.00
Perimetric joint waterstop	periWS	m	$ 100.00
Waterstop joint	watStop	m	$ 50.00
Radial gate steel	steelRadGate	t	$ 18,000.00
Stoplog gate steel	steelStopLog	t	$ 10,000.00
General hydraulic structure steel	steelHydGen	t	$ 12,000.00
Steel pipe	steelPipe	t	$ 4,000.00
2 L/s capacity pump	puml2LS	No	$ 2,000.00
Road bridge cost based on plan area	bridgePlan	sq.m	$ 1,250.00
Retaining wall based on face area	retainWall	sq.m	$ 300.00
Instrumentation			
Flow monitoring equipment	flowMon	no	$ 1,500.00
Deformation marker	defMark	no	$ 1,000.00
Contingencies, percentages etc			
Civil minor items			5%
Civil contingency			20%
Civil engineering			10%
Civil P&G			15%
E&M minor items			5%
E&M contingency			10%
E&M engineering			5%
E&M P&G			10%

This sheet contains base information for titles, revs etc on all other sheets & is dynmically linked.

Project:	LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES	
JobNo:	24727.303	
Current Ver & date:	Ver 01a	19-Nov-09

Release History	Ver	Date	Notes
	00	19-Nov-09	First cut at construction cost estimate, prior to internal/external review
	01	9-Nov-09	Following constructor review and prelim optimisation
	01a	18-Nov-09	Recosting for reduced storage volume - NTWL 193.6mRL

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
1.1	**Diversion Through Permanent Culverts**						
1.1.1	Excavation	cu.m	8,711.	cutToWasteSoil	$ 4	$ 34,844.00	
1.1.2	Diversion Culverts (Upstream Section)						
1.1.2.1	Concrete	cu.m	4,213.	massConc	$ 350	$ 1,474,550.00	
1.1.2.2	Steel	t	620.	reoSteel	$ 3,200	$ 1,984,000.00	
1.1.2.3	Formwork	sq.m	6,535.	formStr	$ 150	$ 980,250.00	
1.1.3	Diversion Culverts (Downstream Section)						
1.1.3.1	Concrete	cu.m	723.	massConc	$ 350	$ 253,050.00	
1.1.3.2	Steel	t	146.	reoSteel	$ 3,200	$ 467,200.00	
1.1.3.3	Formwork	sq.m	1,791.	formStr	$ 150	$ 268,650.00	
	Subtotal						$ 5,462,544.00
1.2	**Diversion Through Temporary Culverts (upstream end)**						
1.2.1	Upstream coffer dam	cu.m	1,800.	cutToWasteSoil	$ 4	$ 7,200.00	
1.2.2	50m long by 5 m high Sheet Piling	t	11.3	SteelSheetPile	$ 4,000	$ 45,200.00	
1.2.3	Miscellaneous sealing concrete	cu.m	20	massConc	$ 350	$ 7,000.00	
1.2.4	Dewatering allowance during low plinth construction	LS	1		$ 100,000	$ 100,000.00	
	Subtotal						$ 159,400.00
1.2	**Dam Site Preparation**						
1.2.1	Stripping (Cut to Waste)	cu.m	62,714.	cutToWasteSoil	$ 4	$ 250,856.00	
	Subtotal						$ 250,856.00
	SUBTOTAL CIVIL ITEMS					$ 5,872,800.00	
	Civil contingency allowance		$ 5,872,800.00		20%	$ 1,174,560.00	
	CIVIL INCL CONTINGENCY					$ 7,047,360.00	
	Civil Engineering		$ 7,047,360.00		10%	$ 704,736.00	
	Civil P&G		$ 7,047,360.00		15%	$ 1,057,104.00	
	TOTAL CIVIL						$ 8,809,200.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 8,809,200.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
2.1	Preparation						
2.1.1	Excavation (Cut to Fill)	cu.m	21,262.	cutToFill3B	$ 10	$ 212,620.00	
2.1.2	Foundation Cleaning (plan area)	sq.m	5,833.	fndCleanConc	$ 10	$ 58,330.00	
2.1.3	Defects (1 per 200 sq.m)	no	30.	defectTreat	$ 4,000	$ 120,000.00	
2.1.4	Slope Reinforcement (Face area of 0.25H:1V Slope)	sq.m	1,986.	slopeProtect	$ 100	$ 198,600.00	
	Subtotal						$ 589,550.00
2.2	Plinth						
2.2.1	Drilling and Grouting						
2.2.2.1	Length of 15m deep curtain grouting	m	270.	drillGrout	$ 500	$ 135,000.00	
2.2.2.2	Length of 7m deep side grouting	m	270.	drillGrout	$ 500	$ 135,000.00	
2.2.2	Grouted Anchor Bars (1m depth at 1m centers)	no	1,620.	anchorDowell	$ 200	$ 324,000.00	
2.2.3	Plinth						
2.2.3.1	Concrete	cu.m	810.	massConc	$ 350	$ 283,500.00	
2.2.3.2	Formwork	t	285.	reoSteel	$ 3,200	$ 912,000.00	
2.2.3.3	Steel	sq.m	162.	formStr	$ 150	$ 24,300.00	
2.2.4	Peremetric Joint (waterstop and Hypalon cover)	m	270.	periWS	$ 100	$ 27,000.00	
	Subtotal						$ 1,840,800.00
	SUBTOTAL CIVIL ITEMS					$ 2,430,350.00	
	Civil contingency allowance		$ 2,430,350.00		20%	$ 486,070.00	
	CIVIL INCL CONTINGENCY					$ 2,916,420.00	
	Civil Engineering		$ 2,916,420.00		10%	$ 291,642.00	
	Civil P&G		$ 2,916,420.00		15%	$ 437,463.00	
	TOTAL CIVIL						$ 3,645,525.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 3,645,525.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
3.1	**Rockfill**						
3.1.1	Zone 1A Cut to Fill	cu.m	0.	cutToFill1A	$ 5	$ -	
3.1.2	Zone 1A Borrow to Fill	cu.m	2,765.	cutToFill1A	$ 5	$ 13,825.00	
3.1.3	Zone 1B Cut to Fill	cu.m	11,442.	cutToFill1B	$ 5	$ 57,210.00	
3.1.4	Zone 1B Borrow to Fill	cu.m	0.	cutToFill1B	$ 5	$ -	
3.1.5	Zone 2A Cut to Fill	cu.m	0.	cutToFill2A	$ 20	$ -	
3.1.6	Zone 2A Borrow to Fill	cu.m	5,289.	cutToFill2A	$ 20	$ 105,780.00	
3.1.7	Zone 2B Cut to Fill	cu.m	0.	cutToFill2B	$ 20	$ -	
3.1.8	Zone 2B Borrow to Fill	cu.m	29,482.	cutToFill2B	$ 20	$ 589,640.00	
3.1.9	Zone 3A Cut to Fill	cu.m	135,878.	cutToFill3A	$ 10	$ 1,358,780.00	
3.1.10	Zone 3A Borrow to Fill	cu.m	53,275.	cutToFill3A	$ 10	$ 532,750.00	
3.1.11	Zone 3B Cut to Fill	cu.m	112,622.	cutToFill3B	$ 10	$ 1,126,220.00	
3.1.12	Zone 3B Borrow to Fill	cu.m	0.	cutToFill3B	$ 10	$ -	
	Subtotal						$ 3,784,205.00
3.2	**Concrete Face**						
3.2.1	Concrete (250mm)	cu.m	2,358.3	massConc	$ 350	$ 825,405.00	
3.2.2	Formwork (15m slipformed panel areas)	sq.m	9,433.2	formSlip	$ 100	$ 943,320.00	
3.2.3	Steel	t	471.66	reoSteel	$ 3,200	$ 1,509,312.00	
3.2.4	Concrete underfill (100mm)	cu.m	943.32	massConc	$ 350	$ 330,162.00	
3.2.5	Vertical joint waterstop	m	471.66	watStop	$ 50	$ 23,583.00	
	Subtotal						$ 3,631,782.00
3.3	**Crest**						
3.3.1	Parapet Wall						
3.3.1.1	Concrete	cu.m	313.	massConc	$ 350	$ 109,550.00	
3.3.1.2	Formwork	sq.m	2,010.	formStr	$ 150	$ 301,500.00	
3.3.1.3	Steel	t	62.6	reoSteel	$ 3,200	$ 200,320.00	
3.3.2	Road Aggregate (300mm)	cu.m	315.	roadAgg	$ 60	$ 18,900.00	
	Subtotal						$ 630,270.00
3.4	**Instrumentation**						
3.4.1	Flow monitoring equipment	no	6.	flowMon	$ 1,500	$ 9,000.00	
3.4.2	Deformation Markers (U/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
3.4.3	Deformation Markers (D/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
	Subtotal						$ 49,000.00
3.5	**Electrical and Mechanical**						
3.5.1	Description						
3.5.1.1	Description				$ -	$ -	
	Subtotal						$ -
	SUBTOTAL CIVIL ITEMS					$ 8,095,257.00	
	Civil contingency allowance		$ 8,095,257.00		20%	$ 1,619,051.40	
	CIVIL INCL CONTINGENCY					$ 9,714,308.40	
	Civil Engineering		$ 9,714,308.40		10%	$ 971,430.84	
	Civil P&G		$ 9,714,308.40		15%	$ 1,457,146.26	
	TOTAL CIVIL						$ 12,142,885.50
	SUBTOTAL TOTAL E&M ITEMS					$ -	
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 12,142,885.50**

LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES							Ver 01a
ITEM 4	Spillway						19-Nov-09
No	**Description**	**Unit**	**Quantity**	**Rate Code**	**Rate**	**Amount**	**Subtotals**
4.1	**Bulk Earthworks**						
4.1.1	Cut to Waste - Soil	cu.m	133,771.	cutToWasteSoil	$ 4	$ 535,084.00	
4.1.2	Cut to Waste - Rock	cu.m	197,693.	cutToWasteRock	$ 8	$ 1,581,544.00	
4.1.3	Slope protection and reinforcement	sq.m	1,000.	slopeProtect	$ 100	$ 100,000.00	
	Subtotal						$ 2,216,628.00
4.2	**Fuse Embankment**						
4.2.1	Armoring to U/S Face (0.5m thk)	cu.m	199.	armourHeavy	$ 60	$ 11,940.00	
4.2.2	Filter Layer (0.5m thk)	cu.m	199.	cutToFill2A	$ 20	$ 3,980.00	
4.2.3	Inclined Geotextile	sq.m	398.	geoTex	$ 10	$ 3,980.00	
4.2.4	Bulk Fill	cu.m	3,493.	cutToFill2A	$ 20	$ 69,860.00	
4.2.5	Concrete Slab						
4.2.5.1	Concrete	cu.m	768.	massConc	$ 350	$ 268,800.00	
4.2.5.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.5.3	Steel (0.1t/cu.m)	t	76.8	reoSteel	$ 3,200	$ 245,760.00	
4.2.6	Approach Slab						
4.2.6.1	Concrete	cu.m	216.	massConc	$ 350	$ 75,600.00	
4.2.6.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.6.3	Steel (0.1t/cu.m)	t	21.6	reoSteel	$ 3,200	$ 69,120.00	
	Subtotal						$ 749,040.00
4.3	**Primary Spillway Chute**						
4.3.1	Ogee Weir						
4.3.1.1	Concrete	cu.m	278.75	massConc	$ 350	$ 97,562.50	
4.3.1.2	Formwork	sq.m	178.4	formCur	$ 350	$ 62,440.00	
4.3.1.3	Steel (0.05t/cu.m)	t	13.9	reoSteel	$ 3,200	$ 44,480.00	
4.3.2	Approach Slab						
4.3.2.1	Concrete	cu.m	164.5	massConc	$ 350	$ 57,575.00	
4.3.2.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.3.2.3	Steel (0.1t/cu.m)	t	16.45	reoSteel	$ 3,200	$ 52,640.00	
4.3.2.4	Foundation preparation	sq.m	658.	fndCleanConc	$ 10	$ 6,580.00	
4.3.3	Chute Floor						
4.3.3.1	Concrete	cu.m	499.	massConc	$ 350	$ 174,650.00	
4.3.3.2	Formwork (Slipformed)	sq.m	758.5	formSlip	$ 100	$ 75,850.00	
4.3.3.3	Steel (0.1t/cu.m)	t	49.9	reoSteel	$ 3,200	$ 159,680.00	
4.3.3.4	Waterstop joints	m	400.	watStop	$ 50	$ 20,000.00	
4.3.3.5	Foundation preparation	sq.m	760.	fndCleanConc	$ 10	$ 7,600.00	
4.3.4	Chute Walls						
4.3.4.1	Concrete	cu.m	382.6	massConc	$ 350	$ 133,910.00	
4.3.4.2	Formwork	sq.m	1,275.33	formStr	$ 150	$ 191,300.00	
4.3.4.3	Steel (0.2t/cu.m)	t	76.52	reoSteel	$ 3,200	$ 244,864.00	
4.3.5	Wall Backfill	cu.m	1,087.	cutToFill2A	$ 20	$ 21,740.00	
4.3.6	Chute Under Drainage (drains at 10m centers)						
4.3.6.1	Filter Material	cu.m	11.21	roadAgg	$ 60	$ 672.60	
4.3.6.2	Pipe (100 dia HDPE perforated)	m	170.	pipeHDPE100	$ 20	$ 3,400.00	
4.3.9	Chute Slab Anchors (1 Per 10 sq.m)	no	166	anchorDowell	$ 200	$ 33,266.67	
	Subtotal						$ 1,388,210.77
4.4	**Flip Bucket**						
4.4.1	Bucket						
4.4.1.1	Concrete	cu.m	681.5	massConc	$ 350	$ 238,525.00	
4.4.1.2	Formwork and curved surface formation	sq.m	438.16	formCur	$ 350	$ 153,356.00	
4.4.1.3	Steel (0.1t/cu.m)	t	68.15	reoSteel	$ 3,200	$ 218,080.00	
4.4.2	Bucket Rock Anchors (6 assumed)	no	6.	anchorDowell	$ 200	$ 1,200.00	
4.4.3	Low flow channel lining						
4.4.3.1	Concrete	cu.m	375.	massConc	$ 350	$ 131,250.00	
4.4.3.2	Formwork (Slipformed)	sq.m	0.	formStr	$ 150	$ -	
4.4.3.3	Steel (0.05t/cu.m)	t	18.75	reoSteel	$ 3,200	$ 60,000.00	
	Subtotal						$ 802,411.00
4.5	**Plungepool**						
4.5.1	Excavation	cu.m	2600	cutToWasteRock	$ 8	$ 20,800.00	
4.5.2	Rock Armour	cu.m	1,500.	armRockHeavy	$ 100	$ 150,000.00	
	Subtotal						$ 170,800.00
	SUBTOTAL CIVIL ITEMS					$ 5,327,089.77	
	Civil contingency allowance		$ 5,327,089.77		20%	$ 1,065,417.95	
	CIVIL INCL CONTINGENCY					$ 6,392,507.72	
	Civil Engineering		$ 6,392,507.72		10%	$ 639,250.77	
	Civil P&G		$ 6,392,507.72		15%	$ 958,876.16	
	TOTAL CIVIL						$ 7,990,634.65
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 7,990,634.65**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
5.1	**Inclined Inlets**						
5.1.1	Conduits on U/S Face						
5.1.1.1	Concrete	cu.m	139.94	massConc	$ 350	$ 48,979.00	
5.1.1.2	Formwork (may be precast)	sq.m	921.	formStr	$ 150	$ 138,150.00	
5.1.1.3	Steel (0.2t/cu.m)	t	27.99	reoSteel	$ 3,200	$ 89,561.60	
5.1.3	Rails embedded in upstream face	t	1.66	strSteel	$ 5,000	$ 8,300.00	
	Subtotal						$ 284,990.60
5.2	**Outlet Conduits**						
5.2.1	Box Inlet Plug						
5.2.1.1	Concrete	cu.m	105.	massConc	$ 350	$ 36,750.00	
5.2.1.2	Formwork (may be precast)	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.1.3	Steel (0.05t/cu.m)	t	5.25	reoSteel	$ 3,200	$ 16,800.00	
5.2.2	Gate Structure						
5.2.2.1	Concrete	cu.m	37.5	massConc	$ 350	$ 13,125.00	
5.2.2.2	Formwork	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.2.3	Steel (0.1t/cu.m)	t	3.75	reoSteel	$ 3,200	$ 12,000.00	
5.2.3	Overhead Beam	t	8.46	strSteel	$ 5,000	$ 42,282.00	
	Subtotal						$ 143,457.00
5.3	**Outlet Structure**						
5.3.1	Outlet Stoplog Concrete Structure						
5.3.1.1	Concrete	cu.m	12.5	massConc	$ 350	$ 4,375.00	
5.3.1.2	Formwork	sq.m	25.	formStr	$ 150	$ 3,750.00	
5.3.1.3	Steel	t	1.25	reoSteel	$ 3,200	$ 4,000.00	
5.3.2	Outlet Wingwall Structure						
5.3.2.1	Concrete	cu.m	143.	massConc	$ 350	$ 50,050.00	
5.3.2.2	Formwork	sq.m	186.	formStr	$ 150	$ 27,900.00	
5.3.2.3	Steel (0.2t/cu.m)	t	28.6	reoSteel	$ 3,200	$ 91,520.00	
	Subtotal						$ 181,595.00
5.4	**Gates, Screens and Related**						
5.4.1	Removable Bellmouth Inlet and Screen	t	5.997	steelHydGen	$ 12,000	$ 71,968.80	
5.4.2	Removable Stoplogs for intake structure	t	3.25	steelStopLog	$ 10,000	$ 32,500.00	
5.4.3	Stoplog and Screen Derrick and Winch for intake structure	t	5.	steelHydGen	$ 12,000	$ 60,000.00	
5.4.4	Radial Gates 1m x 1m x 2 gates for irrigation outlet	t	9.29	steelHydGen	$ 12,000	$ 111,480.00	
5.4.5	Tailrace area outlet Stoplogs	t	6.78	steelStopLog	$ 10,000	$ 67,820.00	
	Subtotal						$ 343,768.80
	SUBTOTAL CIVIL ITEMS					$ 610,042.60	
	Civil contingency allowance		$ 610,042.60		20%	$ 122,008.52	
	CIVIL INCL CONTINGENCY					$ 732,051.12	
	Civil Engineering		$ 732,051.12		10%	$ 73,205.11	
	Civil P&G		$ 732,051.12		15%	$ 109,807.67	
	TOTAL CIVIL						$ 915,063.90
	SUBTOTAL TOTAL E&M ITEMS					$ 343,768.80	
	E&M contingency allowance		$ 343,768.80		10%	$ 34,376.88	
	E&M INCL CONTINGENCY					$ 378,145.68	
	E&M Engineering		$ 378,145.68		5%	$ 18,907.28	
	E&M P&G		$ 378,145.68		10%	$ 37,814.57	
	TOTAL E&M						$ 434,867.53
	TOTAL						**$ 1,349,931.43**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
6.1	Fishpass						
6.1.1	Pump (2l/s)	No	1.	puml2LS	$ 2,000	$ 2,000.00	
6.1.2	Grouted rock channel	cu.m	250.	groutRock	$ 150	$ 37,500.00	
6.1.3	750 mm Culvert Crossing	m	10.	culvert750	$ 750	$ 7,500.00	
6.1.4	U/S Face Sluice	m	15.	culvert750	$ 750	$ 11,250.00	
	Subtotal						$ 58,250.00
	SUBTOTAL CIVIL ITEMS					$ 58,250.00	
	Civil contingency allowance		$ 58,250.00		20%	$ 11,650.00	
	CIVIL INCL CONTINGENCY					$ 69,900.00	
	Civil Engineering		$ 69,900.00		10%	$ 6,990.00	
	Civil P&G		$ 69,900.00		15%	$ 10,485.00	
	TOTAL CIVIL						$ 87,375.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 87,375.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
7.1	**Power Station Access Road**						
7.1.1	Surfacing Aggregate (250 mm)	cu.m	742.5	roadAgg	$ 60	$ 44,550.00	
	Subtotal						$ 44,550.00
7.2	**Auxillary Spillway Access**						
7.2.1	Surfacing Aggregate (250 mm)	cu.m	132.	roadAgg	$ 60	$ 7,920.00	
	Subtotal						$ 7,920.00
7.3	**Lee Valley Road Diversion**						
7.3.1	Surfacing Aggregate (250 mm)	cu.m	767.25	roadAgg	$ 60	$ 46,035.00	
	Subtotal						$ 46,035.00
7.4	**Spillway Bridging**						
7.4.1	Auxillary Spillway Bridge (10 m Spans)	sq.m	146.	bridgePlan	$ 1,250	$ 182,500.00	
7.4.2	Abutment Retaining	sq.m	251.	retainWall	$ 300	$ 75,300.00	
7.4.3	Primary Spillway Bridge (10 m Spans)	sq.m	160.	bridgePlan	$ 1,250	$ 200,000.00	
7.4.4	Chute Bucket Bridge (10 m Span)	sq.m	106.	bridgePlan	$ 1,250	$ 132,500.00	
7.4.5	Abutment Retaining	sq.m	209.	retainWall	$ 300	$ 62,700.00	
	Subtotal						$ 653,000.00
	SUBTOTAL CIVIL ITEMS					$ 751,505.00	
	Civil contingency allowance		$ 751,505.00		20%	$ 150,301.00	
	CIVIL INCL CONTINGENCY					$ 901,806.00	
	Civil Engineering		$ 901,806.00		10%	$ 90,180.60	
	Civil P&G		$ 901,806.00		15%	$ 135,270.90	
	TOTAL CIVIL						$ 1,127,257.50
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 1,127,257.50**

Attachment 2

Cost Evaluation: **Reduction B - Removal of 1500 ha. <u>and</u> future regional demand**

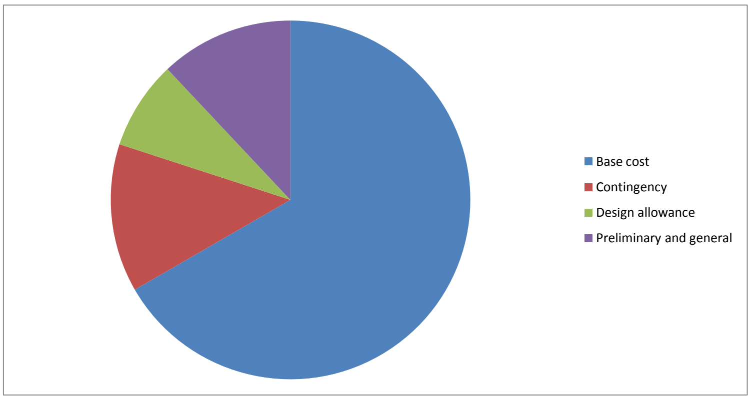
			Current Cost Estimate			Previous Cost Estimate			Percent Variation		
Item	Components	% Cost	Civil	E&M	Total	Civil	E&M	Total	Civil	E&M	Total
1	Site Preparation & Diversion Works	26.6%	$8,809,200	$0	$8,809,200	$10,316,730	$0	$10,316,730	-15%	100%	-15%
2	Plinth Work	10.2%	$3,398,325	$0	$3,398,325	$4,379,565	$0	$4,379,565	-22%	100%	-22%
3	Embankment	31.4%	$10,399,728	$0	$10,399,728	$14,916,046	$0	$14,916,046	-30%	100%	-30%
4	Spillway	24.1%	$7,990,635	$0	$7,990,635	$6,600,268	$0	$6,600,268	+21%	100%	+21%
5	Outlet Works	4.1%	$915,064	$434,868	$1,349,931	$941,034	$434,868	$1,375,902	-3%	0%	-2%
6	Fishpass	0.3%	$87,375	$0	$87,375	$87,375	$0	$87,375	+0%	100%	+0%
7	Access	3.4%	$1,127,258	$0	$1,127,258	$1,127,258	$0	$1,127,258	+0%	100%	+0%
	TOTALS	100.0%	$32,727,584	$434,868	$33,162,452	$38,368,275	$434,868	$38,803,143	-15%	0%	-15%

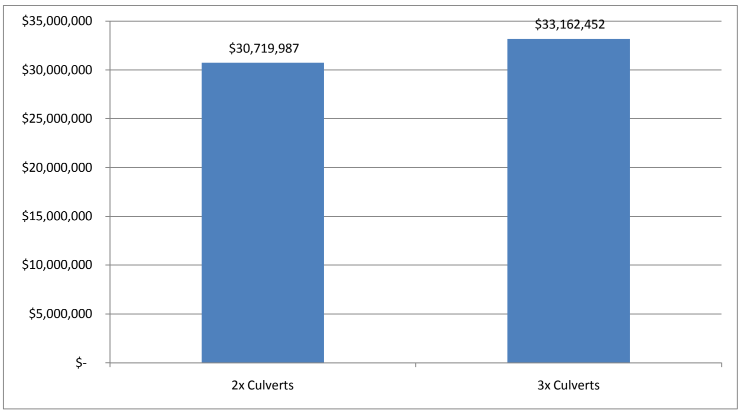
Base Cost Included Above	$ 21,818,389	$ 343,769	$ 22,162,158	Base cost
Contingency Allowance Included Above	$ 4,363,678	$ 34,377	$ 4,398,055	Contingency
Design Allowance Included Above	$ 2,618,207	$ 18,907	$ 2,637,114	Design allowance
P&G Allowance Included Above	$ 3,927,310	$ 37,815	$ 3,965,125	Preliminary and general
TOTALS	$ 32,727,584	$ 434,868	$ 33,162,452	

Contingency Allowance Calculated Explicitly $ 4,303,159.50

Costs with 2x Culverts

1	Site Preparation & Diversion Works	20.7%	$ 6,366,735.00	$ -	$6,366,735	2x Culverts	$ 30,719,987
2	Plinth Work	11.1%	$3,398,325	$0	$3,398,325	3x Culverts	$ 33,162,452
3	Embankment	33.9%	$10,399,728	$0	$10,399,728		
4	Spillway	26.0%	$7,990,635	$0	$7,990,635		
5	Outlet Works	4.4%	$915,064	$434,868	$1,349,931		
6	Fishpass	0.3%	$87,375	$0	$87,375		
7	Access	3.7%	$1,127,258	$0	$1,127,258		
	TOTALS		$ 30,285,119.05	$ 434,867.53	$ 30,719,986.58		





Description	Code	Unit	Rate
Earthworks & Related			
Soil cut to fill (Zone 1A)	cutToFill1A	cu.m	$ 5.00
Soil cut to fill (Zone 1B)	cutToFill1B	cu.m	$ 5.00
Soil cut to fill incl processing (Zone 2A)	cutToFill2A	cu.m	$ 20.00
Soil cut to fill incl processing (Zone 2B)	cutToFill2B	cu.m	$ 20.00
Rock cut to fill (Zone 3A)	cutToFill3A	cu.m	$ 10.00
Rock cut to fill (Zone 3B)	cutToFill3B	cu.m	$ 10.00
Rock cut to waste	cutToWasteRock	cu.m	$ 8.00
Soil cut to waste	cutToWasteSoil	cu.m	$ 4.00
Foundation cleanup for concrete placement	fndCleanConc	sq.m	$ 10.00
Road aggregate	roadAgg	cu.m	$ 60.00
Shotcrete slope protection	slopeProtect	sq.m	$ 100.00
100 dia HDPE pipe	pipeHDPE100	m	$ 20.00
Dowell anchor drilled into rock	anchorDowell	no	$ 200.00
Heavy rock armour	armRockHeavy	cu.m	$ 100.00
Treatment of foundation defects at plinth	defectTreat	no	$ 4,000.00
Drill and grout	drillGrout	m	$ 500.00
Geotextile	geoTex	sq.m	$ 10.00
Grouted rock	groutRock	cu.m	$ 150.00
750 dia concrete culvert	culvert750	m	$ 750.00
Sheet Piling	SteelSheetPile	t	$ 4,000.00
Bulk borrow to fill	borToFill	cu.m	$ 5.00
Liner placement	liner	cu.m	$ 12.00
Liner protection armour	linerPro	cu.m	$ 6.00
Wave armour	waveArm	cu.m	$ 12.00
Topsoil stripping	strip	cu.m	$ 2.00
Topsoiling and grassing	topsoil	sq.m	$ 2.00
Hydroseeding	hydroseed	sq.m	$ 2.00
Filter material	filter	cu.m	$ 60.00
Coarse filter	corFilter	cu.m	$ 60.00
Drainage material	drainage	cu.m	$ 60.00
Drainage pipe	drnPipe	m	$ 30.00
Steel sheet piling	SteelSheetPile	t	$ 4,000.00
Crest fence (farm type)	wireFence	m	$ 10.00
Heavy armour	armourHeavy	cu.m	$ 60.00
Coffer dam placement	cofferPlace	cu.m	$ 12.00
Coffer dam removal	cofferRemove	cu.m	$ 8.00
Piles	piles	m	$ 1,000.00
Heavy rubber bearings	heavyBearing	no	$ 4,000.00
Light rubber bearings	lightBearing	no	$ 2,000.00
Structural & Power Station Related			
Mass concrete	massConc	cu.m	$ 350.00
Structural concrete including reo	struConc	cu.m	$ 800.00
Formwork - straight	formStr	sq.m	$ 150.00
Formwork - curved	formCur	sq.m	$ 350.00
Formwork - slip formed	formSlip	sq.m	$ 100.00
Reinforcing steel	reoSteel	t	$ 3,200.00
Structural steel	strSteel	t	$ 5,000.00
Roller compacted concrete	RCC	cu.m	$ 150.00
Perimetric joint waterstop	periWS	m	$ 100.00
Waterstop joint	watStop	m	$ 50.00
Radial gate steel	steelRadGate	t	$ 18,000.00
Stoplog gate steel	steelStopLog	t	$ 10,000.00
General hydraulic structure steel	steelHydGen	t	$ 12,000.00
Steel pipe	steelPipe	t	$ 4,000.00
2 L/s capacity pump	puml2LS	No	$ 2,000.00
Road bridge cost based on plan area	bridgePlan	sq.m	$ 1,250.00
Retaining wall based on face area	retainWall	sq.m	$ 300.00
Instrumentation			
Flow monitoring equipment	flowMon	no	$ 1,500.00
Deformation marker	defMark	no	$ 1,000.00
Contingencies, percentages etc			
Civil minor items			5%
Civil contingency			20%
Civil engineering			10%
Civil P&G			15%
E&M minor items			5%
E&M contingency			10%
E&M engineering			5%
E&M P&G			10%

This sheet contains base information for titles, revs etc on all other sheets & is dynmically linked.

Project:	LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES	
JobNo:	24727.303	
Current Ver & date:	Ver 01b	19-Nov-09

Release History	Ver	Date	Notes
	00	19-Nov-09	First cut at construction cost estimate, prior to internal/external review
	01	9-Nov-09	Following constructor review and prelim optimisation
	01a	18-Nov-09	Recosting for reduced storage volume - NTWL 193.6mRL
	01b	19-Nov-09	Recosting for reduced storage volume - NTWL 189.9mRL

LEE VALLEY IRRIGATION STORAGE DAM - SCHEDULE OF QUANTITIES — Ver 01b
ITEM 1 Site Preparation & Diversion Works — 19-Nov-09

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
1.1	**Diversion Through Permanent Culverts**						
1.1.1	Excavation	cu.m	8,711.	cutToWasteSoil	$ 4	$ 34,844.00	
1.1.2	Diversion Culverts (Upstream Section)						
1.1.2.1	Concrete	cu.m	4,213.	massConc	$ 350	$ 1,474,550.00	
1.1.2.2	Steel	t	620.	reoSteel	$ 3,200	$ 1,984,000.00	
1.1.2.3	Formwork	sq.m	6,535.	formStr	$ 150	$ 980,250.00	
1.1.3	Diversion Culverts (Downstream Section)						
1.1.3.1	Concrete	cu.m	723.	massConc	$ 350	$ 253,050.00	
1.1.3.2	Steel	t	146.	reoSteel	$ 3,200	$ 467,200.00	
1.1.3.3	Formwork	sq.m	1,791.	formStr	$ 150	$ 268,650.00	
	Subtotal						$ 5,462,544.00
1.2	**Diversion Through Temporary Culverts (upstream end)**						
1.2.1	Upstream coffer dam	cu.m	1,800.	cutToWasteSoil	$ 4	$ 7,200.00	
1.2.2	50m long by 5 m high Sheet Piling	t	11.3	SteelSheetPile	$ 4,000	$ 45,200.00	
1.2.3	Miscellaneous sealing concrete	cu.m	20	massConc	$ 350	$ 7,000.00	
1.2.4	Dewatering allowance during low plinth construction	LS	1		$ 100,000	$ 100,000.00	
	Subtotal						$ 159,400.00
1.2	**Dam Site Preparation**						
1.2.1	Stripping (Cut to Waste)	cu.m	62,714.	cutToWasteSoil	$ 4	$ 250,856.00	
	Subtotal						$ 250,856.00
	SUBTOTAL CIVIL ITEMS					$ 5,872,800.00	
	Civil contingency allowance		$ 5,872,800.00		20%	$ 1,174,560.00	
	CIVIL INCL CONTINGENCY					$ 7,047,360.00	
	Civil Engineering		$ 7,047,360.00		10%	$ 704,736.00	
	Civil P&G		$ 7,047,360.00		15%	$ 1,057,104.00	
	TOTAL CIVIL						$ 8,809,200.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 8,809,200.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
2.1	**Preparation**						
2.1.1	Excavation (Cut to Fill)	cu.m	21,262.	cutToFill3B	$ 10	$ 212,620.00	
2.1.2	Foundation Cleaning (plan area)	sq.m	5,833.	fndCleanConc	$ 10	$ 58,330.00	
2.1.3	Defects (1 per 200 sq.m)	no	30.	defectTreat	$ 4,000	$ 120,000.00	
2.1.4	Slope Reinforcement (Face area of 0.25H:1V Slope)	sq.m	1,986.	slopeProtect	$ 100	$ 198,600.00	
	Subtotal						$ 589,550.00
2.2	**Plinth**						
2.2.1	Drilling and Grouting						
2.2.2.1	Length of 15m deep curtain grouting	m	250.	drillGrout	$ 500	$ 125,000.00	
2.2.2.2	Length of 7m deep side grouting	m	250.	drillGrout	$ 500	$ 125,000.00	
2.2.2	Grouted Anchor Bars (1m depth at 1m centers)	no	1,500.	anchorDowell	$ 200	$ 300,000.00	
2.2.3	Plinth						
2.2.3.1	Concrete	cu.m	750.	massConc	$ 350	$ 262,500.00	
2.2.3.2	Formwork	t	255.	reoSteel	$ 3,200	$ 816,000.00	
2.2.3.3	Steel	sq.m	150.	formStr	$ 150	$ 22,500.00	
2.2.4	Peremetric Joint (waterstop and Hypalon cover)	m	250.	periWS	$ 100	$ 25,000.00	
	Subtotal						$ 1,676,000.00
	SUBTOTAL CIVIL ITEMS					$ 2,265,550.00	
	Civil contingency allowance		$ 2,265,550.00		20%	$ 453,110.00	
	CIVIL INCL CONTINGENCY					$ 2,718,660.00	
	Civil Engineering		$ 2,718,660.00		10%	$ 271,866.00	
	Civil P&G		$ 2,718,660.00		15%	$ 407,799.00	
	TOTAL CIVIL						$ 3,398,325.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 3,398,325.00**

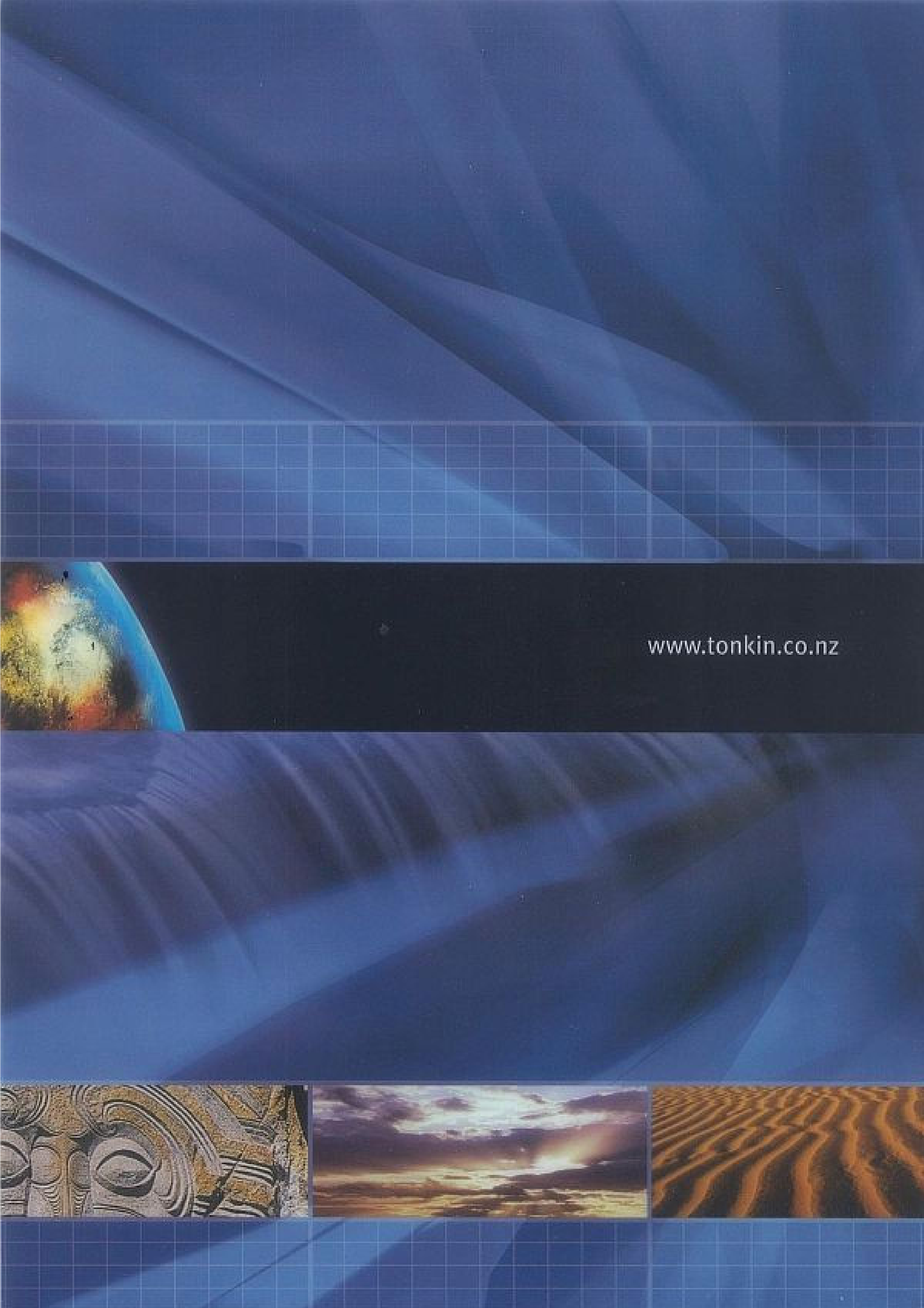
No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
3.1	**Rockfill**						
3.1.1	Zone 1A Cut to Fill	cu.m	0.	cutToFill1A	$ 5	$ -	
3.1.2	Zone 1A Borrow to Fill	cu.m	2,179.	cutToFill1A	$ 5	$ 10,895.00	
3.1.3	Zone 1B Cut to Fill	cu.m	10,691.	cutToFill1B	$ 5	$ 53,455.00	
3.1.4	Zone 1B Borrow to Fill	cu.m	0.	cutToFill1B	$ 5	$ -	
3.1.5	Zone 2A Cut to Fill	cu.m	0.	cutToFill2A	$ 20	$ -	
3.1.6	Zone 2A Borrow to Fill	cu.m	4,527.	cutToFill2A	$ 20	$ 90,540.00	
3.1.7	Zone 2B Cut to Fill	cu.m	0.	cutToFill2B	$ 20	$ -	
3.1.8	Zone 2B Borrow to Fill	cu.m	25,483.	cutToFill2B	$ 20	$ 509,660.00	
3.1.9	Zone 3A Cut to Fill	cu.m	153,444.	cutToFill3A	$ 10	$ 1,534,440.00	
3.1.10	Zone 3A Borrow to Fill	cu.m	0.	cutToFill3A	$ 10	$ -	
3.1.11	Zone 3B Cut to Fill	cu.m	92,366.	cutToFill3B	$ 10	$ 923,660.00	
3.1.12	Zone 3B Borrow to Fill	cu.m	0.	cutToFill3B	$ 10	$ -	
	Subtotal						$ 3,122,650.00
3.2	**Concrete Face**						
3.2.1	Concrete (250mm)	cu.m	2,076.3	massConc	$ 350	$ 726,705.00	
3.2.2	Formwork (15m slipformed panel areas)	sq.m	8,305.2	formSlip	$ 100	$ 830,520.00	
3.2.3	Steel	t	415.26	reoSteel	$ 3,200	$ 1,328,832.00	
3.2.4	Concrete underfill (100mm)	cu.m	830.52	massConc	$ 350	$ 290,682.00	
3.2.5	Vertical joint waterstop	m	415.26	watStop	$ 50	$ 20,763.00	
	Subtotal						$ 3,197,502.00
3.3	**Crest**						
3.3.1	Parapet Wall						
3.3.1.1	Concrete	cu.m	280.	massConc	$ 350	$ 98,000.00	
3.3.1.2	Formwork	sq.m	1,800.	formStr	$ 150	$ 270,000.00	
3.3.1.3	Steel	t	56.	reoSteel	$ 3,200	$ 179,200.00	
3.3.2	Road Aggregate (300mm)	cu.m	280.	roadAgg	$ 60	$ 16,800.00	
	Subtotal						$ 564,000.00
3.4	**Instrumentation**						
3.4.1	Flow monitoring equipment	no	6.	flowMon	$ 1,500	$ 9,000.00	
3.4.2	Deformation Markers (U/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
3.4.3	Deformation Markers (D/S)	no	20.	defMark	$ 1,000	$ 20,000.00	
	Subtotal						$ 49,000.00
3.5	**Electrical and Mechanical**						
3.5.1	Description						
3.5.1.1	Description				$ -	$ -	
	Subtotal						$ -
	SUBTOTAL CIVIL ITEMS					$ 6,933,152.00	
	Civil contingency allowance		$ 6,933,152.00		20%	$ 1,386,630.40	
	CIVIL INCL CONTINGENCY					$ 8,319,782.40	
	Civil Engineering		$ 8,319,782.40		10%	$ 831,978.24	
	Civil P&G		$ 8,319,782.40		15%	$ 1,247,967.36	
	TOTAL CIVIL						$ 10,399,728.00
	SUBTOTAL TOTAL E&M ITEMS					$ -	
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 10,399,728.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
4.1	**Bulk Earthworks**						
4.1.1	Cut to Waste - Soil	cu.m	133,771.	cutToWasteSoil	$ 4	$ 535,084.00	
4.1.2	Cut to Waste - Rock	cu.m	197,693.	cutToWasteRock	$ 8	$ 1,581,544.00	
4.1.3	Slope protection and reinforcement	sq.m	1,000.	slopeProtect	$ 100	$ 100,000.00	
	Subtotal						$ 2,216,628.00
4.2	**Fuse Embankment**						
4.2.1	Armoring to U/S Face (0.5m thk)	cu.m	199.	armourHeavy	$ 60	$ 11,940.00	
4.2.2	Filter Layer (0.5m thk)	cu.m	199.	cutToFill2A	$ 20	$ 3,980.00	
4.2.3	Inclined Geotextile	sq.m	398.	geoTex	$ 10	$ 3,980.00	
4.2.4	Bulk Fill	cu.m	3,493.	cutToFill2A	$ 20	$ 69,860.00	
4.2.5	Concrete Slab						
4.2.5.1	Concrete	cu.m	768.	massConc	$ 350	$ 268,800.00	
4.2.5.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.5.3	Steel (0.1t/cu.m)	t	76.8	reoSteel	$ 3,200	$ 245,760.00	
4.2.6	Approach Slab						
4.2.6.1	Concrete	cu.m	216.	massConc	$ 350	$ 75,600.00	
4.2.6.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.2.6.3	Steel (0.1t/cu.m)	t	21.6	reoSteel	$ 3,200	$ 69,120.00	
	Subtotal						$ 749,040.00
4.3	**Primary Spillway Chute**						
4.3.1	Ogee Weir						
4.3.1.1	Concrete	cu.m	278.75	massConc	$ 350	$ 97,562.50	
4.3.1.2	Formwork	sq.m	178.4	formCur	$ 350	$ 62,440.00	
4.3.1.3	Steel (0.05t/cu.m)	t	13.9	reoSteel	$ 3,200	$ 44,480.00	
4.3.2	Approach Slab						
4.3.2.1	Concrete	cu.m	164.5	massConc	$ 350	$ 57,575.00	
4.3.2.2	Formwork	sq.m	0.	formStr	$ 150	$ -	
4.3.2.3	Steel (0.1t/cu.m)	t	16.45	reoSteel	$ 3,200	$ 52,640.00	
4.3.2.4	Foundation preparation	sq.m	658.	fndCleanConc	$ 10	$ 6,580.00	
4.3.3	Chute Floor						
4.3.3.1	Concrete	cu.m	499.	massConc	$ 350	$ 174,650.00	
4.3.3.2	Formwork (Slipformed)	sq.m	758.5	formSlip	$ 100	$ 75,850.00	
4.3.3.3	Steel (0.1t/cu.m)	t	49.9	reoSteel	$ 3,200	$ 159,680.00	
4.3.3.4	Waterstop joints	m	400.	watStop	$ 50	$ 20,000.00	
4.3.3.5	Foundation preparation	sq.m	760.	fndCleanConc	$ 10	$ 7,600.00	
4.3.4	Chute Walls						
4.3.4.1	Concrete	cu.m	382.6	massConc	$ 350	$ 133,910.00	
4.3.4.2	Formwork	sq.m	1,275.33	formStr	$ 150	$ 191,300.00	
4.3.4.3	Steel (0.2t/cu.m)	t	76.52	reoSteel	$ 3,200	$ 244,864.00	
4.3.5	Wall Backfill	cu.m	1,087.	cutToFill2A	$ 20	$ 21,740.00	
4.3.6	Chute Under Drainage (drains at 10m centers)						
4.3.6.1	Filter Material	cu.m	11.21	roadAgg	$ 60	$ 672.60	
4.3.6.2	Pipe (100 dia HDPE perforated)	m	170.	pipeHDPE100	$ 20	$ 3,400.00	
4.3.9	Chute Slab Anchors (1 Per 10 sq.m)	no	166	anchorDowell	$ 200	$ 33,266.67	
	Subtotal						$ 1,388,210.77
4.4	**Flip Bucket**						
4.4.1	Bucket						
4.4.1.1	Concrete	cu.m	681.5	massConc	$ 350	$ 238,525.00	
4.4.1.2	Formwork and curved surface formation	sq.m	438.16	formCur	$ 350	$ 153,356.00	
4.4.1.3	Steel (0.1t/cu.m)	t	68.15	reoSteel	$ 3,200	$ 218,080.00	
4.4.2	Bucket Rock Anchors (6 assumed)	no	6.	anchorDowell	$ 200	$ 1,200.00	
4.4.3	Low flow channel lining						
4.4.3.1	Concrete	cu.m	375.	massConc	$ 350	$ 131,250.00	
4.4.3.2	Formwork (Slipformed)	sq.m	0.	formStr	$ 150	$ -	
4.4.3.3	Steel (0.05t/cu.m)	t	18.75	reoSteel	$ 3,200	$ 60,000.00	
	Subtotal						$ 802,411.00
4.5	**Plungepool**						
4.5.1	Excavation	cu.m	2600	cutToWasteRock	$ 8	$ 20,800.00	
4.5.2	Rock Armour	cu.m	1,500.	armRockHeavy	$ 100	$ 150,000.00	
	Subtotal						$ 170,800.00
	SUBTOTAL CIVIL ITEMS					$ 5,327,089.77	
	Civil contingency allowance		$ 5,327,089.77		20%	$ 1,065,417.95	
	CIVIL INCL CONTINGENCY					$ 6,392,507.72	
	Civil Engineering		$ 6,392,507.72		10%	$ 639,250.77	
	Civil P&G		$ 6,392,507.72		15%	$ 958,876.16	
	TOTAL CIVIL						$ 7,990,634.65
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 7,990,634.65**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
5.1	**Inclined Inlets**						
5.1.1	Conduits on U/S Face						
5.1.1.1	Concrete	cu.m	139.94	massConc	$ 350	$ 48,979.00	
5.1.1.2	Formwork (may be precast)	sq.m	921.	formStr	$ 150	$ 138,150.00	
5.1.1.3	Steel (0.2t/cu.m)	t	27.99	reoSteel	$ 3,200	$ 89,561.60	
5.1.3	Rails embedded in upstream face	t	1.66	strSteel	$ 5,000	$ 8,300.00	
	Subtotal						$ 284,990.60
5.2	**Outlet Conduits**						
5.2.1	Box Inlet Plug						
5.2.1.1	Concrete	cu.m	105.	massConc	$ 350	$ 36,750.00	
5.2.1.2	Formwork (may be precast)	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.1.3	Steel (0.05t/cu.m)	t	5.25	reoSteel	$ 3,200	$ 16,800.00	
5.2.2	Gate Structure						
5.2.2.1	Concrete	cu.m	37.5	massConc	$ 350	$ 13,125.00	
5.2.2.2	Formwork	sq.m	75.	formStr	$ 150	$ 11,250.00	
5.2.2.3	Steel (0.1t/cu.m)	t	3.75	reoSteel	$ 3,200	$ 12,000.00	
5.2.3	Overhead Beam	t	8.46	strSteel	$ 5,000	$ 42,282.00	
	Subtotal						$ 143,457.00
5.3	**Outlet Structure**						
5.3.1	Outlet Stoplog Concrete Structure						
5.3.1.1	Concrete	cu.m	12.5	massConc	$ 350	$ 4,375.00	
5.3.1.2	Formwork	sq.m	25.	formStr	$ 150	$ 3,750.00	
5.3.1.3	Steel	t	1.25	reoSteel	$ 3,200	$ 4,000.00	
5.3.2	Outlet Wingwall Structure						
5.3.2.1	Concrete	cu.m	143.	massConc	$ 350	$ 50,050.00	
5.3.2.2	Formwork	sq.m	186.	formStr	$ 150	$ 27,900.00	
5.3.2.3	Steel (0.2t/cu.m)	t	28.6	reoSteel	$ 3,200	$ 91,520.00	
	Subtotal						$ 181,595.00
5.4	**Gates, Screens and Related**						
5.4.1	Removable Bellmouth Inlet and Screen	t	5.997	steelHydGen	$ 12,000	$ 71,968.80	
5.4.2	Removable Stoplogs for intake structure	t	3.25	steelStopLog	$ 10,000	$ 32,500.00	
5.4.3	Stoplog and Screen Derrick and Winch for intake structure	t	5.	steelHydGen	$ 12,000	$ 60,000.00	
5.4.4	Radial Gates 1m x 1m x 2 gates for irrigation outlet	t	9.29	steelHydGen	$ 12,000	$ 111,480.00	
5.4.5	Tailrace area outlet Stoplogs	t	6.78	steelStopLog	$ 10,000	$ 67,820.00	
	Subtotal						$ 343,768.80
	SUBTOTAL CIVIL ITEMS					$ 610,042.60	
	Civil contingency allowance		$ 610,042.60		20%	$ 122,008.52	
	CIVIL INCL CONTINGENCY					$ 732,051.12	
	Civil Engineering		$ 732,051.12		10%	$ 73,205.11	
	Civil P&G		$ 732,051.12		15%	$ 109,807.67	
	TOTAL CIVIL						$ 915,063.90
	SUBTOTAL TOTAL E&M ITEMS					$ 343,768.80	
	E&M contingency allowance		$ 343,768.80		10%	$ 34,376.88	
	E&M INCL CONTINGENCY					$ 378,145.68	
	E&M Engineering		$ 378,145.68		5%	$ 18,907.28	
	E&M P&G		$ 378,145.68		10%	$ 37,814.57	
	TOTAL E&M						$ 434,867.53
	TOTAL						**$ 1,349,931.43**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
6.1	**Fishpass**						
6.1.1	Pump (2l/s)	No	1.	puml2LS	$ 2,000	$ 2,000.00	
6.1.2	Grouted rock channel	cu.m	250.	groutRock	$ 150	$ 37,500.00	
6.1.3	750 mm Culvert Crossing	m	10.	culvert750	$ 750	$ 7,500.00	
6.1.4	U/S Face Sluice	m	15.	culvert750	$ 750	$ 11,250.00	
	Subtotal						$ 58,250.00
	SUBTOTAL CIVIL ITEMS					$ 58,250.00	
	Civil contingency allowance		$ 58,250.00		20%	$ 11,650.00	
	CIVIL INCL CONTINGENCY					$ 69,900.00	
	Civil Engineering		$ 69,900.00		10%	$ 6,990.00	
	Civil P&G		$ 69,900.00		15%	$ 10,485.00	
	TOTAL CIVIL						$ 87,375.00
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 87,375.00**

No	Description	Unit	Quantity	Rate Code	Rate	Amount	Subtotals
7.1	**Power Station Access Road**						
7.1.1	Surfacing Aggregate (250 mm)	cu.m	742.5	roadAgg	$ 60	$ 44,550.00	
	Subtotal						$ 44,550.00
7.2	**Auxillary Spillway Access**						
7.2.1	Surfacing Aggregate (250 mm)	cu.m	132.	roadAgg	$ 60	$ 7,920.00	
	Subtotal						$ 7,920.00
7.3	**Lee Valley Road Diversion**						
7.3.1	Surfacing Aggregate (250 mm)	cu.m	767.25	roadAgg	$ 60	$ 46,035.00	
	Subtotal						$ 46,035.00
7.4	**Spillway Bridging**						
7.4.1	Auxillary Spillway Bridge (10 m Spans)	sq.m	146.	bridgePlan	$ 1,250	$ 182,500.00	
7.4.2	Abutment Retaining	sq.m	251.	retainWall	$ 300	$ 75,300.00	
7.4.3	Primary Spillway Bridge (10 m Spans)	sq.m	160.	bridgePlan	$ 1,250	$ 200,000.00	
7.4.4	Chute Bucket Bridge (10 m Span)	sq.m	106.	bridgePlan	$ 1,250	$ 132,500.00	
7.4.5	Abutment Retaining	sq.m	209.	retainWall	$ 300	$ 62,700.00	
	Subtotal						$ 653,000.00
	SUBTOTAL CIVIL ITEMS					$ 751,505.00	
	Civil contingency allowance		$ 751,505.00		20%	$ 150,301.00	
	CIVIL INCL CONTINGENCY					$ 901,806.00	
	Civil Engineering		$ 901,806.00		10%	$ 90,180.60	
	Civil P&G		$ 901,806.00		15%	$ 135,270.90	
	TOTAL CIVIL						$ 1,127,257.50
	SUBTOTAL TOTAL E&M ITEMS						
	E&M contingency allowance		$ -		10%	$ -	
	E&M INCL CONTINGENCY					$ -	
	E&M Engineering		$ -		5%	$ -	
	E&M P&G		$ -		10%	$ -	
	TOTAL E&M						$ -
	TOTAL						**$ 1,127,257.50**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