REPORT

Waimea Water Augmentation Committee

Lee Valley Dam
Detailed Design Report Stage 3

Report prepared for:

Waimea Water Augmentation Committee

Report prepared by:

Tonkin & Taylor Ltd

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Waimea Water Augmentation Committee	**1 copy (+ PDF)**
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Executive summary

The report describes the Stage 3 design of the Lee Valley Dam. The dam is a Concrete Face Rockfill Dam (CFRD) which is approximately 53 m high at its highest point. The dam would be located on the Lee River (a tributary of the Waimea River). It is being developed by a community backed committee known as the Waimea Water Augmentation Committee (WWAC). The dam is intended to contain a reservoir to be used for water augmentation purposes, for provision of irrigation and community water supply for Waimea Plains and environs, and to augment water flows in the Lee, Wairoa and Waimea Rivers during low flow periods for environmental, cultural and aesthetic purposes.

The current design adopts WWAC's storage criteria of 13 Mm^3 of which there is a 1 Mm^3 dead storage allowance. The dam is intended to supplement the Lee River's natural flows to provide a minimum residual flow as well as water supply flow. Reticulation and subsequent use of the water is outside the scope of the project.

The dam is classified as a high Potential Impact Category or Classification (PIC) dam in accordance with New Zealand Society on Large dams Dam Safety Guidelines (NZSOLD, 2000). The dam is therefore designed to the highest standards currently applicable in New Zealand for dams.

The Stage 3 design is intended to be an "80%" design, i.e. approximately 80% complete.

We understand that the WWAC will make decisions on the following tasks upon receipt of this report:

1. Whether to proceed with the Resource Consent application process
2. Whether to proceed with the final Stage 4 design (i.e. completion of detailed design, suitable for tendering and construction)
3. We understand that WWAC may await the outcome of a district Plan Change currently being progressed by others before deciding on Tasks 1 or 2 (above).

The reservoir will be impounded by a CFRD at a location of Ch 12,430 m (measured upstream from the confluence of the Wairoa and Lee Rivers). The dam would be approximately 53 m high and 210 m long at crest level. The storage reservoir will have a normal top water level of RL 197.2 m and will extend approximately 3.7 km upstream of the dam. Arms of the reservoir will extend approximately 1 km into Waterfall Creek on the right bank, and 350 m into Flat Creek on the left bank.

The proposed dam site is located within Rai Formation greywacke sandstone and siltstone/mudstone basement rocks within the Caples Terrane. The site is flanked by the Gordon Range to the west and the Richmond Range to the east.

The dam site is very likely to have been affected by significant earthquake shaking as nearby active faults include the Waimea Fault 8.5 km to the NW and the Wairau segment of the Alpine Fault 20 km to the SE.

The geology underlying the dam site comprises variably weathered, moderately strong to strong, very close spaced to closely spaced jointed fine sandstone and siltstone of the Rai Formation. The rock has been separated into three rock classes based on rock mass condition. These are summarised below:

1. Class 1 – Unweathered, strong to very strong, blocky to very blocky rock mass
2. Class 2 – Slightly weathered, moderately strong to strong, very blocky rock mass

3. Class 3 – Moderately to highly weathered, weak to moderately strong, very blocky rock mass

Foundation requirements for most of the dam require that the rock is stiffer than the rock fill. This should be achieved by all classes of rock encountered and no surface treatment should be needed for the majority of the dam footprint. Higher quality rock will need to be exposed downstream for the foundation and anchorage of mesh protection.

The CFRD external batter slopes of 1.0V: 1.5H have been used to provide a degree of conservatism for the high earthquake loads. They also allow the use of a processed gravel in the upstream Zone 2B and the use of coarse gravel material in downstream Zone 4.

Foundation treatment will include curtain and blanket grouting to reduce seepage caused by foundation disturbance during excavation, and reduce seepage along defects.

The dam rockfill will be obtained from excavations for the spillway and, to a lesser extent, from the diversion conduit and road excavations. While the selected high quality zone of Class 3 rock is probably acceptable for embankment construction, there is expected to be sufficient Class 1 and 2 material available and these materials have been specified.

Seismic analysis has been undertaken for the proposed dam. We consider that this shows that the requirements of NZSOLD Dam Safety Guidelines have been met. The permanent deformations estimated are of an order that would not be expected to compromise the dam function at the Operational Base Earthquake (OBE) level. The displacements estimated (20 mm to 35 mm) would be expected to be accommodated by the dam structure and result in little significant damage. At the Maximum Design Earthquake (MDE) level event, the permanent deformations estimated (400 mm to 510 mm) would likely contribute to damage to the embankment structure, with cracking in the dam face, and in the parapet wall. The damage associated with the permanent deformations would not be expected to be sufficient to compromise the required performance of the embankment immediately following the seismic event. However, they are likely to compromise the performance of the embankment to the extent that repair, potentially of a very significant nature, would be required for the embankment to remain in service.

Geological investigations (T&T 2012) for the dam identified a number of potential slope instability or landslide features around the potential reservoir. The two landslides considered to pose the most significant risk to the dam were then selected for hydrodynamic modelling to more accurately predict their impact on the dam. The modelling concludes that there is adequate freeboard to accommodate the modelled landslide generated waves.

The selected river diversion comprises the following main components. A concrete culvert with two rectangular barrels, each 2.5 m wide by 4 m high and approximately 165 m long. A main coffer dam with a crest at 173.4 mRL, 6 m wide, located in the downstream shoulder of the permanent rockfill embankment. This main coffer dam is described as the "downstream stage" and will comprise reinforced rockfill (also described as "meshing") designed to enable floods to flow over and through the embankment without failure.

The primary spillway is a reinforced concrete chute located on the dam left abutment terminating in a flip bucket to dissipate energy. The weir crest level is 197.2 mRL (NTWL) with a minimum approach depth of 2.5 m. During the design flood (OBF) the operating head will be 3.3 m. The operational design flow will be 472 m^3/s during the OBF. The spillway is designed to pass the MDF (PMF) flood with 300 mm freeboard to the top of the parapet wall. Spot bolting of the spillway cut slopes will likely be required to prevent rock falls.

The outlet works comprise two 1200 mm diameter inclined steel intake pipes located on the upstream face of the dam. The pipes transition through the diversion conduit exiting at the downstream end. The sizing of the outlet works are also based on the expected requirements for environmental flows, i.e. a minimum residual flow at the base of the dam of 511 l/s and provision for flushing flows of 5000 l/s. The maximum irrigation release from the dam is 2230 l/s.

WWAC has requested that the dam allow for the future addition of a mini hydro power station at the toe of the dam. The power station addition was identified during the feasibility study (T&T 2009), but only developed (at that time) to a pre-feasibility level. The current Stage 3 design of the dam incorporates the necessary valve and penstock arrangements to supply water to a future power station. The feasibility arrangements and analysis are for a below ground twin Francis turbine powerstation located at the end of the diversion culverts. The installed combined capacity is 1.2 MW and would generate approximately 6.1 GWh per annum. A transmission line of 22 KV would be required to be constructed for this scheme to be operable. The design of the power station, transmission line, switchyard etc. itself remains outside the scope of T&T's current design and this report. We understand however, that WWAC intends that the construction of the power station would occur at the same time as the dam itself.

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Pages	**Date**	**Issue No**	**Description**	**Initials**
	03/12	Draft	Wave climate, wind speed, ogee weir, spillway and flip bucket,	JICR
	04/12	Draft	Bridge design	MCNT
	06/12	Draft	Road design	MFP
	05/12	Draft	River diversion strategy	DMK
	04/12	Draft	Embankment stability	GRT
	05/12	Draft	Conduit structural design	MCNT
	07/12	Draft	M&E and Hydro design	NML
	13/07/12	Draft	Fish pass and landslides	SGB
	13/07/12	Draft	Crest wall design	MCNT
	17/07/12	Draft	Spillway Weir Design, Flipbucket Design	JICR
	21/07/12	Draft	Starter dam. Update peer review comments. update table and figure numbers	MCNT
	21/07/12	Draft	Plunge pool section	KKSN
	21/07/12	Draft	Diversion section completed	DMK
	23/07/12	Draft	Hydrology peer review questions answered	RDM
	25/07/12	Draft	Phil Carter - embankment and plinth design	PC
	26/07/12	Draft	Mini hydro and outlet works	NML
	30/07/12	Draft	SFC spillway	SFC
	30/07/12	Draft	Issued to Len McDonald for review	MCNT
	31/07/12	Draft	SFC completion of spillway section	SFC
	23/08/12	Draft	GHR, KJD, MCBR and Len McDonald comments received and addressed	MCNT/SFC
	13/09/12	Draft	SLM high level review for project consistency with feasibility assumptions	SLM
	22/09/12	Draft	Introduction and executive summary expansion	MCNT
	12/10/12	Draft	KJD PD review	MCNT
	19/10/12	Issue 1	Issued for peer review	MCNT

Abbreviations

Term	Definition
AEP	Annual Exceedance Probability
ALARP	As low as reasonably practicable
ARI	Average Recurrence Interval
Cd	Coefficient of Discharge
CDF	Construction Diversion Flood
CFRD	Concrete Face Rockfill Dam
CL	Centreline
d/s	Downstream
DHI	DHI Water & Environment Ltd
DSAP	Dam Safety Assurance Programme
DSEP	Dam Safety Emergency Plan
ESI	Earthquake Severity Index
Fletcher	Fletcher Construction Company
FOS	Factor of Safety
GCM	General Circulation Models
GNS	GNS Science (NZ)
GWh	Gigawatt hour
H&S	Health & Safety
H.E.C.	Hydro-Electric Commission
HAZOP	Hazard and Operations
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modelling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
Hs	Significant wave height
ICOLD	International Commission on Large Dams
IPCC	Intergovernmental Panel on Climate Change
LiDAR	Light Detection And Ranging
MAF	Mean Annual Flood
MALF	Mean Annual Low Flow
MDE	Maximum Design Earthquake
MDF	Maximum Design Flood
MDFL	Maximum Design Flood Level
MfE	Ministry for the Environment (NZ)
mRL	metres Reduced Level
NIWA	National Institute Of Water & Atmospheric Research
HIRDS	High Intensity Rainfall Design System

NPV	Net Present Value
NTWL	Normal Top Water Level
NWL	Normal Water Level
NZSOLD	New Zealand Society on Large Dams
NZTA	New Zealand Transport Agency
O&M	Operations & Maintenance
OBE	Operational Basis Earthquake
OBF	Operational Basis Flood
OBFL	Operational Basis Flood Level
PAR	Population at Risk
PB	Parsons Brinckerhoff
PGA	Peak Ground Acceleration
PIC	Potential Impact Category or Classification
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSF	Peak Sliding Factor
R/y	Flow Depth Ratio
RCC	Roller Compacted Concrete
RS	Run up of significant wave
RMR	Rock Mass Rating
RQD	Rock Quality Designation
RSF	Residual Sliding Factor
T&T	Tonkin & Taylor
Tp	Wave period
TPA	Test Pit [NAME]
UDL	Uniformly Distributed Load
USACE	US Army Corps of Engineers
USBR	US Bureau of Reclamation
WL	Water Level
WWAC	Waimea Water Augmentation Committee

1 Introduction

1.1 General

This report summarises the Stage 3 design undertaken for the proposed Lee Valley Dam, Tasman District. It is being developed by a community backed committee known as the Waimea Water Augmentation Committee (WWAC). The dam is intended for use as an irrigation dam to provide drought security to the Waimea Plains. The dam's purpose is water augmentation for irrigation and community water supply. The dam is intended to supplement the Lee River's natural flows to provide a constant residual flow as well as an irrigation flow. The proposed dam site is located on the Lee River approximately 40 minutes by car to the south of Nelson. The Lee River is a tributary of the Waimea River.

A CFRD up to approximately 53 m high is currently proposed to store water in the Lee River headwaters for distribution as irrigation on the Waimea Plains. The proposed dam is located at chainage (horizontal distance measured) 12,430 m upstream from the confluence between the Wairoa and Lee rivers. The dam location is shown in Figures 1.1 and 1.2. The Lee Valley Dam site is accessed by forestry roads off Lee Valley Road as shown on Figure 1.2.

The approximate NZTM coordinates of the dam location are:

1613437 Latitude, 5409020 Longitude.

Figure 1.1: Location of the proposed dam site

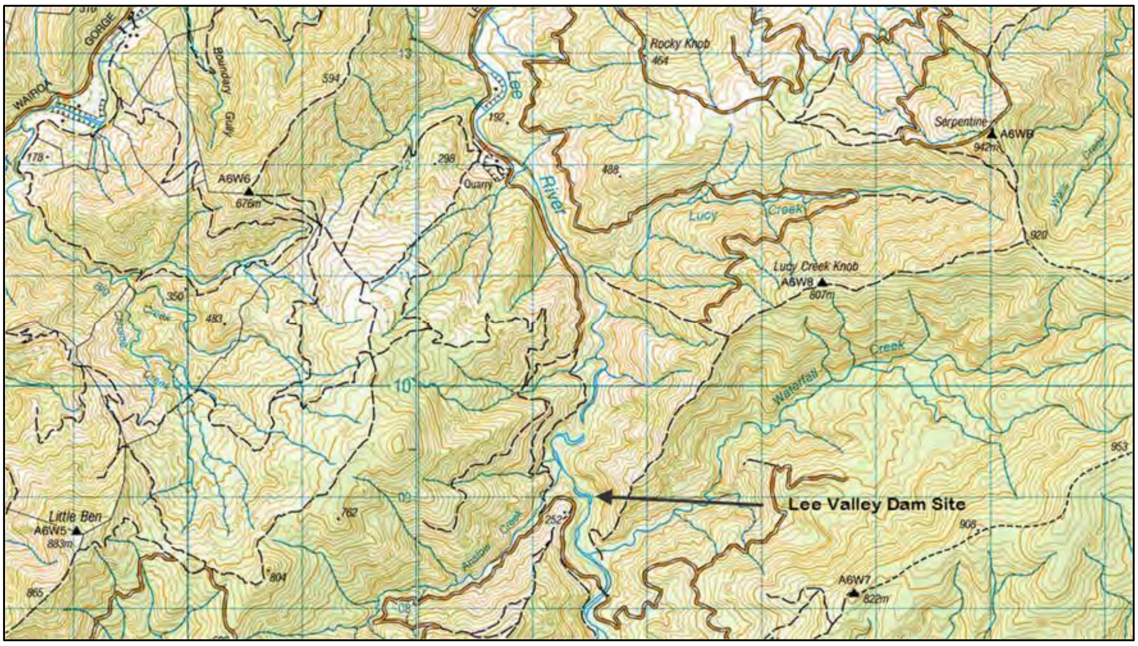


Figure 1.2: Location of the proposed dam site on the Lee River.

1.2 Proposed dam

The project comprises the construction of a dam and 13 Mm^3 reservoir in the upper Lee Valley approximately 200 m upstream of Anslow Creek. The Lee River is one of two major tributaries of the Wairoa River which drains the Richmond Range east of the Waimea Plains. The Wairoa River is then joined by the Wai-iti River, and together they form the Waimea River.

The reservoir will be impounded by a CFRD at a location of Ch 12,430 m (measured upstream from the confluence of the Wairoa and Lee Rivers). The dam would be approximately 53 m high and 210 m long at crest level. The location and layout of the dam is shown in Figure 1.3. The storage reservoir will have a top water level of RL197.2 m and will extend approximately 3.7 km upstream of the dam. Arms of the reservoir will extend approximately 1 km into Waterfall Creek on the right bank, and 350 m into Flat Creek on the left bank. The reservoir will be drawn down to about RL 166.5 m during periods of river augmentation draw-off.

The dam is classified as a high PIC (Potential Impact Category or Classification) dam in accordance with New Zealand Society on Large Dams Dam Safety Guidelines (NZSOLD, 2000). The dam is therefore designed to the highest standards currently applicable in New Zealand for dams.

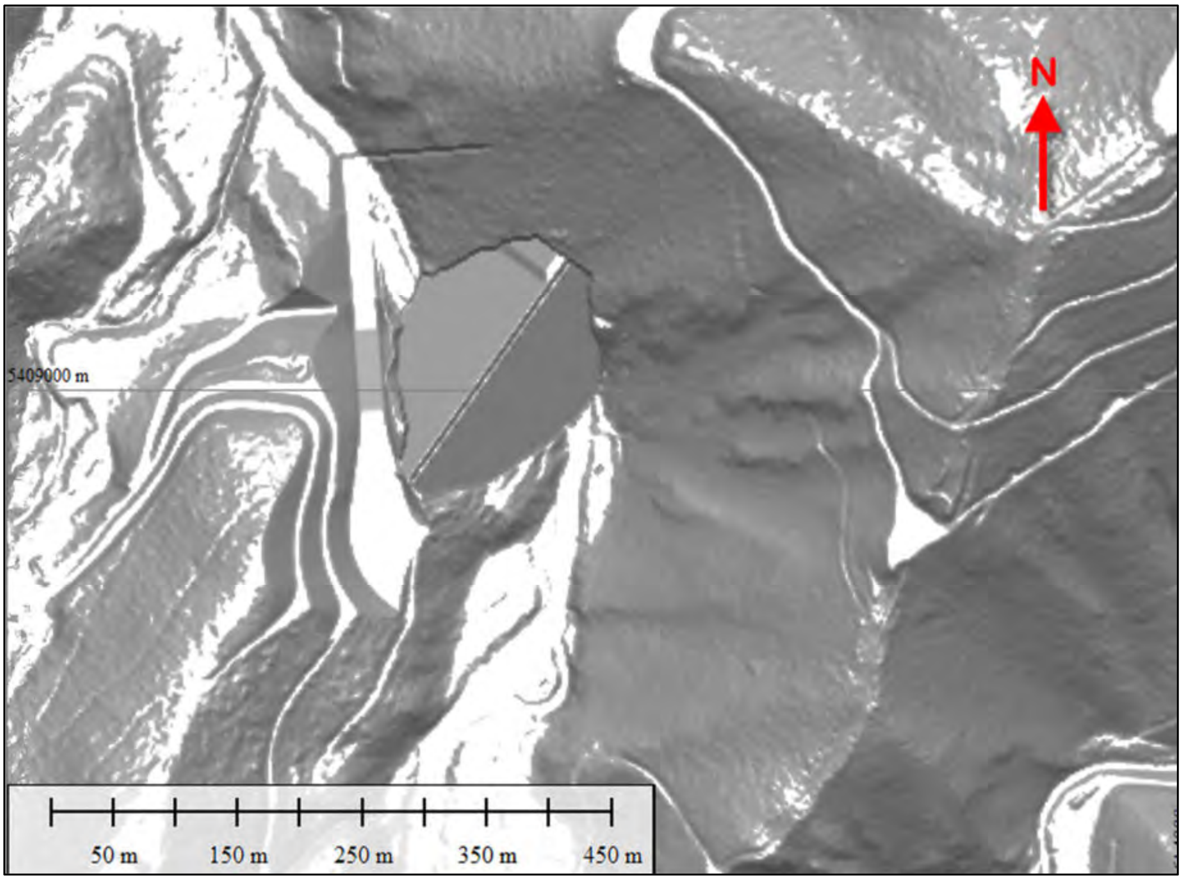


Figure1.3: Location and layout of the proposed dam.

The proposed CFRD will be constructed from approximately 430,000 m^3 of locally sourced rockfill. Structures associated with the dam include a spillway and a diversion conduit that will be utilised after construction as the irrigation off-take.

1.3 Background, staged design process and peer review

1.3.1 Background

WWAC engaged T&T to undertake both pre-feasibility and feasibility designs of the irrigation dam. Feasibility was completed in December 2009. The feasibility study concluded that a CFRD at chainage 12,430 m was the most appropriate location and dam type.

In December 2010 T&T was appointed to undertake detailed design of the dam. The current detailed design phase has not re-considered dam type, location or dam storage volume requirements. This is because the type, storage and location were peer reviewed and endorsed by WWAC prior to completion of feasibility.

Key T&T engineering reports documenting the feasibility design are listed in Table 1.1. The reader should be aware that many more non-engineering investigations and reports were produced as part of the Engineering Feasibility Report (T&T 2009). These are summarised in the report entitled "Waimea Water Augmentation Phase 2 - Lee Valley Dam Feasibility Investigations - Summary Report T&T ref 24727.800". Pre-feasibility documents are not listed in Table 1.1 because they investigated regional solutions for water augmentation and are therefore not directly relevant to the current detailed design phase.

Table 1.1 Key feasibility design reports

Title	Date	T&T Reference
Lee Valley Dam Feasibility Investigations Geotechnical Investigations Report	December 2009	24727.204
Lee Valley Storage Dam Engineering Feasibility Report	December 2009	24727.303
Waimea Water Augmentation Phase 2 - Water Resource Investigations	December 2009 (Rev 1.0)	24727.100
Waimea Water Augmentation Phase 2 - Lee Valley Dam Feasibility Investigations - Summary Report	February 2010	24727.800

1.3.2 Staged design process

The design and investigation of the dam has been carried out in a staged approach in accordance with the T&T proposal. The intent of this approach is to keep both WWAC and WWAC's peer reviewer informed of the design as it progresses. Furthermore, the feedback from WWAC and the peer reviewer enables that feedback to be incorporated into subsequent design stages. The staged design process is illustrated in Figure 1.4.

This document describes the detailed design aspects of Stage 3 of the Lee Valley Dam.

Stage 3 design is primarily concerned with detailed hydraulic, geotechnical, and general civil design. In terms of structural design, this has been carried out to a level that enables the overall size of concrete and steel members to be determined and the mass of reinforcing steel to be estimated to a reasonable degree of accuracy.

Stage 4 design will cover full structural design and detailing and an update, as necessary, of the design report.

1.3.3 Peer review

In accordance with NZSOLD (2000) WWAC has commissioned a peer review of the current detailed design of the Lee Valley Dam. WWAC initially engaged MWH to independently review (The feasibility design had already been peer reviewed by Engineering Geology Ltd) the Engineering Feasibility Report (T&T 2009). Subsequently Opus International Consultants Ltd (Opus) was appointed by WWAC as their peer reviewer for detailed design in mid May 2011.

Table 1.2 summarises the delivery of key reports and meetings with WWAC, the peer reviewer, the date the peer review was received and the date of any response. This illustrates the involvement that the peer reviewer(s) has had to date with the design.

In addition to the key documents listed in Table 1.2, there have been occasional email and telephone discussions between T&T and Opus.

Peer review comments received from Opus that have not already been addressed have been considered at the end of each section related to the comment. Comments from peer review that concur with the design have not been repeated in this report. Opus peer review letters are contained in Appendix **B**.

Table 1.2 Summary of peer review since completion of feasibility

Description	Communication type	Date issued	Peer reviewer	Date peer review received	Date responded	Comment
Feasibility						
Lee Valley Storage Dam Engineering Feasibility Report	Report	December 2009	MWH	July 2009	6 October 2010	The feasibility report had already been reviewed by Engineering Geology Ltd
Detailed design						
Lee River Dam Project Quality Plan	Report	February 2011	Opus	NA	NA	Issued for information only
Lee River Dam Detailed Design Criteria	Report	19 May 2011	Opus	27 January 2012	Stage 3 Design Report	Issued to WWAC in April 2011
Lee Valley Dam Design Stage Geotechnical Investigation Programme and Methodology	Memo	19 May 2011	Opus	3 June 2011	12 September 2011	Issued to WWAC on 27 January 2011 Response to Peer review was in letter entitled "Response to Peer review of Lee Valley Dam Design" dated 12 September 2012
Lee Valley Dam Geotechnical Investigations – Technical Review Visit	Memo	19 May 2011	Opus	3 June 2011	12 September 2011	
Peer Review Site Visit accompanied by Mark Foley of T&T, and associated discussions including Joseph Thomas.	Site visit	1 June 2012	Opus	3 June 2011	12 September 2011	
Stage 1 Design Report	Report	September 2011	Opus	25 October 2011	Stage 3 Design Report	Informal discussions have been had with Opus on specific aspects (e.g. climate change assumptions)
Discussion Paper on Procurement and Delivery Options	Report	28 June 2011	Opus	NA	NA	The discussion paper was discussed at Contractual workshop on 27 March 2012
Risk Workshop	Workshop	26 October 2011	Opus	NA	NA	Risk workshop was interactive rather than having a formal review output. Minutes issued to all attendees
Lee River Dam – Hydropower design and interfacing	Letter	18 January 2012	Opus	8 February 2012	NA	The Opus peer review was for WWAC's benefit. i.e. The comment did not

						require a response from T&T
Contractual procurement workshop	Workshop	27 March 2012	Opus	NA	NA	Risk workshop was interactive rather than having a formal review output. Minutes issued to all attendees
HAZOP Workshop	Workshop	27 March 2012	Opus	NA	NA	Risk workshop was interactive rather than having a formal review output. Minutes issued to all attendees
Progress reports	Report	Various (January 2011 to September 2012)	Opus	None	NA	The progress reports have reported significant design developments (e.g. change from two spillways to a single spillway) to keep WWAC and the peer review informed of key decisions
Lee Valley Dam - Hydropower Preliminary Design	Report	7 August 2012	D. Inch/ Opus	None	NA	We understand that WWAC has received feedback from D Inch on the report. This has not yet been communicated to T&T

NA - Not applicable

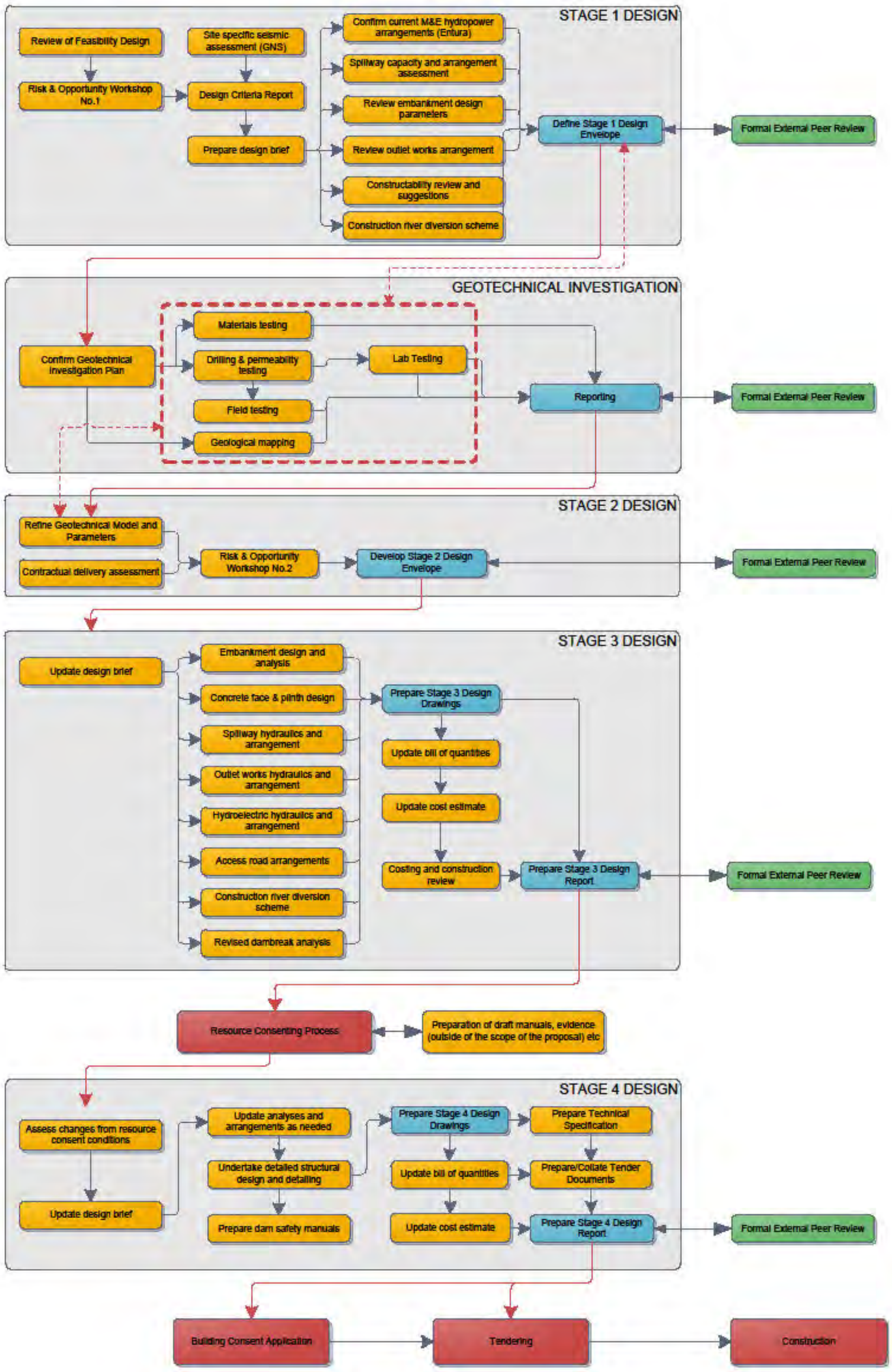


Figure 1.4 Staged design process

1.4 Summary of key dam information

Table 1.3 summaries key information related to the dam design.

Table 1-3 Summary and specifications

Embankment characteristics	
Embankment type	Concrete Face Rockfill Dam (CFRD)
Embankment volume (approximate)	435,000 m^3
Nominal crest elevation (excluding camber)	201.23 mRL
Top of parapet wall (excluding camber)	202.83 mRL
Design Camber	0.3 m
Maximum dam height (from riverbed to dam crest on CL)	53 m
Crest length (approximately)	220 m
Crest width	6 m

Hydrology, reservoir and flood routing characteristics	
Catchment area	77.5 km^2
Normal top water level (NTWL)	197.2 mRL
Reservoir storage at NTWL	13,000,000 m^3
Reservoir area at NTWL	630,000 m^2
Maximum design flood level (MDFL)	202.53 mRL
Reservoir storage at MDFL	16,600,000 m^3
Operational basis flood level (OBFL)	200.48 mRL
Reservoir storage at OBFL	15,200,000 m^3
Reservoir storage at top of parapet wall (202.83 mRL)	16,800,000 m^3

Spillway characteristics	
Primary spillway type	Ogee Weir
Ogee weir effective length (on arc)	41.89 m
Peak outflow – Mean Annual Flow (MAF)	179 m^3/s
Peak outflow – Operational Basis Flow (OBF)	472 m^3/s
Peak outflow – Maximum Design Flood (MDF)	1060 m^3/s
Capacity outflow – Reservoir level at top of parapet wall	1152 m^3/s

Spillway and Energy dissipation characteristics	
Chute length (plan – ogee crest to start of flip bucket)	124 m
Chute width, narrow section	20 m
Chute horizontal transition length	71 m
Chute vertical curve length	21 m
Chute minimum height of concrete lining	2.8 m
Dissipation type	Flip Bucket

Flip bucket radius	20 m
Bucket lip level	156.6 mRL

Outlet characteristics	
Outlet type	Sloping outlet conduits on upstream face with removable screens and valve control.
Number of outlets	2
Outlet level – Upper (elevation of top of bellmouth)	181.5 mRL
Outlet level – Lower (elevation of top of bellmouth)	163.0 mRL
Control type	Twin 800mm Free Discharge Valves
Maximum design discharge capacity (Valve manufacturer velocity limits applied)	15.1 m^3/s
Concrete conduit size under embankment (internal dimensions)	Twin 2.5 m Wide x 4.0 m High

River tailwater characteristics	
Tailwater level MAF	150.85 mRL
Tailwater level OBF	153.46 mRL
Tailwater level MDF/PMF	156.54 mRL

Irrigation and environmental flow release[1]	
Irrigation release at dam toe (at minimum operating level and from either intake)	2.23 m^3/s
Environmental residual flow (7 day Mean Annual Low Flow (MALF) at minimum operating level and from either intake)	0.51 m^3/s
Environmental flushing flow (at minimum operating level and from either intake)	5.0 m^3/s
Note 1: The criterion design capacity of the outlet is the largest of the requirements of 5.0 cumecs and is not additive (i.e. It is not 2.23 + 5 + 0.51)	

2 Design criteria

A design criteria report was prepared for the project in October 2011 (T&T, 2011). Any criteria that were either omitted from that report or were ambiguous, or which have been considered subsequently necessary to be changed are identified in the current report. Key criteria are repeated in Table 2-1 for ease of reference. Of most importance for the dam, we recommend that WWAC in its Dam Safety Assurance Programme (DSAP) classifies the structure as a high Potential Impact Category or Classification dam (PIC). This classification is based on the dam break assessment carried out during the feasibility study (T&T 2009).

Table 2-1 Primary design criteria

Item	Value	Source	Notes
Potential Impact Classification (PIC)	High	Assessment	*Based on dam break assessment*
Operational Basis Earthquake (OBE)	1:150 yr	NZSOLD	*Based on PIC, with ground response based on a site specific seismic assessment*
Maximum Design Earthquake (MDE)	MCE	NZSOLD	*Based on PIC, with ground response based on a site specific seismic assessment*
Seismic loading for non-critical structural elements	1:500 yr	NZSOLD	*Ground response based on a site specific seismic assessment*
Operational Basis Flood (OBF)	1:200 yr	NZSOLD	*Industry custom/precedent*
Maximum Design Flood (MDF)	PMF	NZSOLD	*Based on PIC*
Construction Diversion Flood (CDF)	varies		*Refer discussion in this report*
Minimum freeboard for 100 yr wave or;	0.5 m		*Industry custom*
Tolerable overtopping discharge for same	$1x10^{-6}$ m³/s/m	(R&D report W178, 1999)	*"No damage to buildings"*
Minimum freeboard at OBF+10 yr wave or;	0.5 m		*Industry custom*
Tolerable overtopping discharge for same	$1x10^{-6}$ m³/s/m	(R&D report W178, 1999)	*"No damage to buildings"*
Minimum freeboard at MDF+10 yr wave or;	0.0 m		*Industry custom*
Tolerable overtopping discharge for same	0.002 m³/s/m	(R&D report W178, 1999)	*"No damage to embankment seawalls"*

2.1 Response to peer review comments

The peer review reports raise a number of queries in relation to the design criteria proposed in the T&T Design Criteria Report (2011). These items are addressed in Table 2.2.

Table 2.2 Responses to Opus design criteria peer review

Peer review comment	T&T response
Operational basis earthquake: OBE. I concur that site specific seismic assessment can be applied to establish the appropriate ground response for the chosen annual exceedance probability event(s). However, I am of the view that the NZSOLD OBE criteria can be non-conservative in some instances if it is applied without due consideration of the specific nature of the progressive failure mechanism(s) relevant to the structure and its critical elements. For example, for a CFRD structure of this nature, I would expect that the water retaining element including all joints should be designed to remain fully serviceable in an event with an AEP of say at least 500 years. I therefore consider the project would benefit from some development of the OBE design condition from a serviceability perspective. I anticipate that such development will not significantly change the physical nature of the final design, but it will give greater clarity to the important progressive failure mechanisms and improved confidence in understanding the failure risks.	The concrete face joints have been developed by precedence not by specific design. We are not aware of any designers successfully designing (by numerical analysis) the joints for seismically induced movements. However appropriate detailing of the joints is undertaken to provide some ability to move. The movements that are likely to occur during filling of the reservoir are likely to be greater than those to occur during the 1 in 150 year OBE. Because this detail has been tested in service on other CFRD, we consider it appropriate to use here. Also refer Section 12.5 pertaining to earthquake induced deformations.
Seismic loading for non-critical structural elements. I presume this category is intended to apply to elements that do not have a primary or secondary function related to safe retention of the impoundment and that are not expected to be associated with response to an impoundment related incident. That is, by definition not "appurtenant works", and therefore requiring compliance within the "normal" building control provisions of the Building Act rather than the specific dam safety clauses. I concur that site specific seismic assessment can be applied, but actual design loading should be checked against the methodology and annual exceedance probability criteria of NZS1170.5, specifically the importance level rating and the design service life for such elements.	The access bridges are the only significant structure in the project that we consider fit into this category. The bridges have been designed for a 1 in 500 year return period earthquake. This return period earthquake is appropriate for the bridges which are 'normal' structures with a design life of 50 years.
Seismic loading for appurtenant works. As appurtenant works by definition are elements that have a primary or secondary function	We are not aware of any non-critical structures except for the access bridges in this project.

related to safe retention of the impoundment and/or that are expected to be associated with safe response to an impoundment related incident, the applicable design standards may not be fully captured by the listing presented in section 4.3. There is potential for other criteria to apply to any such "critical" elements, subject to specific assessment of the specific potential failure mechanisms identified, and their influence on risk exposure.	
Safety of O&M personnel and the public: Adopted design criteria do not satisfy personnel safety compliance obligations related to sections of the NZ Building Code other than B1 – Structure, and B2 – Durability, or any other relevant standards pertaining to safety of personnel and the public.	Whilst no specific criteria were set, the design is carried out in accordance with the New Zealand Building code and the NZSOLD Dam Safety guidelines. Ensuring public safety is an underlying principle of these documents. We have also held a HAZOP workshop to consider the H&S aspects of the design. Minutes of the workshop are appended to this report. The workshop was attended by the peer reviewer, as well as the designers and the client. The design of the dam has been influenced by outcomes of the HAZOP workshop.
Design criteria for establishing compliance with relevant sections of the NZ Building Code covering such aspects as safety from falling (F4), are not specifically addressed in the design criteria report. Furthermore, the aspect of safe access into confined spaces for operational and maintenance activities is also not covered. These considerations may influence final design layouts and/or detailing of the works for such aspects as access ways, barrier systems, drainage, and ventilation of potential work spaces, etc., and may be relevant to the issue of the full building consent. I suggest that that a section covering these considerations be added to the report to give clarity to the intended compliance process being followed during detailed design.	T&T has taken a pragmatic approach to this clause (F4) of the NZ Building Act. Areas where a fall height of greater than 1m is created due to new building structures (i.e.: The dam embankments, any concrete works, the spillway, bridges, outlet works, etc.) *and* where operational personnel are likely to access, have been provided with industrial type handrails or fences. The design assumes (based on communications with WWAC) that the dam will not be open to the public and therefore fall protection for children requiring more robust hand rails are not required. Fill or cut slopes in natural ground do not meet the definition of a building and therefore no fall protection is provided. If WWAC desires to fence or protect these areas as well, then this can be added at a later date (albeit at additional cost to that estimated herein). The requirement for ventilation of the conduits will be addressed in Stage 4. The further design development of the powerhouse in respect of the peer reviewer's comments is outside the scope of this design given that the current design is to feasibility level only.

3 Storage elevation curve

Figure 3.1 shows the storage elevation curve developed for the Lee Valley Dam. This has been developed using contours derived from LiDAR supplied by WWAC. The storage elevation curve accounts for the volume of the dam itself. The curve does not account for any changes in the reservoir due to:

- The use of alluvial gravels (or any other material in the reservoir) for borrow materials; or
- Changes in storage over time as a result of sedimentation.

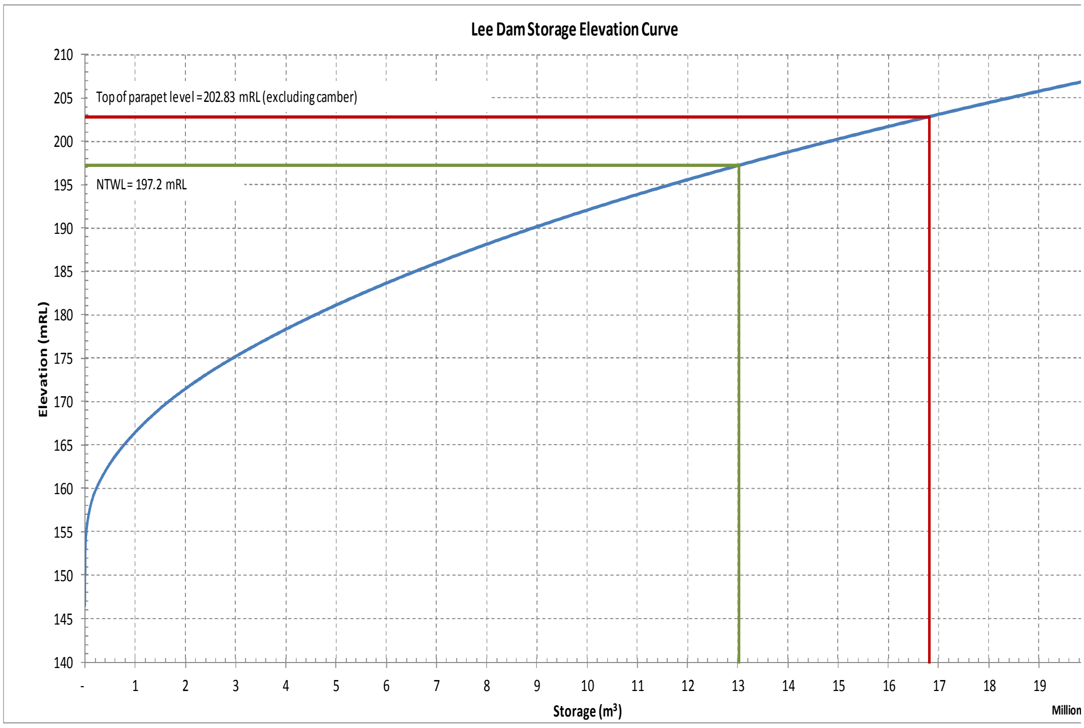


Figure 3-1 Storage elevation curve showing NTWL at 197.2 mRL

4 Geological interpretation

Refer to Appendix F, the Lee Valley Dam Detailed Design Geotechnical Investigation Report (T&T 2012), bound separately.

The key interpretive data derived from the geological and geotechnical site investigations have been considered and commented on in this report. These are covered in each section under the relevant physical works components.

5 Engineering flood hydrology

This section consolidates works described in the Stage 1 Design Report (T&T 2011) and the Engineering Feasibility Report (T&T 2009) along with responses to Opus' peer review comments.

5.1 Climate change adjustment

Design flood hydrographs were presented in the Engineering Feasibility Report (T&T 2009) along with a discussion of how they were developed and their justification. Reference to that report should be made for such details. The Engineering Feasibility Report (T&T 2009) was peer reviewed by MWH. That review provided a suggestion that climate change should be considered for flood hydrology during the detailed design stage. This section sets out our consideration of potential climate change effects on flood hydrology.

The design floods for construction diversion do not need to be adjusted for climate change as river diversion works can be expected to be undertaken in the short-term. If there is a significant delay between design and scheme construction, there may need to be a subsequent review of flood hydrology for construction diversion.

In this section, climate change adjustments have been considered for design floods with a finite return period (excluding the PMF) that will be used for the design of permanent works. Changes to the design rainfall depths were assessed by following the approach set out in the New Zealand Ministry for the Environment (MfE) publication "Tools for Estimating the Effects of Climate Change on Flood Flow: A Guidance Manual for Local Government in New Zealand, May 2010". Corresponding increases in the design flood hydrographs were then computed by considering the increases in runoff depth resulting from the climate-adjusted design storms.

The following parameters were selected for climate change adjustment, consistent with the approach outlined in the 2010 MfE publication:

- Projected temperature change to the year 2090
- The mid-range emission scenario A1B (from the six IPCC illustrative marker scenarios)
- The average of 12 models for the selected IPCC emission scenario (A1B), and
- A uniform 8% increase in rainfall depth per 1°C increase in temperature.

From the 40 emissions scenarios that have been developed (Nakicenovic and Swart, 2000), the Intergovernmental Panel on Climate Change (IPCC) selected 6 illustrative "marker" scenarios, identified as B1, B2, A1T, A1B, A2 and A1FI, in order of increasing influence on global temperature increase over the 21st century (IPCC, 2007). All were considered equally valid with no attempt to assign probabilities of occurrence. These emissions scenarios span a reasonable range of plausible futures and depend on changes in population, economic growth, technology, energy availability and national and international policies.

In the absence of evidence of relative likelihood of these scenarios, the 2010 MfE publication takes account of all six illustrative marker scenarios while focusing on a "middle-of-the-road" scenario namely the A1B scenario. Indeed, an earlier more generalised MfE publication (2008) concentrates almost exclusively on the A1B scenario, providing predictions for this scenario only. For the same reason, the current assessment also focuses on the A1B scenario. Further, the adopted temperature change is the average of the predictions from 12 general circulation models (GCM), which is regarded as the "best

estimate" (MfE, 2008). Clearly we cannot accurately predict future climate changes at present and this needs to be recognised in setting likely increases in flood flow. Any actual changes in climate in the future could vary from those predicted using the A1B scenario.

The projected change in the annual mean temperature, 1990 to 2090 for the A1B scenario in the Tasman-Nelson regional council area is 2.0 degrees Celsius. It is interesting to note that the GCM predictions across 12 models vary widely from 0.9 °C to 3.5 °C but, excluding outliers, there is a relatively tight cluster between 1.5°C and 2.2 °C. The predicted 12-model average temperature increase ranges between 1.3 °C for the low emission B1 marker scenario and 2.9 °C for the A1FI high emission scenario, compared with the 2.0 °C increase predicted for the adopted "middle-of-the-road" A1B scenario.

The adopted 2.0 °C temperature change translates to a predicted 16% increase in rainfall depth. The adjusted runoff depth was calculated by applying the increased rainfall depth in the calibrated rainfall-runoff relationship.

As a comparison, the design flood hydrographs, with and without climate change adjustment, are shown in Figures 5.1 and 5.2. Table 5.1 summarises the peak inflows from both cases. Peak inflows increase by between 20% for the 10,000 year ARI event and 25% for the mean annual flood with climate adjustments. The climate change adjusted inflows have been used in the design of the spillway of the dam.

Table 5-1 Peak inflow at the proposed dam site, with and without climate change adjustment

Flood Return Period (ARI see note)	Peak Inflow (m^3/s)	
	No Climate Adjustment	With Climate Adjustment
2.33 years (mean annual flood)	168	210
5 years	216	267
10 years	255	314
20 years	292	359
50 years	339	415
100 years	375	457
200 years	412	501
1000 years	496	600
10,000 years	616	741
PMF	1094	No change

Note: ARI = average recurrence interval, usually expressed in "years", is equal to the event return period. AEP = annual exceedance probability, usually expressed as a percentage, equal to reciprocal of the ARI or return period.

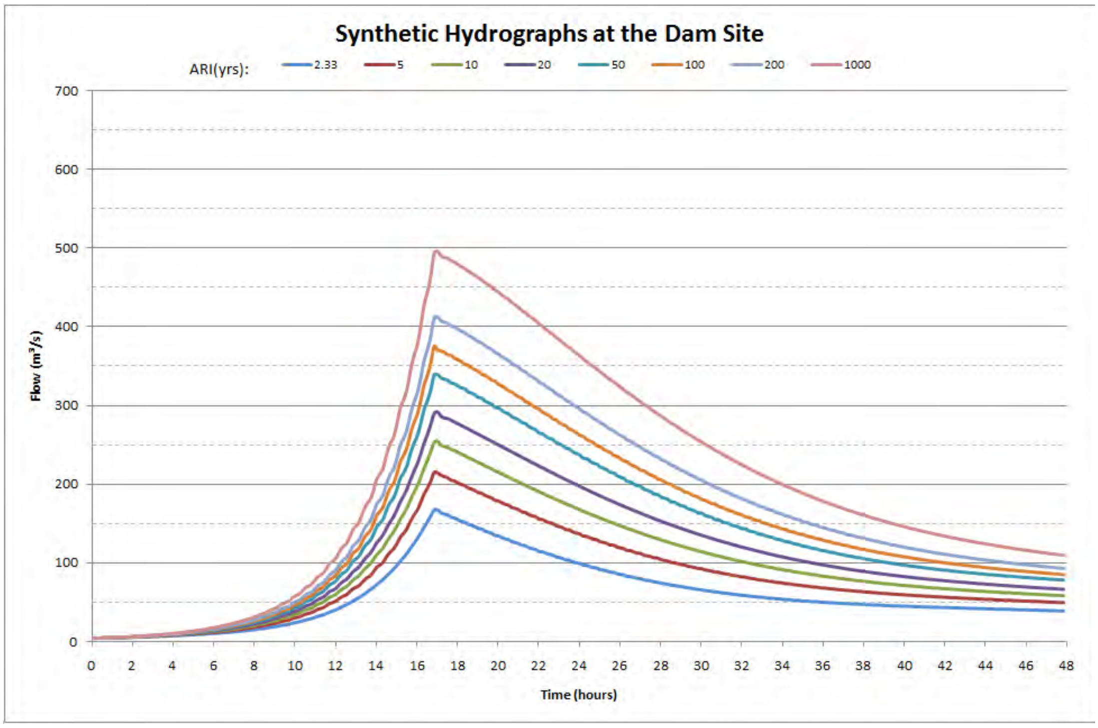


Figure 5.1 Synthetic hydrographs without climate change adjustment

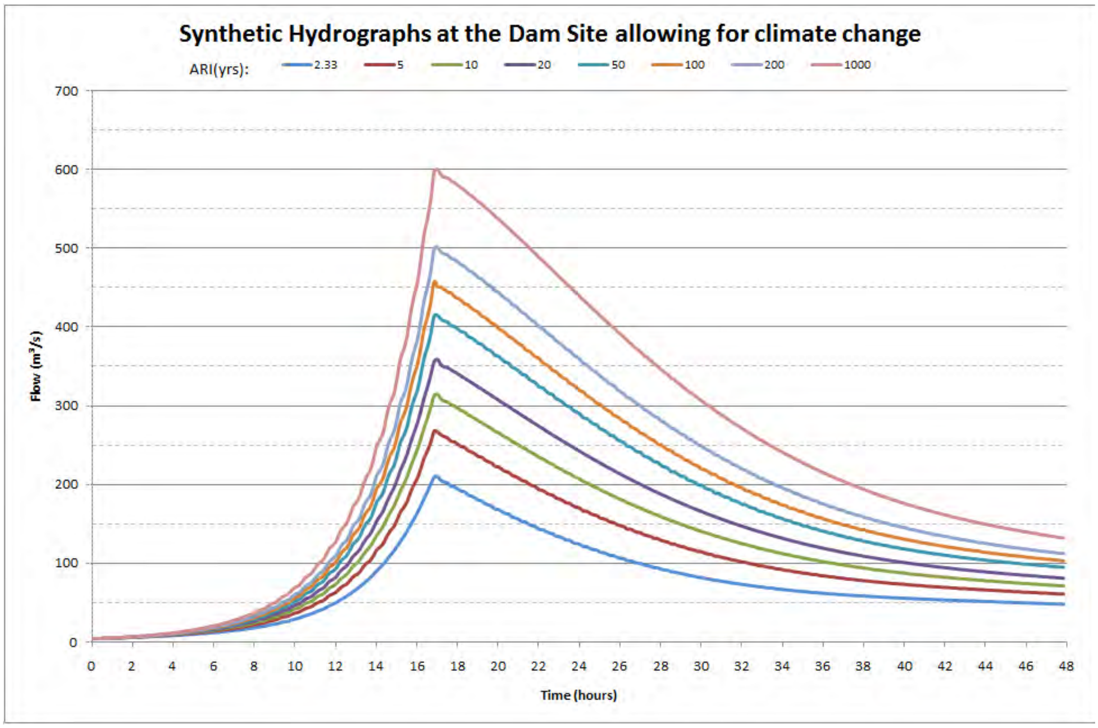


Figure 5.2 Synthetic hydrographs adjusted for climate change

5.2 Validation of calibrated model

A standard catchment rainfall-runoff model was constructed for the Lee Valley Dam site during the feasibility study stage (T&T 2009). The catchment model was calibrated using a number of recorded storm rainfall and flood hydrograph events for the Lee River and wider Wairoa River catchments.

HEC-HMS (Hydrologic Modelling System developed by the US Army Corp of Engineers) was used to model the catchment response to storm rainfall and to subsequently generate the Probable Maximum Flood hydrograph. In the feasibility study, three storm events were used to calibrate the model: 23 May 2007, 22 January 2008 and 24 November 2008. The calibration results gave a reasonably good fit between the predicted flows and the actual flows recorded at the Lee above Waterfall Creek recorder – see for example Figures 5.3 and 5.4.

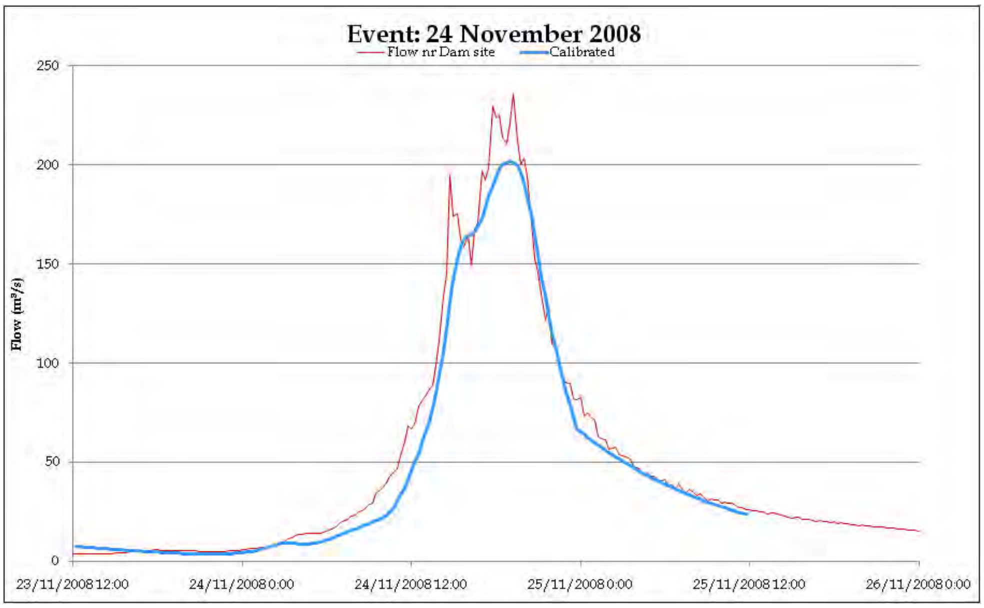


Figure 5.3 Calibration results for rainfall event on 24 November 2008

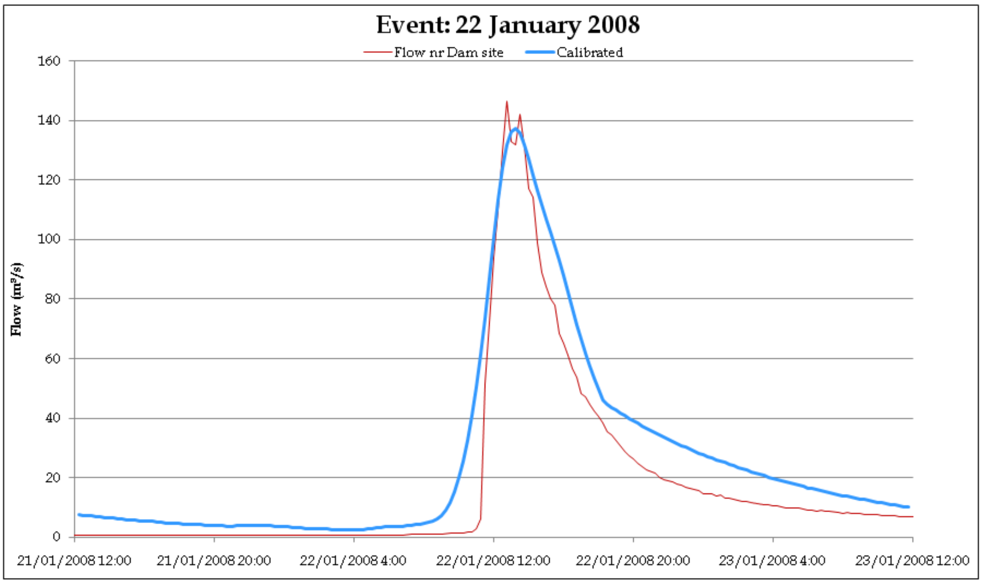
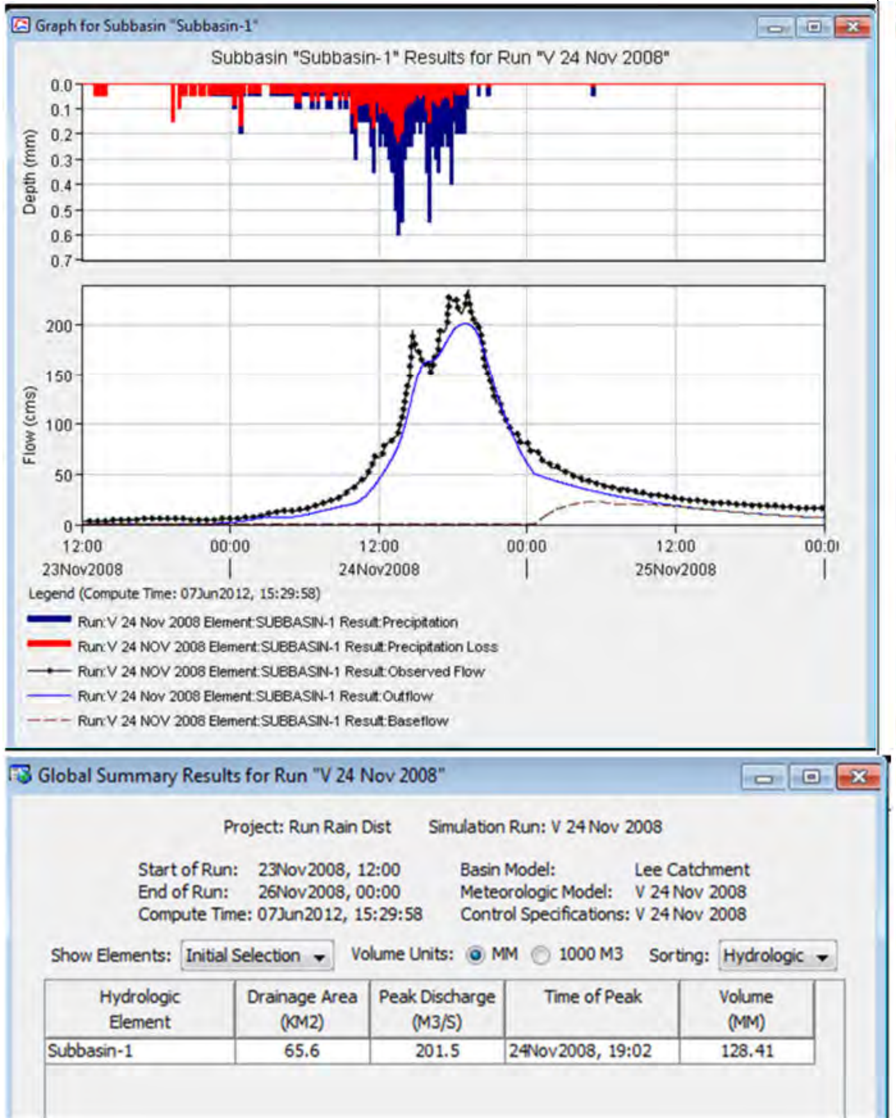
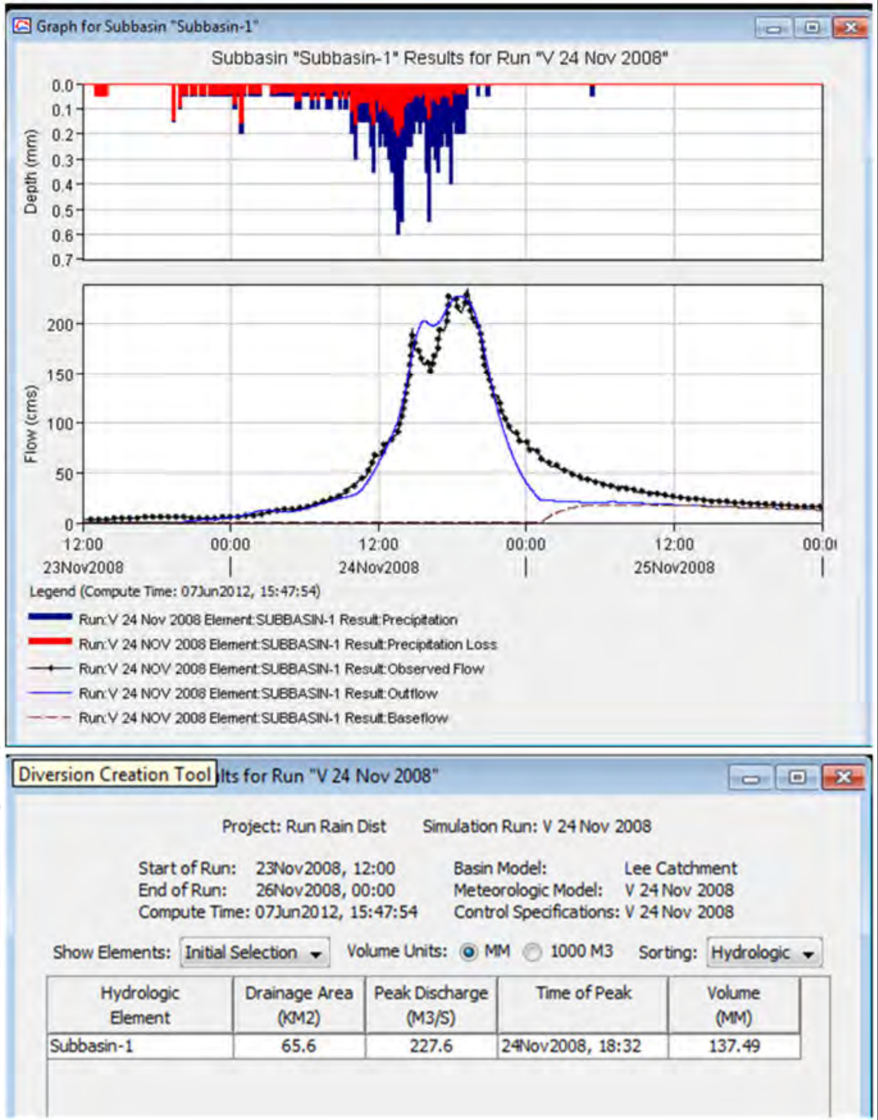
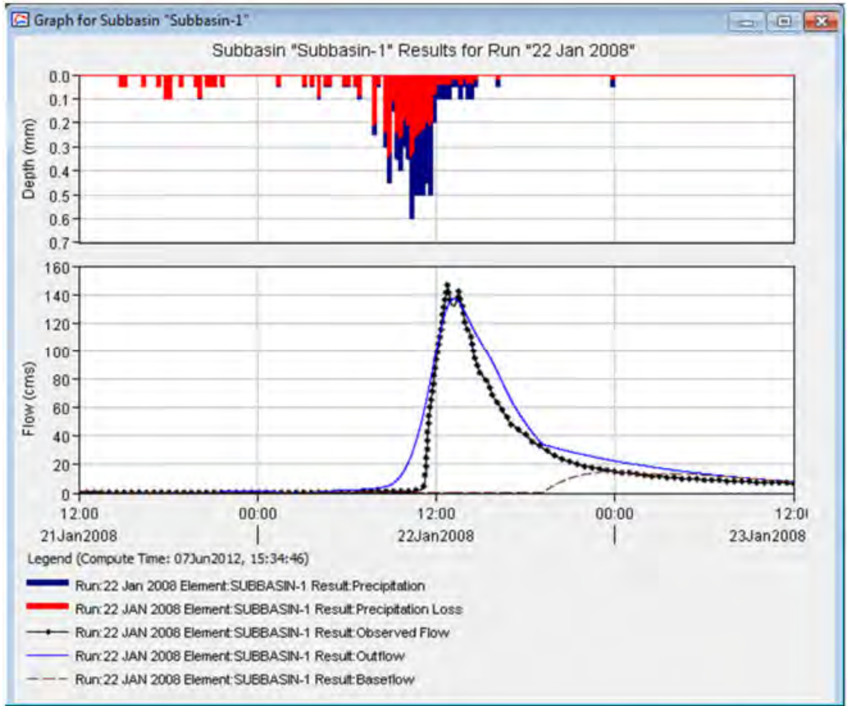


Figure 5.4 Calibration results for rainfall event on 22 January 2008

A significant flood event occurred, on 19 January 2011, after the completion of the feasibility study in 2009. This flood event peaked at 208 m^3/s, which is only about 12% lower than the largest calibration event used previously, viz. the 24 November 2008 flood which peaked at 236 m^3/s and represented an approximately 14 year ARI event.

This most recent flood event was therefore selected as an appropriate independent validation event. The event rainfall was run through the original HEC-HMS model and the original model parameters were adjusted to improve simulation of the hydrograph peaks for the both the events assessed in the original calibration process, and the recent validation event. Calibration and validation results for the improved parameter set are shown by way of screen shots from HEC-HMS in the figures below.

	Original parameters	Revised parameters
20		



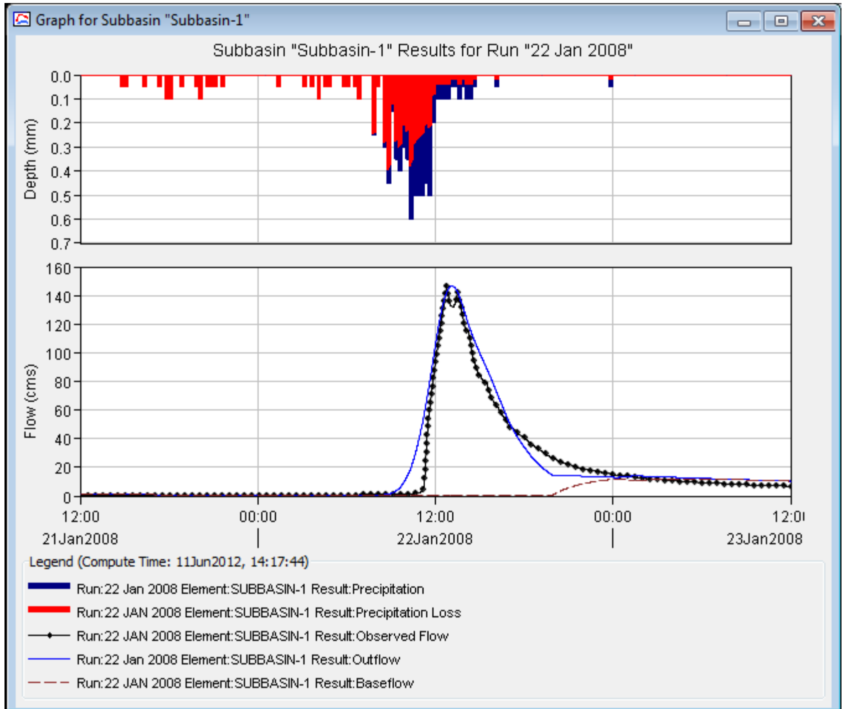
Global Summary Results for Run "22 Jan 2008"

Project: Run Rain Dist Simulation Run: 22 Jan 2008

Start of Run: 21Jan2008, 12:00 Basin Model: Lee Catchment
End of Run: 23Jan2008, 12:00 Meteorologic Model: 22 Jan 2008
Compute Time: 07Jun2012, 15:34:46 Control Specifications: V 22 Jan 2008

Show Elements: Initial Selection Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Subbasin-1	65.6	137.0	22Jan2008, 13:13	60.25



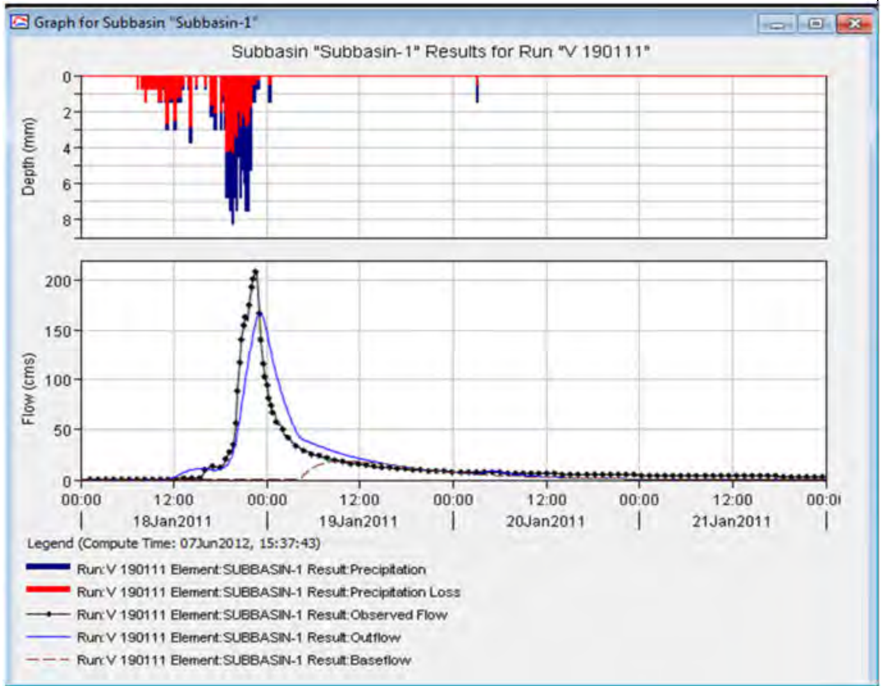
Global Summary Results for Run "22 Jan 2008"

Project: Run Rain Dist Simulation Run: 22 Jan 2008

Start of Run: 21Jan2008, 12:00 Basin Model: Lee Catchment
End of Run: 23Jan2008, 12:00 Meteorologic Model: 22 Jan 2008
Compute Time: 11Jun2012, 14:17:44 Control Specifications: V 22 Jan 2008

Show Elements: Initial Selection Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Subbasin-1	65.6	146.5	22Jan2008, 13:07	51.89



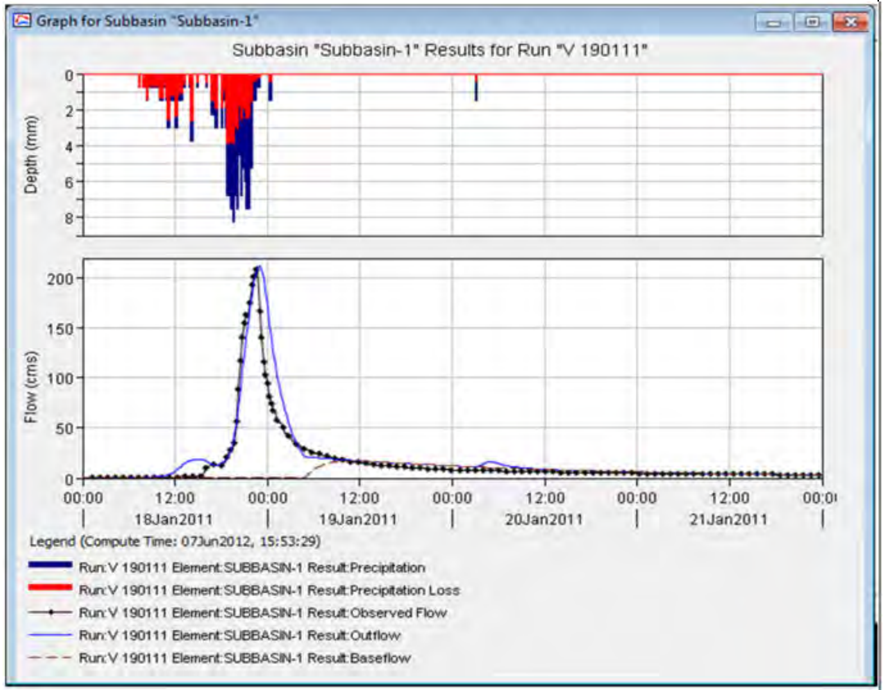
Global Summary Results for Run "V 190111"

Project: Run Rain Dist Simulation Run: V 190111

Start of Run: 18Jan2011, 00:00 Basin Model: Lee Catchment
End of Run: 22Jan2011, 00:00 Meteorologic Model: V 19/01/2011
Compute Time: 07Jun2012, 15:37:43 Control Specifications: V 190111

Show Elements: Initial Selection Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Subbasin-1	65.6	167.3	18Jan2011, 23:15	79.83



Global Summary Results for Run "V 190111"

Project: Run Rain Dist Simulation Run: V 190111

Start of Run: 18Jan2011, 00:00 Basin Model: Lee Catchment
End of Run: 22Jan2011, 00:00 Meteorologic Model: V 19/01/2011
Compute Time: 07Jun2012, 15:53:29 Control Specifications: V 190111

Show Elements: Initial Selection Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Subbasin-1	65.6	212.6	18Jan2011, 23:00	99.35

In summary, the previously calibrated HEC-HMS model has been improved with consideration of the January 2011 flood event. The modelled peak flow for each event assessed in the calibration process is in close agreement with recorded peak flows, and the recorded hydrographs have very similar flood volumes to those produced by the model.

5.3 Probable Maximum Flood (PMF) hydrographs

PMF hydrographs used to assess adequacy of the Lee Valley Dam spillway and safety of the structure were generated from estimated Probable Maximum Precipitation (PMP) depths for a range of storm durations using the HEC-HMS model with revised model parameters and initial loss set to zero to simulate a saturated catchment. The PMP hyetographs used in the original analyses were reviewed and adjusted for size of catchment and effective height of barrier impeding the flow of moisture into the catchment, factors which were not considered in the feasibility study (T&T 2009). The resulting hydrograph peaks and volumes are very similar to the PMF hydrographs from the original analysis.

5.4 Response to peer review comments

Table 5.2 includes responses to Opus peer review comments.

Table 5.2 Responses to Opus hydrology peer review

Peer review comment	T&T response
On p2 a definition of ARI is provided. In fact this definition is incorrect. ARI is generally used as an abbreviation of Average Recurrence Interval and not Annual Recurrence Interval.	Noted. This is corrected in this report.
The assessment of the potential effects of climate change follows a very standard approach. Of particular concern, however, is that most of the 'methods' suggested relate almost solely to the potential effect of climate change on storm rainfall. No consideration is given to all the other consequential effects which may occur within the hydrological cycle, and in particular to the rainfall-runoff relationship.	See next response.
Given the uncertainty and wide range of potential values for the predicted rise in average temperatures we would question the use of 'mid range values'. This is particularly the case for a major infrastructure development such as the Lee Valley dam, and where the potential consequences are significant should under-design result in failure. We would have thought that conservative design might require the use of 'high end temperature increases'; or at least the provision of a sensitivity analysis to show the potential range and variability of such effects. The latest temperature predictions and trends	There is great uncertainty regarding projected climate change and we consider it appropriate to adopt mid-range temperature change when determining design floods. The MfE guide referred to during the design in relation to climate change also recommends the use of mid-range scenario values. We agree that the potential consequences of failure of Lee Valley Dam are significant and we have based the design of the dam on passing the Probable Maximum Flood (PMF) without overtopping and with a 300mm freeboard allowance. The PMF is based on Probable

are that temperatures are actually rising faster than the mean predictions. Each revision of the predictions has resulted in an increase in expected temperatures. It would be useful therefore to see the difference of using perhaps 2.9°C (the mean of the A1FI high emission scenario) as opposed to 2°C used in the analysis presented. The use of a higher predicted temperature rise would also recognise the high level of uncertainty inherent in global warming predictions and their possible effect on storm rainfalls.	Maximum Precipitation (PMP), which is not adjusted for climate change. The effect of increased temperatures on evaporation and the rainfall-runoff relationship will affect yield from the reservoir but the impact on design floods will be secondary and can be ignored. This assignment does not address yield from the reservoir.
Given the significant changes in temperature used in the modelling of storm rainfall, we are concerned that these differences were then used in a rainfall-runoff model calibrated to the current climatic conditions and environment. While it might be impossible to model how the Lee Valley hydrological and rainfall-runoff system will operate in 80 years with a rise in temperature, it is overly simplistic to argue that the only change will be in storm rainfall.	See above response
If such a significant change is expected in rainfall, one would also expect changes in evapotranspiration, soil storage, vegetation cover, runoff coefficient, and a range of other factors. In addition, the formation of the dam is likely to significantly change the rainfall-runoff relationship by resulting in 100% runoff over the dam. This effect may be significant depending on the surface area of the reservoir and the runoff rates used for the rest of the catchment under the existing scenario.	The changes Opus refer to would only affect the OBF. Because the spillway is designed to accommodate the PMF, the spillway capacity design is not affected. In the event that the OBF were to be greater over time as a result of increased climate change (i.e. an upper bound climate scenario), and the freeboard is reduced below acceptable levels; then the dam Owner could consider remedial measures (such as raising the parapet wall). WWAC has directed T&T to optimise the design as much as practical at this stage. Therefore we consider designing to the upper bound is an unnecessarily conservative approach.
Some discussion is therefore required as to the uncertainty of the future rainfall runoff relationship, and how this uncertainty has been incorporated into the design.	See above response.
The issue of uncertainty is critical given the residual error which still remains in the HEC-RAS rainfall-runoff model even after calibration. Although it is argued that the calibration is good, it would appear that the errors are still up to 20% with respect to the peak discharge.	During detailed design the model has been recalibrated using an improved parameter set, improving its ability to accurately simulate both flood peaks and associated event volumes. The calibration has been validated against a recent flood that was recorded after the feasibility study was completed.

	The improved calibration/validation is presented in Section 5.2.
Further to the above comments on the hydrology [refer Appendix] the flood estimates for the dam site have been produced using a rainfall / runoff model calibrated against recorded flood discharges for three relatively modest flood events – 22 January 2008, 24 November 2008 and 19 January 2011. The measured hydrographs for these floods are from the Lee River above Waterfall Creek gauging station. It is unclear how accurate the "measured" peak flood discharges for these calibration events are. Nor is it clear what the accuracy of the flood estimates for much lower flood frequencies obtained from the rainfall / runoff routing model is. The following specific questions relate to these issues.	Flow gauging was undertaken at the Waterfall Creek gauging station during the November 2008 event, and a flow estimate of 244 m^3/s was recorded close to the peak of the hydrograph. This is the highest flow gauging on record at this site. The rating curve for the Waterfall Creek gauge compares well with flow gaugings performed over the full range of measurements and can be used with confidence.
How good is the stage / discharge rating for this gauging station? What is the highest flow that has been gauged with a current meter at this site?	See above response.
How good are the peak flood discharges estimated from the stage / discharge rating for the rainfall / runoff model calibration events? Have they been extrapolated above existing current meter gauging measurements? If so, by how much? What is the possible error in the stage / discharge rating curve when extrapolated to higher stages (river levels)? *What is the possible error in the predicted discharge estimates for lower frequency floods in Table 3-1?*	See above responses in relation to Waterfall Creek flow gauge and model recalibration/validation.

6 Wave environment

A determination of the reservoir wave environment is necessary to assess the effect on the dam and its associated structures, with regard to loads, freeboard, overtopping and the potential for erosion. This is especially important when the water level in the reservoir is elevated above normal levels during flood passage.

Waves can be generated by wind action across the reservoir, landslides into the reservoir, or seismic action and reservoir response. The following sections provide estimates of the windspeeds, wind generated wave run-up heights and landslide generated waves.

6.1 Design wind speeds

Estimates of extreme wind speeds were obtained from the New Zealand Structural Design Actions NZS1170 (New Zealand Standards, 2002) and converted to mean 1 hour wind speeds via empirical methods (USACE, 2011).

The maximum straight line fetch to the dam is 1,100m from the south; the effective fetch is limited by the surrounding land and the irregular shoreline. An effective fetch of 700 m was calculated using the method developed by Saville, McClendon, & Cochran (1962). The most significant fetch is from the south and the mean 1 hour wind speeds for the 1 in 10 and 1 in 100 year return periods in are presented in Table 6.1. These return periods have been selected to match the design loading combinations, as presented in Table 2.1.

Table 6.1 - Mean one hour wind speeds

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

6.2 Wind generated waves

The wave climate was assessed using theory developed by 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 6.1.

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. The Lee Valley Dam has been designed with a concrete facing (with a slope of 1V:1.5H), thus it will absorb less wave energy and result in more reflection and run-up than a rock armoured face.

Run-up is defined in this report 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 (1992) and incorporated in the method used by the USACE 2011. 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 (R_s, $R_{2\%}$ and $R_{0.1\%}$) are presented in Table 6.2. Figure 6.1 provides an illustration of the wave climate.

Table 6.2 - Design wave climate at dam face

Return period (years)	H_s (m)	T_p (s)	R_s (m)	$R_{2\%}$ (m)	$R_{0.1\%}$ (m)
10	0.283	2.069	0.382	0.557	0.723
100	0.343	2.259	0.463	0.675	0.873

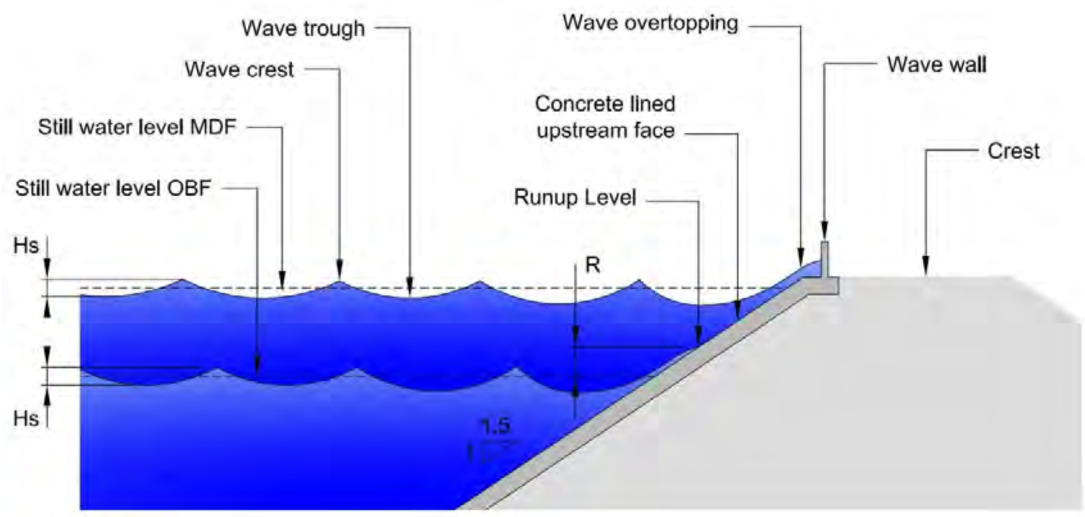


Figure 6.1 – Wave climate

The Lee Valley Dam wave run-up is not particularly high due to the relatively short fetch.

It is common practice in CFRDs to include a parapet wall at the crest providing an economic means of increasing freeboard where the alternative is to provide additional embankment height. The Lee Valley Dam also includes a parapet wall and its effect on wave run-up and preventing overtopping is described below.

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 configured to meet allowable overtopping discharge rates.

The angle of wave attack influences the overtopping rates. The angle of attack adopted for design of the Lee Valley Dam is 54 degrees from the normal to the dam crest. Reduction factors have been incorporated into the methods employed to evaluate overtopping discharge rates.

Overtopping discharge (per metre of wall) will be limited to the flowing allowable rates, as per the design criteria report (T&T, 2011):

- $1x10^{-6}$ $m^3/s/m$ for reservoir at NTWL and a 100-year wind generated wave climate
- $1x10^{-6}$ $m^3/s/m$ for maximum reservoir level whilst routing the OBF and a 10-year wind generated wave climate
- 0.002 $m^3/s/m$ for maximum reservoir level whilst routing the MDF and a 10-year wind generated wave climate.

The overtopping discharge was estimated using methods outlined in the "USACE Coastal Engineering Manual" (2002) and "Wave Overtopping of Sea Defences and Related Structures: Assessment Manual" (EurOtop, 2007).

At the NTWL and with 100-year wind generated waves the freeboard is 5.0 m and no overtopping discharge is anticipated. At the OBF water level and 10-year wind generated wave the freeboard is 1.8 m and no overtopping discharge is anticipated. At the MDF water level and 10-year wind generated wave the freeboard is 0.3 m and the overtopping discharge estimates from the above references range from 5.24×10^{-4} to 1.33×10^{-3} $m^3/s/m$. These rates are less than (i.e. within) the design criterion. Figure 6.2 shows the change in predicted overtopping discharge estimates with increasing freeboard when the waves are impacting on the vertical face of the parapet wall.

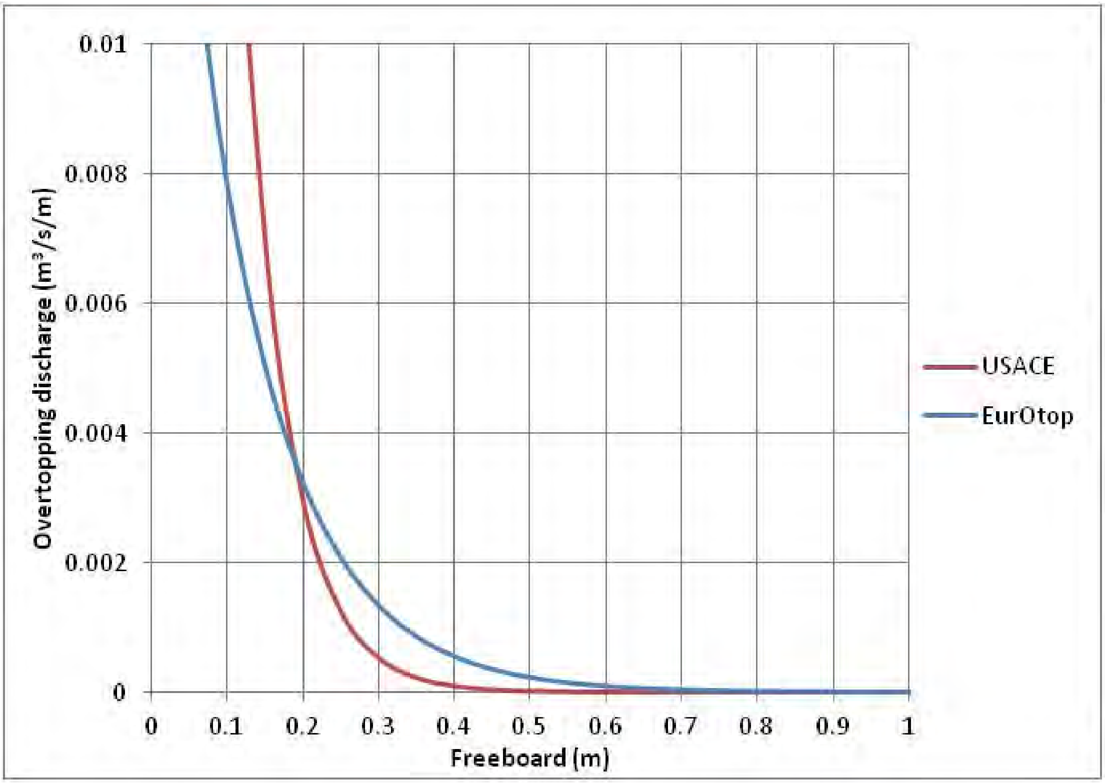


Figure 6.2 Overtopping discharge vs freeboard height

6.3 Reservoir seiching

A seiche is a standing wave in an enclosed or partly enclosed body of water such as a lake or reservoir. Earthquakes may induce seiches, as can climatic conditions on large lakes or reservoirs (such as the Great Lakes in the United States). Seismicity induced seiching is discussed below. However, the Lee Valley Dam reservoir is not considered large enough to warrant investigating climate induced seiching.

Seiches arising from earthquakes have been noted at many lakes and reservoirs, over a number of centuries, and recently include the Chilean and Baja California earthquakes of 2010, as well as the 2011 Tohoku earthquake. Hebgen Dam (a concrete core, earth embankment dam) in Montana was reportedly overtopped four times by seiche waves generated in the 1959 magnitude 7.3 earthquake. This event caused the lake bed to be abruptly down dropped and warped causing lake oscillations lasting for some 12 hours. Despite the overtopping of the dam, it did not fail. Seiching may be significant in small

water bodies such as ponds and swimming pools as the frequency of the seismic excitation is more often closer to the resonant frequencies of small bodies than lakes.

The magnitude of the standing wave generated by an earthquake is dependent on two primary factors:

- The magnitude of energy that a potential earthquake can impart to the water body; dependent on the magnitude of the earthquake, the distance of the lake from the source and the ability of the ground to transmit the energy to the water
- The natural frequency of oscillation of the lake, which is dependent on the geometry of the lake. The larger the lake the greater the difference between the natural frequency of the lake and the frequency of the earthquake. There is a reduction in the magnitude of the wave generated as the two frequencies diverge, all other factors being equal.

Sherrard et al (1963) note that seiches are solitary waves and unlikely to cause the catastrophic failure of an embankment. Consequently, in many instances the effects of seiches are either ignored or estimated based on reports of similar circumstances. This approach notwithstanding, there have been substantial studies undertaken for large lakes, such as Lakes Ohau, Coleridge and Te Anau in the South Island (Carter & Lane, 1996), and Lake Tahoe in the United States (Ichinose et al, 2000).

Three empirical quantitative methods have been identified to estimate the possible magnitude of the initial one-dimensional solitary waves:

- Murty (1979)
- Bohannon and Gardner (2002)
- Synolakis and Uslu (2003).

Each of these presume that energy is transmitted from the ground to the water by a notional sliding mass down slope (landslide) with little or no physical movement of the rest of the slope in general. In line with this approach, the assessment of seiching at the Lee Valley Dam is based on landslide generated wave modelling, discussed in Section 6.4.

Tilting of the reservoir body and/or the ground beneath as a result of earthquakes is a further mechanism that can generate seiches in lakes, as in the case of Hebgen Dam. The Lee Valley Dam reservoir has the active Waimea –Flaxmore transcurrent (strike-slip) fault 8.5 km to the north west and the Wairau segment of the transcurrent Alpine fault 20 km to the south east. Both faults are expected to have horizontal to vertical movements approximately in the ratio of 1V to 10H. We would expect a vertical movement of no more than 1 metre per event. Given the distance of the fault trace from the site (closest is 8.5 km) any potential resulting tilt is likely to be negligible. Therefore no attempt has been made to assess the effects of such small scale tilting on the reservoir; particularly given the normal freeboard is relatively high at 5.6 m.

6.4 Landslide generated waves

Geological investigations (T&T 2012) for the dam identified a number of potential slope instability or landslide features around the potential reservoir. These are shown on the Reservoir Landslide Map presented in the Design Drawings. Waves generated by a landslide into or within the reservoir may have the potential to overtop the dam crest and cause damage.

To understand and manage the risk associated with reservoir landslide events, the potential landslides identified were prioritised with guidance from methods described in ICOLD

Bulletin 124 (2000). The two landslides considered to pose the most significant risk to the dam were then selected for hydrodynamic modelling to more accurately predict their impact on the dam.

Modelling assumptions, results and conclusions are presented below. For detailed assessment of the landslides refer to Appendix F.

6.4.1 Modelling and assumptions

A MIKE21 2D hydrodynamic model was used to investigate the effects of the two landslides on the reservoir, specifically wave heights and periods in the vicinity of the dam. MIKE21 is a two dimensional modelling software package developed by DHI. A brief summary of the modelling assumptions and results are below:

- The movement of each of the landslides into the reservoir was modelled using a time varying bathymetry (vertical displacement of the bed)
- No water was entering or exiting the reservoir during the wave's propagation around the reservoir, to simplify the modelling process
- A landslide velocity of 19 m/s was chosen based on information presented in “Review of natural terrain landslide debris-resisting barrier design - Geo Report No. 104”.

Two landslides were selected for detailed hydrodynamic modelling as follows:

- Scenario 1, landslide (labelled as landslide 6 and 7 on drawing 27425-GEO-09) at approximately ch 1400 m upstream of the dam, being the worst case likely landslide to occur under OBFL conditions (triggered by extreme rainfall) with an approximate volume of 84,000 m^3
- Scenario 2, landslide (labelled as landslide 3 on drawing 27425-GEO-09) at approximately ch 600-800 m upstream of the dam, being the worst likely landslide to occur under OBE and NTWL conditions (triggered by seismic event) with an approximate volume of 80,000 m^3.

6.4.2 Hydrodynamic modelling results

Figure 6.3 and Figure 6.4 show the modelled waves propagating through the reservoir to the dam face for Scenario 1 and 2 respectively. Concurrent modelling of Scenario 1 and 2 is not considered necessary given that Scenario 1 is related to extreme rainfall and Scenario 2 is related to seismic events.

Scenario 1:

A maximum water level of 201.91 mRL was calculated at the right abutment of the dam. This equates to a wave height of 1.43 m above the OBFL, and 0.92 m below the top of the parapet wall. The wave period is approximately 56 seconds.

Scenario 2:

A maximum water level of 201.91 mRL was calculated at the right abutment of the dam. This equates to a wave height of 4.71 m above the NTWL, and 0.92 m below the top of the parapet wall. The wave period is approximately 8 seconds. Note that although the maximum water levels in the two scenarios are the same, this is a coincidence.

The resultant wave heights were also calculated using empirical methods described by Pugh and Hubert (ICOLD, 2000) as a check on the model results.

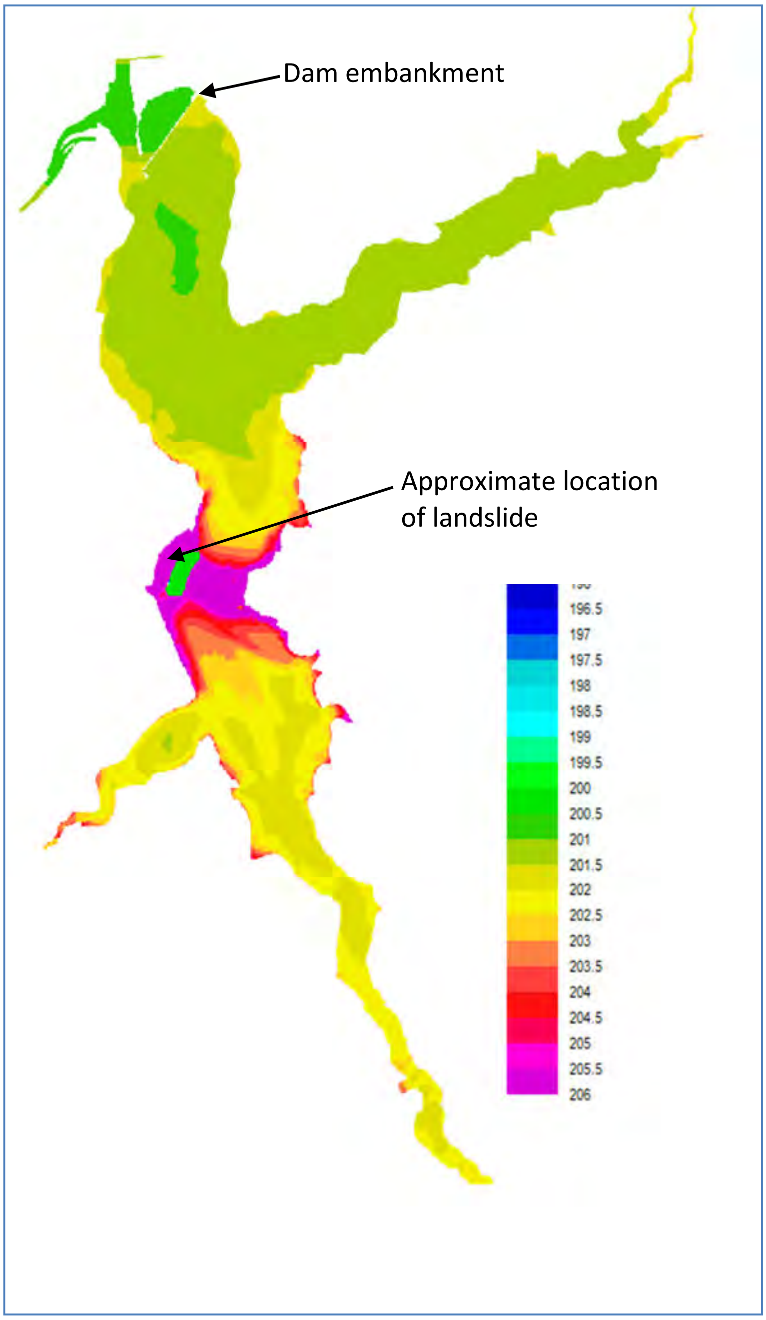


Figure 6.3 Modelled landslide induced wave propagating through the reservoir for Scenario 1

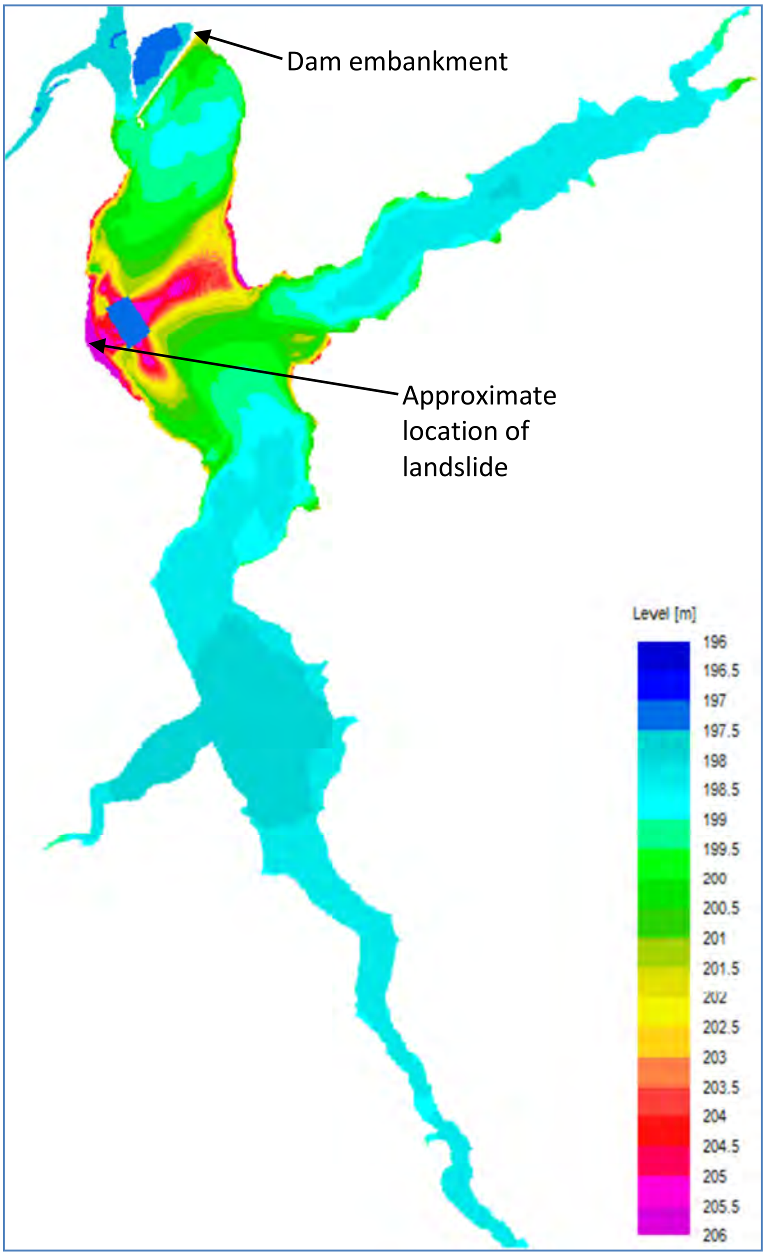


Figure 6.4 Modelled landslide induced wave propagating through the reservoir for Scenario 2

6.4.3 Discussion and conclusion

Scenario 1:

Under the action of the landslide the wave height at the dam is calculated to be 1.43 m. This height can be safely contained below the top of the parapet wall with approximately 0.91 m freeboard. It is also expected to pass safely under the spillway bridge deck.

The wave height at the dam is less than for Scenario 2 due to the orientation of the landslide relative to the dam. In this case the landslide is facing in the upstream direction. The wave height 1100 m upstream of the landslide is estimated to be approximately 1.9 m. This height agrees with estimates based on empirical methods described by Pugh and Hubert (ICOLD, 2000).

The wave period is deemed to be too long to produce a dynamic impact wave. Therefore the force on the parapet wall for this scenario is approximated as a hydrostatic force. No dynamic forces were calculated.

Scenario 2:

The wave height at the dam under this landslide scenario is calculated to be 4.71 m. This height can be safely contained below the top of the parapet wall with approximately 0.91 m freeboard. It is also expected to pass safely under the spillway bridge deck.

Given that the wave period equates to approximately 8 seconds, forces from a dynamic impact wave on the parapet wall were calculated using the approach outlined in the Shore Protection Manual 1984 Volume II. The calculated loading that may be applied to the wall as a result of the wave has been allowed for in the design of the parapet wall.

6.5 Response to peer review comments

Table 6.3 includes responses to Opus peer review comments.

Table 6.3 Responses to Opus Stage 1 report peer review on freeboard

Opus peer review comment	Response
Table 11-4 in Section 8.1 (page 45) refers to10% and 1% AEP wave heights on the dam. These will be a significant factor in determining the height of the parapet wall on the dam crest. However there is no discussion of wind wave effects and how these affect the design of the dam crest.	The wave heights referred to wind wave effects. These have been considered in Section 6.2.
What is the general philosophy that is being adopted with respect to design of the wave wall on the dam crest?	The height of the wall is determined by freeboard requirements described in Section 6. The structural design of the wall is described in Section 8.
What is the critical wind direction that gives rise to the maximum wind speeds at the dam site? What is the critical wind direction that is aligned with the proposed reservoir? What wave heights are generated by the maximum wind speeds with the critical direction? What is the runup on the dam face from these wave heights? How does the proposed reservoir operation impact on reservoir levels and hence the height of wave runup on the dam?	Refer to Sections 6.1 and 6.2. In response to the final question, should the reservoir be at a lower level coinciding with a wave height considered in the design, then the wave will be more likely to be contained within the reservoir.

7 Embankment

7.1 General details

7.1.1 Design concept

The proposed embankment is a CFRD construction, very similar to that proposed in the Engineering Feasibility Report (T&T 2009) and in the Stage 1 Design Report (T&T 2011). Minor modifications have been made to the embankment zoning and the zone identification numbers have been changed to conform to international practice.

The development of CFRD design was documented in the 1985 Symposium (Cooke & Sherard, 1985) and by follow-up articles by the same authors (Cooke & Sherard, 1987). These have been followed by a series of international symposia and ICOLD conferences.

There has been little change in CFRD practice for small dams, such as Lee Valley Dam, since the above mentioned Cooke and Sherard (1987). Those changes that have occurred are best summarised in the recent ICOLD Bulletin 141 (ICOLD, 2011) and Cruz et al (2010).

The design and development of CFRD construction has been primarily based on precedent and empiricism. The conventional rockfill embankment batter slopes of 1.3H:1V are roughly the angle of repose of dumped rockfill. The compacted rockfill on a sound rock foundation has no water in the voids and is inherently stable. Stability analyses are not carried out unless the foundation has unfavourable joints or other planes of weakness or, as with the Lee Valley Dam, the dam is subjected to unusually high earthquake loadings.

A large number of CFRD constructions have been completed in Australia, mostly in NSW and Tasmania, but also South Australia, Victoria and Queensland. The highest is the 122 m high Reece Dam in Tasmania. No serious problems have been encountered with these dams, nor with similar height dams constructed overseas. A CFRD design produces a high quality embankment with few problems for a project such as the Lee Valley Dam.

Some dams have suffered from leakage through the concrete face, generally due to poor construction practice. Leakage is a business risk and not a dam safety issue as the design can safely handle flow through the rockfill without the concrete face in place. The 40 m high Brogo Dam (New South Wales, Australia) filled and the spillway operated prior to construction of the concrete face. Although based on an older design with pervious Zone 2B material, the dam handled this situation without difficulty, passing an estimated discharge of 7 m^3/sec though the rockfill. Current designs provide a reasonably impervious Zone 1 material that limits leakage from any face slab deficiencies. The exposed concrete face lends itself to comparatively simple repair operations if excessive leakage does occur.

The CFRD embankment for the Lee Valley Dam has a height of approximately 52.3 m and a crest length of 220 m with the following embankment crest parameters:

- NTWL at RL 197.2
- MDFL at RL 202.53 giving a maximum head of 5.33 m on the spillway crest
- Embankment parapet level at RL 202.83 giving a dry freeboard of 0.3 m.

A typical cross section is shown at Figure 7-1 and the embankment details are shown on the Drawings.

The external batter slopes of 1.0V: 1.5H used in the initial layout have been retained to provide a degree of conservatism for the high earthquake loads. They also allow the use of

a processed gravel in the upstream Zone 2B and the use of coarse gravel material in downstream Zone 4.

7.1.2 Response to peer review comments

Table 7-1 provides a response to Opus peer review comments.

Table 7-1 Responses to Opus Stage 1 report peer review on upstream batter slope

Opus peer review comment	Response
The adopted upstream slope is critical in determining the location of the plinth. As the design and geotechnical investigation work progress there is less scope to vary the slope, despite the comment that final batter slope geometry is yet to be determined.	The final batter slope has been confirmed as 1V to 1.5H

7.2 Foundation excavation and treatment

7.2.1 General foundation

Site investigations (T&T 2012) indicate that Class 1, 2 and 3 rock are all likely to form a suitable general foundation. This will require removal of soils that are locally up to 12 m deep, consisting of slope derived silt and sand and alluvial gravel that overlie bedrock on the left abutment. On the right abutment, scree and colluvium that is generally less than 2 m thick, but is locally up to 5 m thick, will be removed.

Gravel may be left in place but any significant sand deposits need to be removed. While sand is unlikely to be a problem, should it be encountered, it needs to be checked for susceptibility to liquefaction and stability under seismic loading conditions and given the small quantities expected to be involved, removal is preferable.

It is envisaged that the general foundation will be excavated by bulldozers or excavators to expose hard in-situ rock points. Over the majority of the general foundation surface, no treatment is envisaged. However, in the upstream third of the foundation, weak seams and any gravel-filled crevices in the valley base should be excavated with small machinery, such as a 5-tonne hydraulic digger. Overhangs and vertical faces higher than 2 m will be trimmed to 1.0V:0.5H.

Below RL 173 m, additional excavation is required at the downstream toe as a foundation area for anchorage of the mesh covering the downstream face. This is required to provide a surface of better quality rock that will be more resistant to erosion in the circumstance of embankment overtopping and will require clean-up for a concrete slab. Otherwise, clean-up of the embankment profile is only required under the plinth and the adjacent filters as noted below.

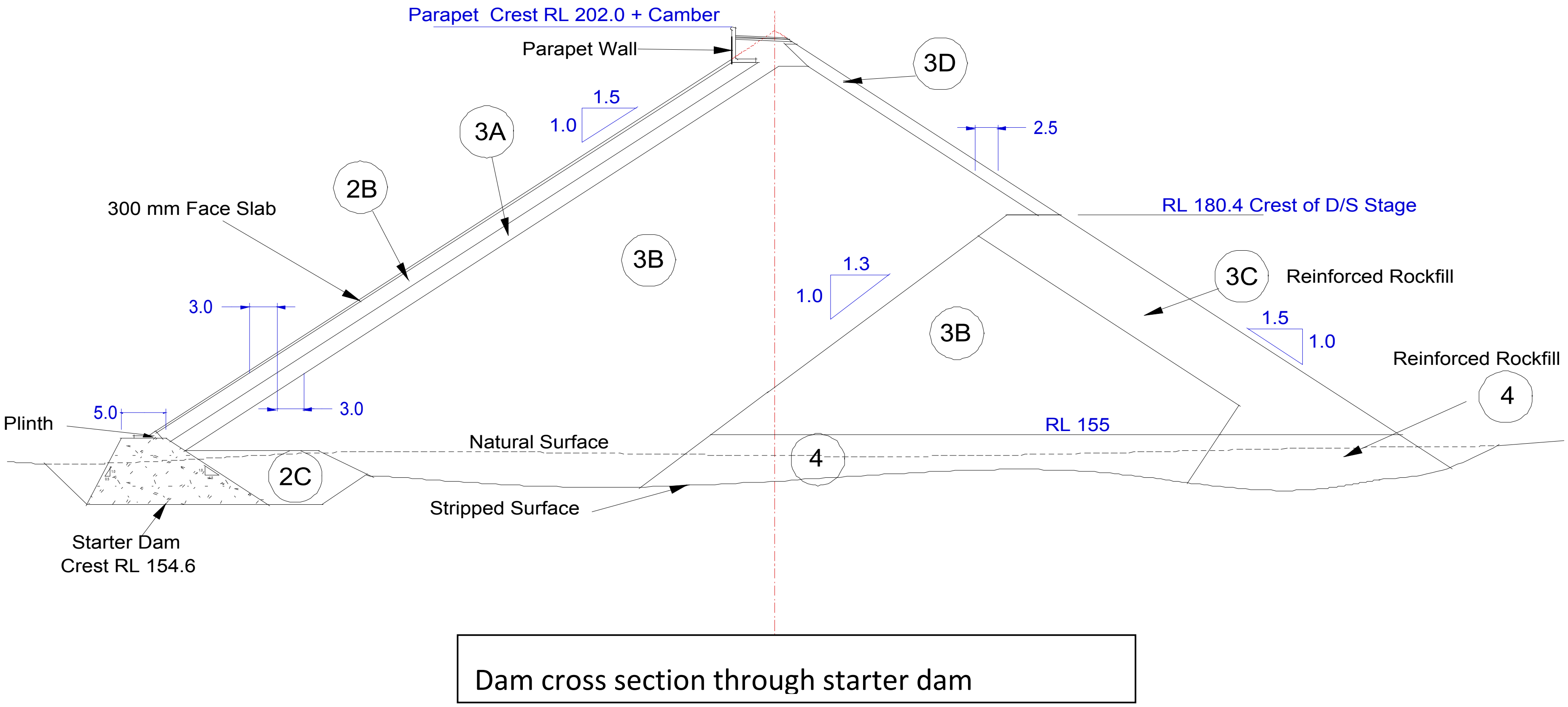


Figure 7-1 Lee Valley Dam Embankment Zoning

Table 7-2 Material description and placement requirements

Zone	Description	Material	Placement
2A	Fine filter	Processed sand filter satisfying grading requirements shown.	Compacted in 200 mm layers to a min RD of 70%.
2B	Semi-pervious zone under concrete face slab.	Processed gravel or crushed rock, satisfying grading requirements shown.	Placed in 400 m layers, compacted with minimum 4 passes.
2B Filter	Special filter for foundation treatment	Processed gravel filter satisfying grading requirements shown.	Compacted in 200 mm layers to a min RD of 70%.
2C	River gravel	River gravel placed behind the starter dam, Maximum size 400 mm and 90% material > 0.075 mm.	Placed in 400 m layers, compacted with minimum 4 passes.
3A	Rockfill transition zone	Free draining rockfill, obtained from class 1 and 2 rock. 50% material > 25 mm; 20% material > 4.75mm and 90% material > 0.075 mm ; Maximum size 200mm	Watered and compacted in 400 m layers with minimum 4 passes.
3B	Compacted rockfill zone	Free draining rockfill, obtained from class 1 and 2 rock. 50% material > 25 mm; 20% material > 4.75 mm and 90% material > 0.075 mm ; Maximum size equal to layer thickness	Watered and compacted in 600 m layers with minimum 4 passes.
3C	Reinforced rockfill	Select fresh large rockfill or processed gravel 400 mm maximum size with 50% larger than 50 mm and 95% > 4.75 mm.	Placed and compacted as shown on drawings.
3D	Facing rock on d/s batter	Select fresh large rockfill	Placed and compacted with excavator
4	Coarse gravel drainage	River gravel, satisfying grading requirements shown	Placed in 600 mm layers, compacted with minimum 4 passes

Note: All compaction by 10 tonne vibratory roller; final layer thickness and number of passes to be determined following trial embankment testing.

7.2.2 Plinth foundation

The plinth is preferably founded on hard, non-erodible, groutable fresh rock although lesser quality rock can be accommodated by lower hydraulic gradients and downstream filter protection. The plinth foundation and any area immediately downstream that is to be provided with shotcrete protection requires a thorough cleaning of the rock surface to obtain a good concrete-rock bond. This area requires:

- Excavation of soft material from joint and shears to a depth equal to twice the width
- Clean-up with air and high pressure water
- Backfilling of cracks, joints, cavities etc. with dental concrete or mortar.

The transition area downstream of the plinth and shotcrete protection requires sufficient clean-up to facilitate inspection and determine the type and extent of foundation treatment.

At the Lee Valley Dam, the plinth has been proportioned such that moderately weathered and less weathered rock is likely to form a suitable foundation. Excavation of lesser quality rock in the upper portions of the rock mass will be required where:

- Rock is closely jointed with a Rock Quality Designation (RQD) of less than 40 or where rock is highly permeable due to dilation
- The rock mass contains bed partings and joints with seams of clay or lesser quality rock.

It is expected that a suitable surface would be obtained on the left abutment and river bed by excavation to refusal using a 40 tonne digger with only localised areas of hard sandstone requiring blasting or a rock breaker. Alternatively extensive blasting could be used to produce long straight lengths that would allow slip forming of the plinth.

The steep right abutment will require blasting to remove around 5 m of dilated rock for plinth construction.

Site investigation to date has shown little in the way of major foundation defects with only one significant shear zone (SZ8) located on the right abutment in drill hole 10. Where the plinth does not provide an adequate hydraulic gradient for foundation defects, the clean-up is extended further downstream and a reinforced shotcrete slab or slab extension is provided. A reverse filter is provided over the shotcrete in case the shotcrete cracks. If the defect infill or shear zone material is erodible, the reverse filter is extended for a distance downstream of the shotcrete (6 m minimum) to allow seepage to emerge in a controlled manner and prevent the migration of fines into the embankment rockfill.

Foundation treatment will include curtain and blanket grouting to reduce seepage caused by foundation disturbance during excavation, and reduce seepage along defects. This is discussed further at Section 7.4.

During construction, foundation quality should be assessed by a Foundation Committee consisting of personnel suitably qualified and experienced in geotechnical dam engineering prior to construction of the plinth or placement of rockfill.

7.3 Embankment zoning

7.3.1 General

The proposed embankment zoning is shown at Figure 7-1 and a description of the materials and compaction requirements is shown at Table 7-2. This zoning is similar to previous arrangements except that:

- The Zone 1A and 1B material covering the lower area of the upstream face has been omitted due to difficulties with incorporating it adjacent to the intake works. These upstream materials are used on some dams to provide a blanket of silty material over the lower perimeter joint to seal any cracks or joint openings. It is not an essential requirement and many CFRD constructions have been successfully constructed without these materials.
- A layer of coarse river gravel has been provided in the river section below RL 155 to provide drainage (Zone 4).

7.3.2 Rockfill

All rockfill will be obtained from required excavations for the spillway and, to a lesser extent, from the diversion conduit and road excavations. While the better quality Class 3 rock is probably acceptable for embankment construction, there is expected to be sufficient Class 1 and 2 material available and these materials have been specified.

The Geotechnical investigations included construction of two trial embankments, one using Class 3 rock and the other using a mixture of Class 2 and 3 rock. All rock was excavated by a 20 tonne digger. This machine was capable of excavating Class 3 rock but had considerable difficulty excavating Class 2 rock indicating that Class 1 and 2 rock will likely need to be blasted.

Compaction was by a 7.5 tonne vibrating roller, a slightly smaller machine than the 10 tonne vibrating roller specified for embankment construction. Up to 9 roller passes were used for compaction of 300 mm layers. This compaction is considerably higher than typical rockfill specifications of 4 passes for a 600 mm layer. Heavy compaction was confirmed by the high field density achieved (2.36 tonnes/m^3).

The gradings are shown at Figure 7-2. The grading of Class 3 rock found in test pits TPR1 and 2 were taken prior to compaction. The grading of Class 2 rock found in test pit TPR3 was taken prior to compaction. Refer to Appendix F for test pit locations and logs.

Also shown is a typical fine limit for hard rockfill as proposed by Cooke (1987). While successful rockfills have used finer materials these are generally regarded as soft rockfill where strength is provided by material density rather than point to point contact and have lower permeabilities.

The grading for the Class 3 material from the test pad shows little or no breakdown into fines following compaction.

The test embankments indicated that the rock will break down along microfractures to produce a small sized but very clean rockfill with a maximum size of 300 mm, less than 25% passing 4.75 mm and less than 2% fines. This grading and the site testing indicate this is a free draining rockfill with a permeability of around $1 * 10^{-1}$ cm/sec that easily satisfies the requirements for a CFRD embankment.

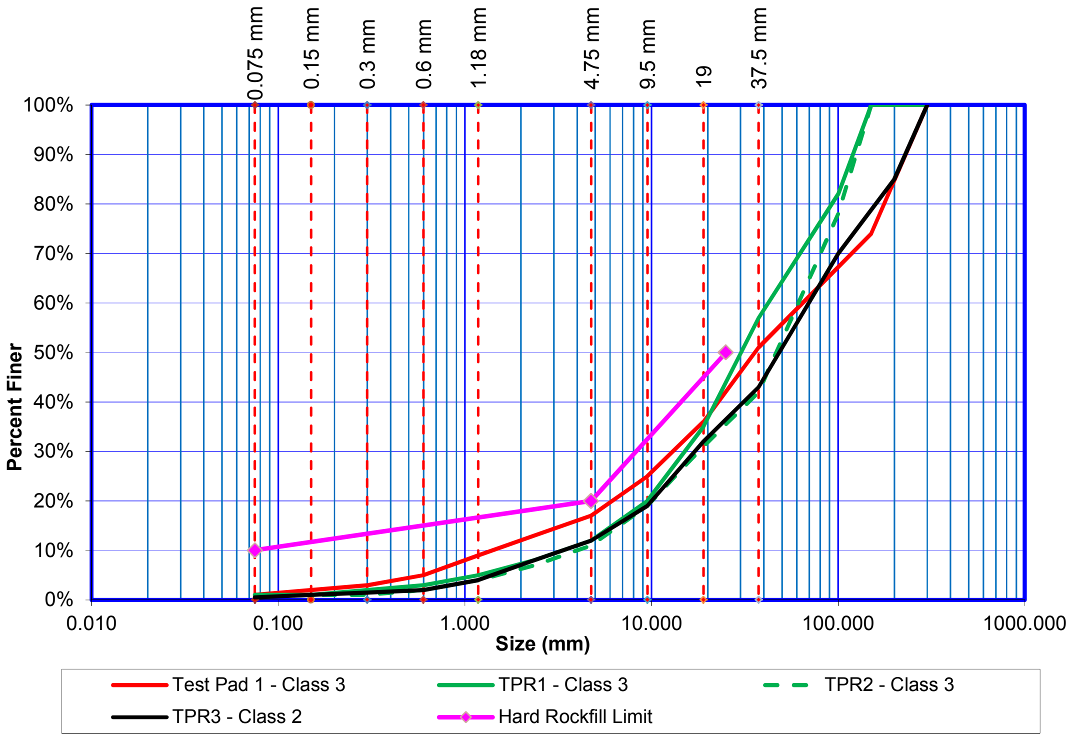


Figure 7-2 Gradings from Trial Embankment Construction

7.3.3 Gravel sources

The Lee River alluvium contains high strength aggregate up to 600 mm diameter. The existing armour layer in the river bed with minus 4.75 mm material removed is a potentially source for Zone 4 material.

7.3.4 Upstream transition & protection zones; Zones 2A, 2B and 2C

Zone 2A is a fine sand filter used in small quantities immediately downstream of the perimetric joint and on jointed or sheared foundations downstream of the plinth. This, together with the adjacent Zone 2B material provides a high modulus fill directly behind the perimetric joint. Zone 2A needs to satisfy conventional filter criteria for retention of joint infill and shear material in the foundation. In practice, a concrete sand is widely used as a fine filter (ICOLD, 1994). A modified concrete sand grading with the coarse boundary adjusted to provide stability protection for a fine dispersive soil is provided as shown at Figure 7-3.

Zone 2B is a sand-gravel sized material with a maximum size of 75 mm that forms the outer upstream layer, upon which the concrete face slab of the main embankment is seated. It provides uniform support for the face slab and acts as a semi-pervious layer to restrict leakage in the event of face slab cracking or joint leakage.

The grading envelope proposed is shown at Figure 7-3 and is similar to that recommended in ICOLD (2010) which in turn is very similar to the traditional envelope proposed by Sherard et al (1985). Minor modifications to the ICOLD envelope included:

- Coarse boundary tightened such that not less than 15% is smaller than 0.7 mm, the standard requirement to retain a silty sand with 35% to 85% passing 0.075 mm
- Fine boundary relaxed to permit a fines content of 2 to 10%, similar to the Casinader recommendation referred to in ICOLD (2010) to reduce permeability. However, it is

important that these fines are non-cohesive and this should be tested using Vaughn's "sand castle".

Zone 2B is expected to be produced from crushed rock. It could also be produced by processing the limited quantities of river gravels. The gradings obtained for river gravel samples TPA 2, 3, 4 and 15 with plus 75 mm material removed are shown at Figure 7-3. The adjusted gradings are a little short of sand sizes and this material would have a higher permeability and be more likely to segregate than an ideal Zone 2B. Sand sizes available from Zone 4 production could be blended to produce the required grading.

Zone 2B is placed in 400mm layers in a damp condition and compacted by 4 passes of a 10 tonne smooth drum vibratory roller. It is sensitive to excess water and the water content is not to be so high that the compaction equipment does not operate on a firm surface. Compaction to 98% of maximum density of the standard laboratory compaction test using minus 19 mm material is used to check the conventional method specification given in Table 7-2.

A modified Zone 2B material referred to as ***"Zone 2B Filter"*** is used where the material covers Zone 2A material. This material has restricted fines to provide better drainage.

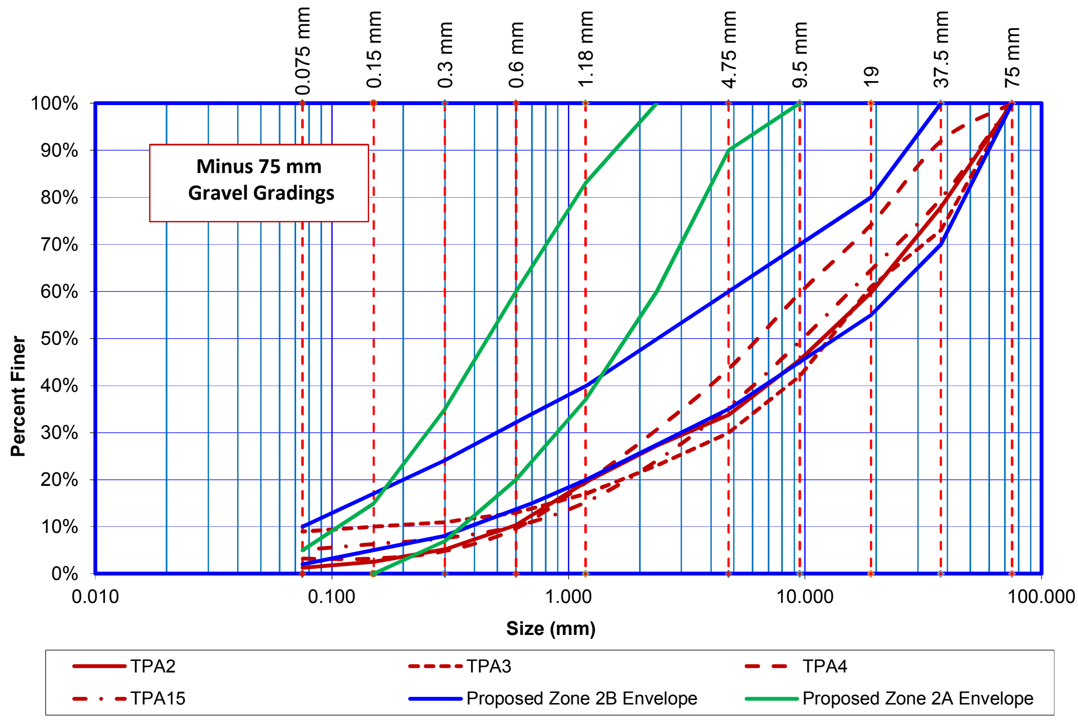


Figure 7-3 Zone 2A and Zone 2B Envelopes

Zone 2B materials do not satisfy conventional filter criteria for segregation such as those provided in ICOLD (1994). The broad grading has a coefficient of uniformity and D_{90}/D_{10} ratios far higher than those required for earth dam filters. This is recognised by ICOLD (2010) and all of the CFRD literature. It has been well established that segregation is not a problem in these materials provided sand sizes exceed 35% and normal care is taken during placement. Lower sand proportions have been used on many dams but require additional care during placement.

Zone 2B Face Protection: The face of the relatively fine Zone 2B material requires protection from rainfall runoff and scour prior to placement of the face slab. Current practice generally uses concrete kerbs placed inside the concrete face slab as shown at Figure 7-1 In addition to protecting the Zone 2B material, the kerbs eliminate the need for face compaction.

The kerbs are a lean concrete mix that is extruded along the face of the dam before the placement of Zone 2B. The height is the same as the Zone 2B layer thickness with the external face at the slope required for the face slab. An inclined internal face provides lateral support for the Zone 2B material during compaction. A 100 to 120 mm wide crest allows some overlap of the kerb for successive layers.

Typical mixes have 75 kg/m^3 of cement, 19 mm maximum aggregate (1170 kg/m^3), sand (1170 kg/m^3), 125 l/m^3 of water and are extruded at 40 to 60 m/hour. Weaker mixes using 60 kg/m^3 of cement have also been used recently. Compressive strengths are around 2 to 5 MPa and Zone 2B can be placed and compacted against it as soon as 1hour after extruding..

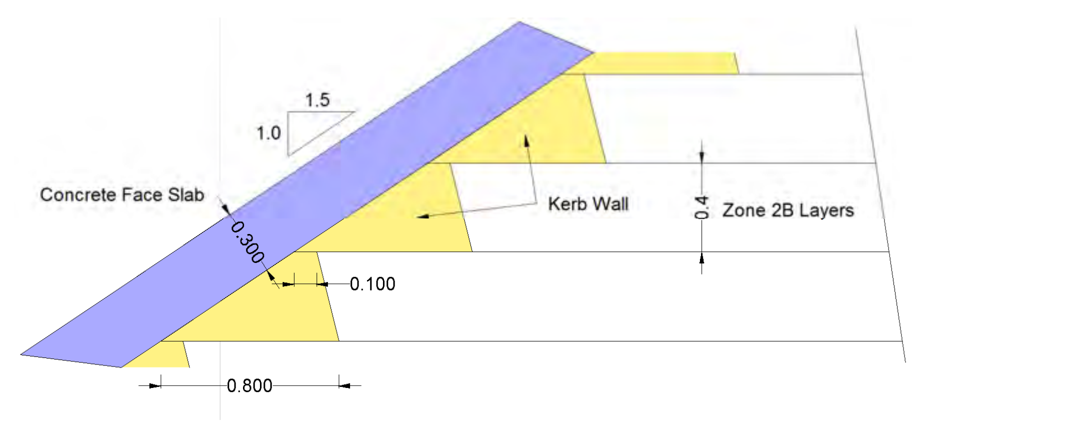


Figure 7-4 Kerb Wall Construction

Zone 2C is an unprocessed sand-gravel with less than 10% passing 75 microns. It is obtained from the alluvial gravels and is placed behind the starter dam. Alluvial gravel has a much higher modulus than rockfill and is used to limit the deformation at the starter dam perimetric joint. It could be replaced with a Zone 2B gravel sourced material but not a Zone 2B obtained from crushed rock.

7.3.5 Rockfill zones 3A, 3B, 3C, 3D and 4

Rockfill is to be obtained from required excavations after removal of the softer Class 3 material. Mechanical excavation of the better Class 3 rock and some Class 2 rock produced the gradings shown at Figure 7-2.

It is expected that the Class 1 and 2 rock will break down on microfractures to produce a similar sized rock or slightly larger material to that shown at Figure 7-2. It is considered neither practical nor necessary to zone the rockfill on the basis of these rock classes and the excavation process will produce a mix of Class 1 and 2 rock.

The rockfill placement requirements shown in Table 7-2 have layer thicknesses that are substantially smaller than normal practice which would typically use 800 mm to 1,000 mm upstream of the centreline and 1,600 mm downstream. The thinner layers reflect the anticipated smaller size of the rockfill. Test embankments will be required during construction to optimise layer thickness and the number of compaction passes required for a competent fill. Where significant changes occur in the excavated rock, new trial embankments will be required to recalibrate.

Zone 3A provides a narrow transition from Zone 2B to Zone 3B that generally satisfies filter criteria. Australian practice (ANCOLD, 1991) has been to specify Zone 3A only by layer thickness and not require a specific grading envelope. Cruz et al (2010) note that Zone 3A is sometimes processed but is generally obtained from finer rockfill selected in the quarry and stockpiled. The rockfill gradings from the trial embankment are compared with the Zone 2B envelope and a theoretical coarse filter at Figure 7-5.

The test pad rockfill gradings are somewhat finer than the theoretical filter requirements. If the test pad rockfill gradings are typical of all rockfill, the only other difference between Zone 3A and Zone 3B is that the former is placed in 400 mm lifts while the latter is placed in 600 mm lifts, producing a slightly finer transition zone. The drawings require Zone 3A to satisfy this coarse boundary only.

Zone 3B is the main rockfill zone. Rockfill in the upstream third of the embankment carries the water load from the concrete face to the foundation. Downstream of the centreline

thicker layers with less compaction could be considered. However, this is a small embankment and given the small size of rockfill, a single specification is preferred. This will produce a slightly stronger rockfill that provides some advantage for the high earthquake loadings.

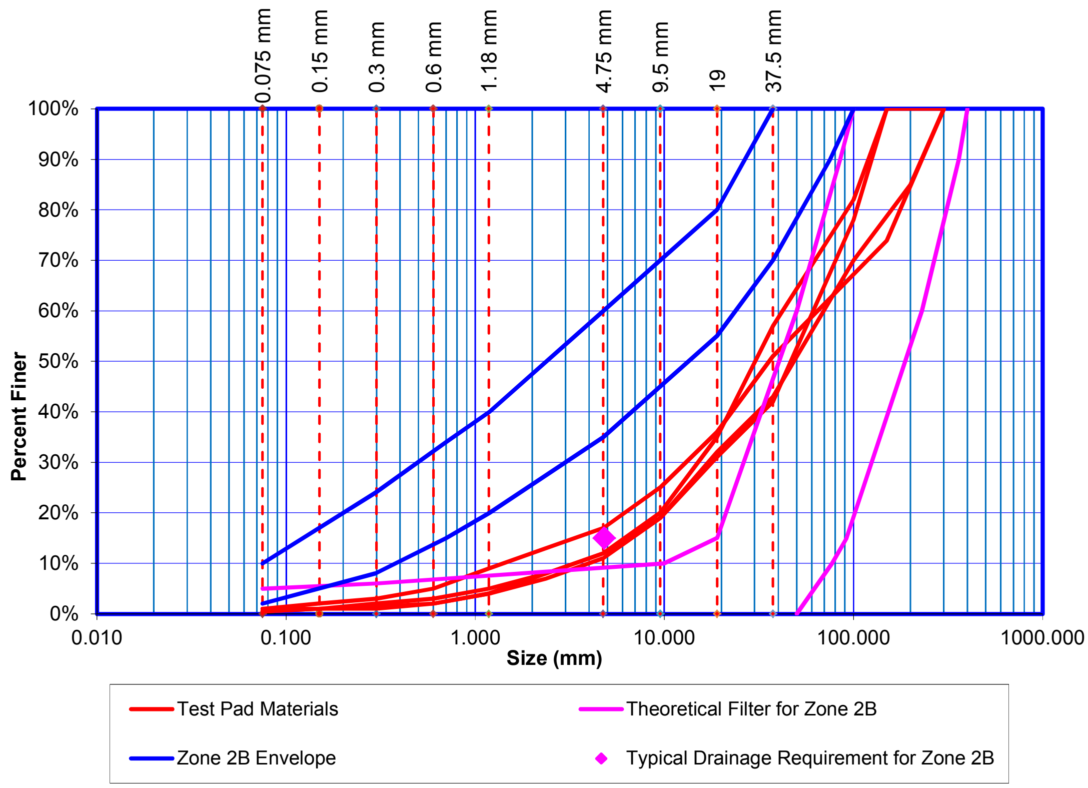


Figure 7-5 Course filter requirements for Zone 2B

The Zone ***3C reinforced rockfill zone*** requires hard sound rock that is free draining and large enough to be retained by the reinforcing mesh. The usual specification is 1.0 m maximum size with 50% larger than 500 mm and 90% larger than 26.5 mm. These large sizes are used on dams where the downstream rockfill zones are placed in 1.6 m to 2.0 m layers and have a maximum rock size equal to the layer thickness.

The Lee Valley Dam rockfill is much smaller and the mesh design has been adjusted to use a smaller mesh (Type 333 with 6.3 mm bars on a 75 mm grid). The reinforced rockfill specification has been similarly reduced and requires a maximum size of 400 mm with 50% larger than 50 mm and 5% passing 4.75 mm. The coarse gravel envelope of plus 4.5 mm material shown at Figure 7-7 would be appropriate.

As noted above, the source of coarse gravel identified to date is limited and quarried rock of this size may be difficult to source. If there is a shortage of Zone 3C material, then a combination of Zone 3B and Zone 4 as shown at Figure 7-6 could be used.

Zone 3D provides a facing of stronger, larger sized material over the downstream face above the reinforced rockfill and is obtained by stockpiling larger rock in the quarry.

Zone 4 is a layer of coarse river gravel in the river section below RL 155 to provide drainage. This is to be sourced from the larger material in the gravel borrow areas as discussed under

riprap in the Geotechnical Report, processed to remove minus 4.75 mm material. It provides a source of high quality coarse drainage material in the initial stages of the construction when high quality material is difficult to obtain. Zone 4 has a higher permeability than the ripped or blasted rock and as shown below, can also be used in the reinforced rockfill zones to provide a larger size material that will not be washed through the reinforcing fabric.

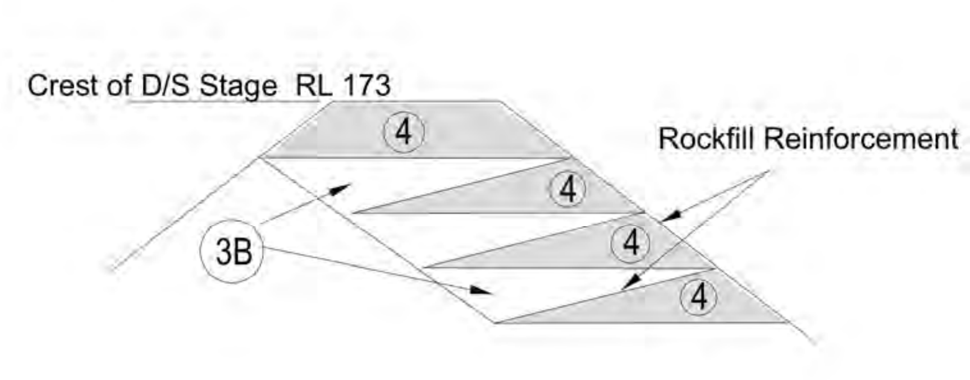


Figure 7-6 Alternative Reinforced Rockfill Detail

7.3.6 Drainage zones

Trial embankment construction indicates a very clean, small sized but free draining rockfill. It was however, a small scale exercise and there remains a possibility that rockfill will break down more than is anticipated producing a less pervious fill. If there is a concern with rockfill permeability during construction of the downstream stage, the embankment zoning may need to be adjusted to incorporate an inclined chimney filter that connects to the Zone 4 gravel zone in the base of the downstream stage.

The chimney filter material should provide filter stability for a finer rockfill with significant breakdown while still providing good drainage capabilities, and a grading similar to that

specified for Zone 4 at Figure 7-7 would be suitable. Additional filter zones between Zone 3B and the chimney filter should not be required.

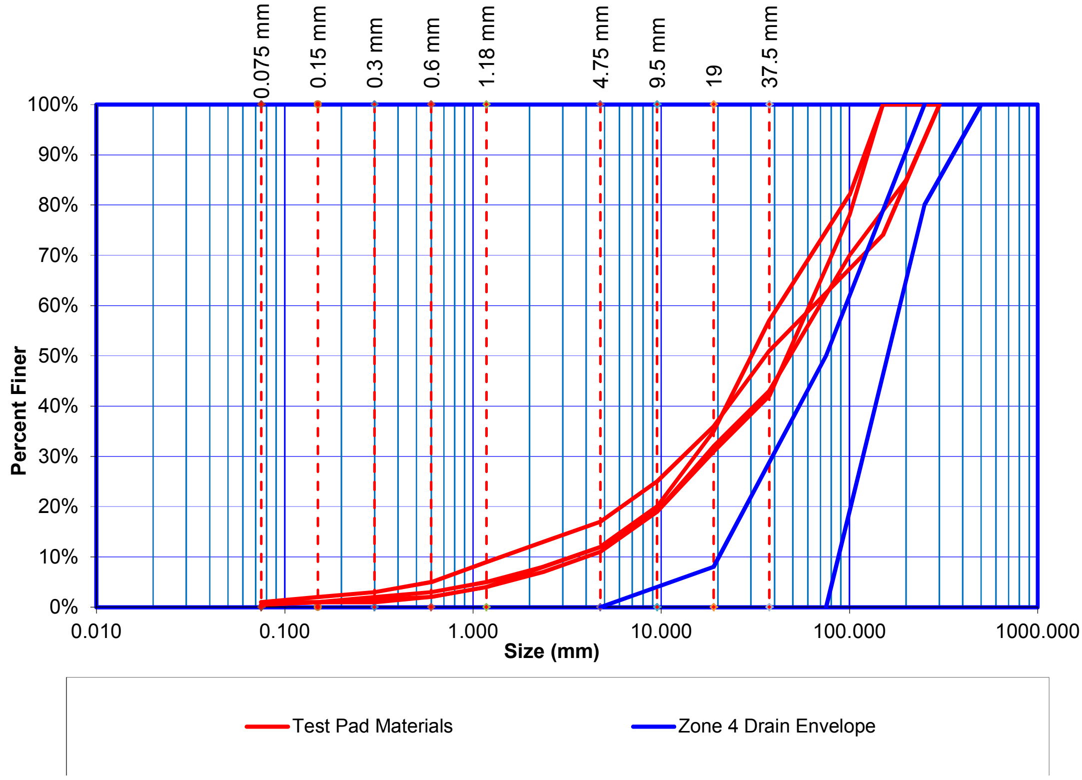


Figure 7-7 Zone 4 Drainage Envelope

7.4 Grouting

7.4.1 Curtain grouting

Curtain grouting along the alignment of the plinth will be required to control leakage beneath the embankment. High and very high leakages in water pressure tests are interpreted to be generally associated with open joints and shear zones. High water takes were experienced at shallow depths.

Investigation drilling along the plinth line consists of relatively short holes. The permeability at the bottom of the holes is open to interpretation. Water takes indicated rock dilation and this is generally accepted as being due to compression of joints above and below the test area. In this case, the assigned permeability is usually taken as the results obtained by lower pressures and would indicate a low permeability at the bottom of drill holes. This is the preferred interpretation as adopted by Houlsby (1990).

An alternative interpretation (Quiñones-Rozo) adopts the water take obtained from a pressure equal to the storage head, and in this case permeability at the bottom of the holes would be generally high.

Site investigations seldom provide sufficient detail for a detailed grouting program. The proposed grouting arrangement provides for 25 to 30 m primary holes at 12 m spacing. These holes will be used to fully investigate the foundation.

Foundation grouting consists of a single line grout curtain for the full length of the plinth and spillway crest flanked by two rows of blanket grouting. An initial arrangement for secondary, tertiary and possibly quaternary holes is shown on the drawings. The final

depth, grout hole spacing and extent of the grout curtain can only be determined during construction, as the results of water pressure testing and grouting become available.

The grouting procedure proposed is downstage grouting without packers. Holes are to be percussion drilled with a minimum diameter of 30 mm. The target permeability standard for the grout curtain is 6 lugeons for surface zones (upper 15 m) as recommended by Houlsby (1990).

In areas of higher permeability, grout takes may be high, and multiple applications of grout may be required. Based on a primary hole spacing of 12 m, grouting is likely to be required to at least Tertiary spacing. Quaternary holes are shown on the drawings.

The defect pattern indicates that inclined holes at 60 degrees to the horizontal should intersect the main defect pattern on the left abutment. Vertical holes have been adopted for the right abutment with a cross-over adjacent to the diversion conduit. Additional angled grout holes specially oriented across major shear zones may occasionally be required.

7.4.2 Blanket grouting

Blanket grouting will be required to reduce seepage due to foundation disturbance during excavation of the plinth. The hydraulic gradient under the plinth is high and blanket grout holes will help to consolidate this part of the foundation and maximise the length of the leakage path. Blanket hole rows upstream and downstream of the centrally placed, grout curtain are proposed. Rows that are 1 m upstream and 1 m downstream of the curtain with holes at 2 m spacing along each row will give an effective spacing of 1m – 1.4 m between each grout hole, including holes of the grout curtain. Blanket grout holes of 5 m depth are proposed.

7.5 Plinth

7.5.1 Plinth design

The face slab is connected to the foundation via a concrete plinth or toe slab. The plinth is anchored to the foundation and a flexible perimetric joint provided between it and the face slab. The face slab is free to “float” on the rockfill face and this joint opens up slightly under water load.

The plinth has the same minimum thickness as the face slab at the perimetric joint (300 mm). A single layer of reinforcement is provided in the top face to prevent cracking but provide sufficient flexibility for the slab to adapt to minor foundation movement. It is anchored to the foundation with triple 32 mm anchor bars at 3 m longitudinal spacing to resist construction loads and pin the concrete to the foundation. Anchorage is based on precedent and foundation characteristics with no specific design requirements. A grouted bar length of 3 m is provided in general with longer bars in areas of moderately to highly weathered rock.

The plinth is subject to a variety of water loads, uplift, and rockfill loads. Conventional plinths of low height on sound rock have high frictional resistance to sliding and are stable. High plinths constructed across low points or overbreak and plinths over weak seams that daylight may be unstable. These require individual stability analyses.

The starter dam shown on the drawings has been proportioned to provide a stable structure that can handle all water loads without any stabilising effect from the rockfill.

While rockfill will tend to provide additional stability, the movement required to develop rockfill forces would likely stress the waterseals.

The plinth is ideally placed on groutable sound fresh or slightly weathered rock. Appropriate plinth widths for a given foundation are generally assessed in terms of the hydraulic gradient across the slab which is calculated as the head divided by the travel path across the plinth. The acceptable hydraulic gradient for a given foundation is a matter of experience and precedent. Widely used criteria include:

- Assessment based on foundation quality as shown at Table 7-3.
- Assessment based on Rock Mass Rating (RMR) values as developed by Materon (Cruz et al, 2009) and shown on Table 7-4
- Assessment based on foundation classifications as shown at Table 7-5.

Table 7-3 Typical Hydraulic Gradients in terms of Foundation Quality; ANCOLD (1991)

Foundation Quality	Acceptable Hydraulic Gradient
Fresh	20
Slightly to moderately weathered	10
Moderately to highly weathered	5
Highly weathered	2

Table 7-4 Typical Hydraulic Gradients in terms of MRI, Cruz et al (2009)

RMR	Acceptable Hydraulic Gradient
> 80	18 to 20
60 to 80	14 to 18
40 to 60	10 to 14
20 to 40	4 to 10
<20	2 Generally handled by excavating to better material or providing a diaphragm wall

The plinth excavation requirement is refusal of a 40 tonne digger. This would remove all Class 3 rock and the weaker Class 2 material. It is expected to produce a moderately to slightly weathered foundation that is slightly erodible to non-erodible with a typical MRI value of 40 to 60 and an RQD of around 50.

The plinth has been designed for a hydraulic gradient of 10, giving a maximum plinth width of 5 m. The minimum width is generally considered to be 3m. The basic slab is detailed for a 3 m width with wider slabs constructed as an extension under the rockfill. The extension is reinforced and connected to the upstream or exterior plinth with a waterseal as shown at Figure 7-8.

Table 7-5 Classification of Foundations, ICOLD (2010)

Foundation Type	Erodibility	Max Hydraulic Gradient	RQD	Weathering Degree	Consist Degree	Discont-inuities	Excavation Class
I	Non erodible	18	>70	I to II	1 to 2	<1	1
II	Slightly erodible	12	50-70	II to III	2 to 3	1 to 2	2
III	Erodible	6	30-50	III to IV	3 to 5	2 to 4	3
IV	Highly Erodible	3	0-30	IV to VI	5 to 6	>4	4

Weathering degree based on I for sound rock, VI for residual soil

Consistency degree based on 1 for hard rock and 6 for friable rock

Discontinuities based on weathered macro discontinuities per 10m length

Excavation classes are 1 for blasting, 2 for heavy rippers with some blasting, 3 for light rippers and 4 for dozer blade

Where the plinth does not provide adequate gradients for foundation defects, the clean-up is extended downstream of the plinth and a reinforced shotcrete extension is provided. Foundation treatment is as noted in Section 7.2.

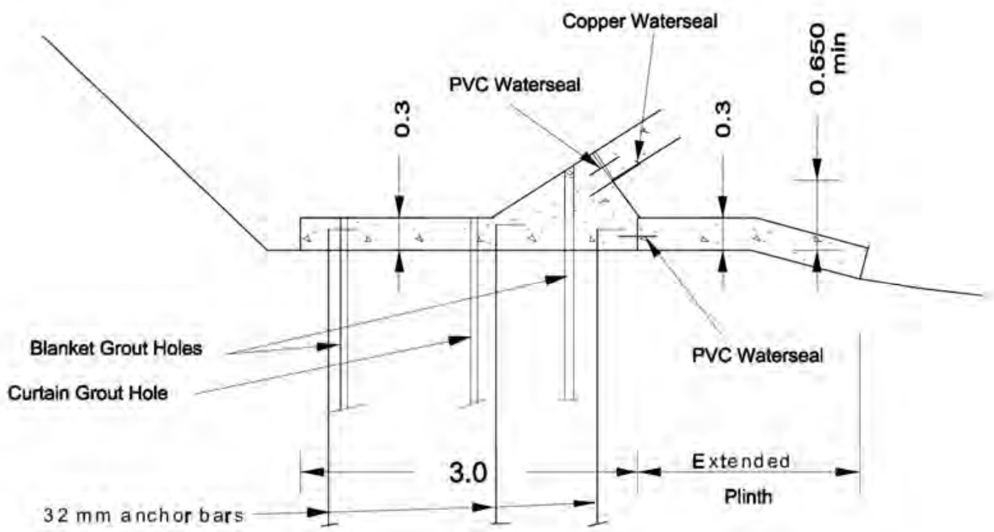


Figure 7-8 Plinth Detail

7.6 Face slab

7.6.1 Thickness and reinforcement

The face slab is slipformed in a continuous operation from foundation level to parapet base level. Vertical contraction joints are typically 12 m to 18 m apart to suit the slip form with 15 m being the most common width. This width is generally left to the Contractor to determine. Where the plinth level varies across the width of a face slab, particularly on the abutments, a triangular starter slab is constructed using conventional formwork. This provides a horizontal surface for commencement of the slip form operation. Face slabs for dams less than 100 m in height generally have a uniform thickness of 300 mm and this is proposed for Lee River.

Concrete strength is not critical and a 25 MPa 56 day compressive strength (or 21 MPa at 28 days) is adequate. Maximum size aggregate of 38 mm, air entrainment and use of flyash (25% of total cementitious material) is standard practice.

A single layer of centrally located reinforcement is provided. Typical reinforcement ratios for CFRD face slabs are 0.3% used horizontally and 0.4% vertically over most of the slab. Horizontal reinforcement is increased to 0.4% within a distance equal to 0.2H from the perimetric joint where some tension may be encountered.

Reinforcement proposed for the Lee Valley Dam is:

- 16 mm bars @ 250 mm centres horizontally and 20 mm bars @ 250 mm centres vertically in the central compression slabs, equivalent to 0.33% and 0.41% respectively
- 20 mm bars @ 250 mm centres in both directions adjacent to the perimetric joint equivalent to 0.41%. This is provided within 10m of the abutment foundation.

Anti-spalling reinforcement is provided at the perimetric joint.

A 4 m high parapet wall minimises rockfill volumes while providing an adequate width of rockfill for slipform operations and minimising wave run-up.

7.6.2 Water seals

A typical perimetric joint detail used in Australian dams (ANCOLD, 1991) is shown on the Drawings and comprises:

- A rear copper waterstop supported by a mortar joint pad
- A central PVC centre bulb waterstop
- A compressible joint filler to prevent edge concentrations of compressive stress during construction and before first filling due to the rockfill settlement. After first filling the joint opens slightly as the rockfill moves downstream.

Recent practice overseas has been to use a water face seal at the perimetric joint, either as a third seal or as a replacement for the PVC centre-bulb. A water face seal comprising a mastic secured by a PVC or Hypalon membrane has been tried in Australia but proved expensive and difficult to construct. An alternative is to use an Omega EPDM type of joint

as shown at Figure 7-9 and this could be adopted as an alternative to the central PVC waterstop if preferred by the Contractor.

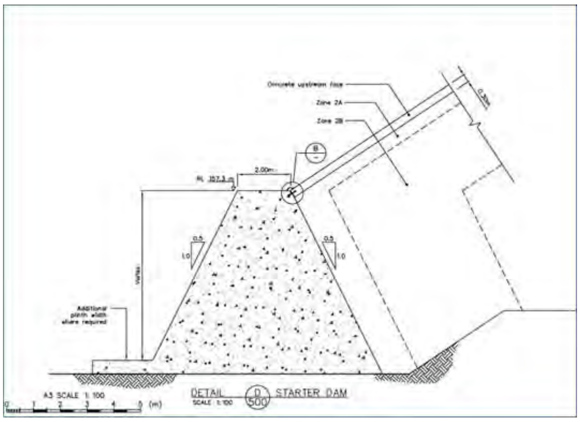
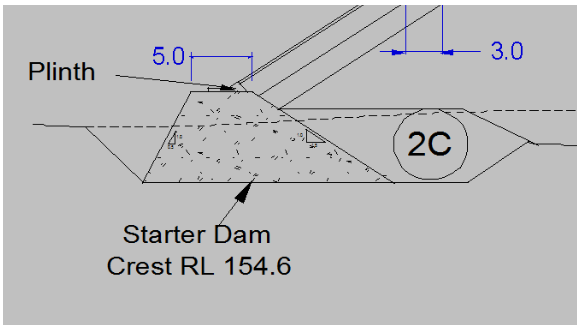
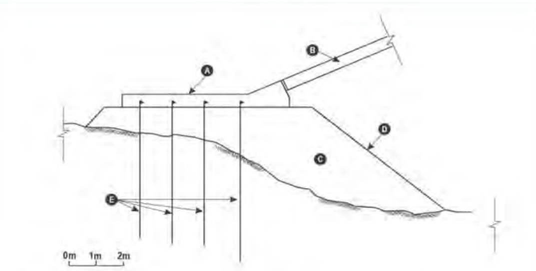
Figure 7-9 External Omega Seal

7.6.3 Response to peer review comments on face slab

Table 7-6 includes responses to Opus peer review comments.

Table 7-6 Responses to Opus Stage 1 Report peer review on concrete face design

Opus peer review comment	Response
We agree that 300mm is an appropriate nominal thickness for the concrete face slab and in line with guidelines for concrete-faced rockfill dams. We presume this is the minimum tolerance; i.e 300mm -0/+xxmm?	Correct
The vertical construction joints in the slipformed panels are shown (dwg 250) with continuous horizontal reinforcement. This detail is inferred to apply to all such joints with no thermal contraction provision across the membrane. Supporting detail is needed to show that the in-service thermal stresses will be adequately accommodated by this approach, especially on a partially full reservoir condition.	This detail has been refined. The reinforcing does not continue through the vertical joints. The concrete face of the CFRD has developed based on precedence rather than by analysis. Concrete thicknesses, joints and reinforcing on CFRD's similar to those proposed for Lee Valley Dam have performed satisfactorily in service. We therefore do not consider that further detail or analysis is required.

The perimetric joint detail at the starter dam (dwg 250) indicates that the supporting internal formed face slope is other than perpendicular to the membrane plane immediately below the joint. Supporting detail needs to be provided to show the performance of the joint under severe seismic loading will not be compromised by this design.	This detail has been refined.  Figure above shows the Stage 1 design commented upon. Differential settlements at the perimetric joint were a potential issue. The design has changed substantially since Stage 1. See figure below.  The Stage 3 design shown below has a downstream slope of 1.5h:1v and is backfilled with relatively stiff alluvial gravels. The combined effect of the stiffer backfill material and flatter starter dam backslope is expected to keep differential movement between the face slab and the starter dam to acceptable levels. In accordance with ICOLD B141 guidelines it avoids a high fill depth adjacent to the perimetric joint and provides a gradual increase in fill thickness with distance from the joint. It is also similar to configurations adopted for the Mohale Dam plinth – See figure below.  Note: section taken perpendicular to toe slab control line Fig. 20 Concrete Backfill Analyzed for Stability, Mohale Dam, Lesotho A Toe slab B Face slab C Concrete backfill D Slope is 2H:1v perpendicular to dam axis E Anchor bars, 4 m into rock

	We believe this design is appropriate for the design seismic loads for the Lee valley Dam.
Evidence of consideration having been given to durability and / or maintenance of the perimetric joint needs to be provided. The basis of the structural separation of the parapet wall from the concrete facing membrane needs to be provided.	Again, this arrangement has been successfully used on a large number of CFRD constructions throughout the world. Dual waterstops (PVC and copper) have been provided and cover to reinforcement generally exceeds NZ building code requirements. Cethana and Wilmot Dams in Tasmania have similar details and are both over 40 years old with leakage of less than 5 l/sec. A dowelled waterstop separates the parapet wall from the concrete face slab is again a conventional parapet arrangement.
it is not clear from the report if the use of kerb wall construction (Figure 7-4) is to be used over the whole dam or only particular areas. For instance the kerb option is not shown in Figures 7-1 or 7-2 or in the details in drawing sheet 530. The use of kerbs to retain the upstream face is considered to be a practical feature, but there are also means of compacting a 1.5:1 face slope.	The kerb wall construction is to be used over the full face area. Earlier constructions overplaced the Zone 2B, compacted the face, cut it back to the final profile and then protected it during construction with bitumen, shotcrete or other systems. The kerb eliminates these operations.
In Table 7-1 Zone 3A has not been included. If kerbs are to be used over the whole face the role of Zone 2A needs to be clarified.	Refer to Section 7 for a description of the embankment zoning.

7.7 Instrumentation

Dams such as the Lee Valley Dam that are constructed in a conventional manner with good quality rockfill have no need for specialised instrumentation such as inclinometers. The essential criteria for satisfactory operation are leakage and embankment settlement and the proposed instrumentation consists of:

- Storage level recording systems
- Seepage measurement weir
- Settlement points on the concrete face, parapet wall, the embankment crest and the downstream batter slopes
- Foundation piezometers, if considered necessary during construction (considered unlikely).

8 Crest (parapet) wall

The parapet wall is approximately 4 m high and has a 4 m wide base slab. The width of the base slab has been determined using software GWALL as the minimum width required to prevent overturning during the MDE (i.e. with a factor of safety FOS = 1).

The wall stem tapers from 350 mm thick at its base to 200 mm at its crest. This taper optimises the quantity of concrete in the wall.

The wall is calculated to slide during the MDE by between approximately 190 mm and 300 mm as calculated using the Jibson method (Jibson 2007). This conservatively ignores possible buttressing from the concrete face on the upstream side of the embankment.

A summary of key wall design parameters is given in Table 8.1. This summary shows that the yield acceleration for the wall is slightly higher than the OBE crest acceleration. Therefore movement (i.e. sliding of the wall) of up to approximately 5mm relative to the foundation may occur. Because the joints have waterbar capable of accommodating approximately 10-20mm vertical or horizontal movement, this displacement is considered acceptable. We recommend that after an OBE event (or higher) the joints are inspected for damage and repaired if necessary.

The wall has been designed to resist the estimated wave impact loading from the landslide generated wave outlined in Section 6.4.

Because the dam as a whole will displace and settle during an earthquake, we do not believe it is practical or economic to design the wall to remain static. The base width has therefore been designed as the minimum width to prevent overturning with a FOS of no less than 1.0 during the MDE.

A W-beam type guardrail runs the downstream length of the dam crest as protection for vehicles on the crest. The guardrail impact loads, embedment depths etc. will be assessed and designed during Stage 4 design.

Table 8.1 Crest (parapet) wall design summary

Description	**Parameter**
Static stability	
Sliding FOS	>5.0
Overturning FOS	>5.0
Seismic stability	
Wall yield acceleration	0.51g
Crest acceleration (OBE)	0.64g
Crest acceleration (MDE)	1.69g
FOS sliding (MDE)	<1.0
Calculated sliding during (OBE) (Jibson 2007 with 1 standard deviation)	1 mm to <5 mm
Calculated sliding during (MDE) (Jibson 2007 with 1 standard deviation)	190 mm to 300 mm
FOS overturning (MDE)	≥1.0

9 River diversion

The selected river diversion comprises the following main components:

- A concrete culvert with two rectangular barrels, each 2.5 m wide by 4 m high and approximately 165m long
- A low height upstream coffer dam with a crest at 154.6 mRL and a diversion wall able to retain flood water to the same elevation
- A starter dam, comprising conventional concrete, in the upstream shoulder of the permanent rockfill embankment with a crest level of 154.6 mRL
- A main coffer dam with a crest at 173.4 mRL, 6 m wide, located in the downstream shoulder of the permanent rockfill embankment.

This main coffer dam is described as the “downstream stage” and will comprise reinforced rockfill (also described as “meshing”) designed to enable large floods to flow over and through the embankment without failure.

9.1 Introduction

Section 9 describes the proposed diversion strategy for the Lee River during construction. It builds on previous work, in particular, hydrological and population-at-risk assessments from the “Lee Valley Dam: Stage 1 Design Report” (T&T September 2011). In addition, the diversion strategy is likely to be refined further once a contractor is selected since the strategy depends on several contractor design elements. It is noted that temporary works are typically within the contractor’s scope of works.

The purpose of the diversion strategy is to allow dam construction while achieving the following objectives:

1. To adequately protect public safety during construction
2. To optimise the balance between the cost of providing diversion capacity and the probable costs of losses incurred if that capacity is exceeded.

The first objective will govern whenever public safety is an issue. Sizing of capacity must be conservative in terms of meeting the first objective, but there is more flexibility in meeting the second objective.

The second objective will depend somewhat on the cost-risk profile a contractor is willing to accept and the premium they incorporate into their tender to accept that risk. However, a “reasonable” cost-risk profile has been adopted for the purpose of sizing components and estimating diversion costs, and the contractor may choose to construct a larger capacity provided that it does not impact on the first objective.

Section 9 starts with a discussion of the background information, specifically the hydrology, persons at risk from a hypothetical breach, and dam break analyses. It also describes the diversion strategy and its key components namely the two concrete culverts, the main coffer dam (or downstream stage), the quick rise berm, the starter dam and the low height upstream coffer dam. Finally a brief discussion of the optimisation of each of these key components is provided.

9.2 Background information: hydrology

The hydrological information used in this assessment comprises:

1. Synthetic inflow hydrographs(Figure 9.1) at the dam site without climate change (since negligible climate change will have occurred at the expected time of construction) as presented in the Stage 1 Design Report (T&T September 2011)

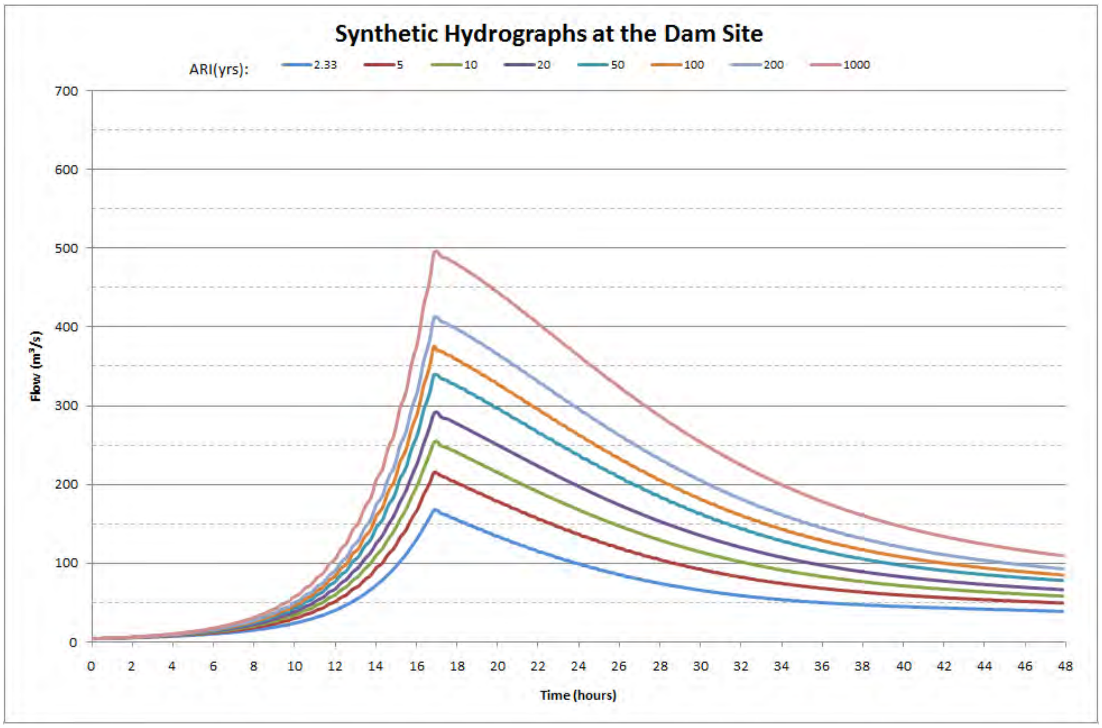


Figure 9.1 Synthetic inflow hydrographs

2. Historical 52 year flow record consisting of peak daily flows between 1957 to 2009 measured at the following gauges:
   - 1957 – 1992 Wairoa at Irvines gauge which has a catchment area of 462 km^2
   - 1992 – 2009 Wairoa at Gorge gauge which has a catchment area of 464 km^2

   Consistent with methods described in McKerchar and Pearson, (1989) peak flows have been adjusted for the smaller catchment area at site (77.5 km^2) by scaling the measured flows in proportion to:

   $(\text{Site catchment area/Gauge catchment area})^{0.8}$

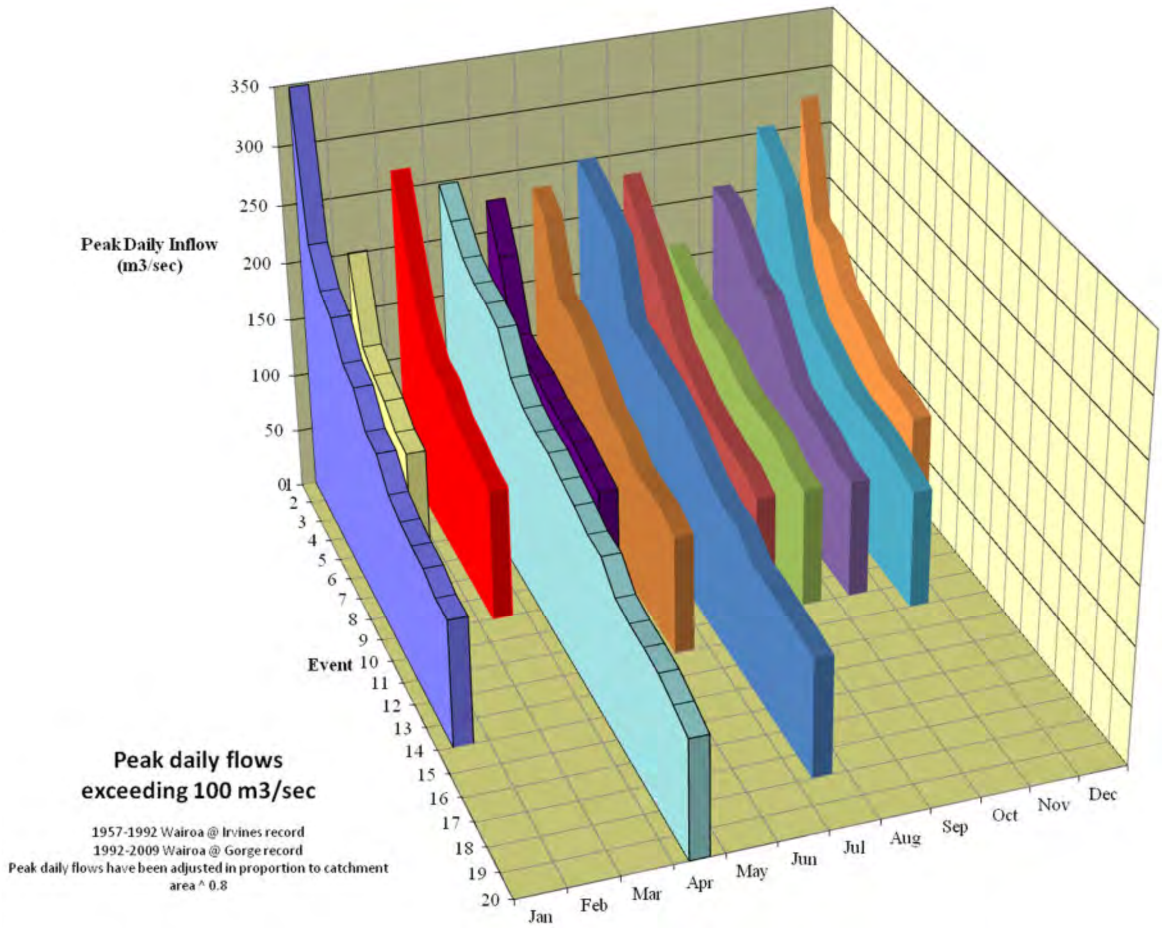


Figure 9.2 Peak daily flows exceeding 100 cumecs

All peak flows exceeding 100 m^3/s (after adjustment for catchment area) during the 52 year record are plotted by month in Figure 9.2 to provide an indication of seasonal variation in flows that could be expected at the dam site.

Seasonality does not appear to have a pronounced impact in terms of number of events and magnitude of peak flows. On this basis, annual AEP events were adopted rather than deriving seasonal AEP estimates.

9.3 Background information: population at risk (PAR)

An assessment of PAR versus embankment height was presented and described in more detail in the Stage 1 Design Report (T&T September 2011). In summary, the assessment utilised a hydrodynamic model of the Lee and Wairoa/Waimea River systems, which extended from the toe of the dam to the coast, to map inundation extents for flood induced dam breaches. The estimation of PAR was based on the inundation extents and depths and available aerial photography and census data.

A diversion conduit discharge capacity must be assumed in order to assess the point during the flood event (e.g. on the rising limb, the falling limb or near the peak of the flood) that the dam breaches. The conduit capacities adopted in the assessment were sized to just overtop the embankment for the given flood event. The estimates of total PAR are reproduced in Table 9.1.

Table 9.1 Estimates of PAR

Scenario	Population at Risk (PAR)	
	1 in 1000 AEP flood	**1 in 5 AEP flood**
No dam	704	150
157.3mRL breach	698 (conduit area 75.0 m^2)	134 (conduit area 35.0 m^2)
165mRL breach	730 (conduit area 40.0 m^2)	157 (conduit area 14.4 m^2)
175mRL breach	710 (conduit area 23.8 m^2)	339 (conduit area 7.0 m^2)
185mRL breach	966 (conduit area 15.0 m^2)	717 (conduit area 2.9 m^2)
194mRL breach	1114 (conduit area 9.4 m^2)	643 (conduit area 0.9 m^2)

NOTE: Diversion conduit capacity sized to just overtop the embankment for the given flood event.

The PAR values in the table do not increase steadily with increasing dam height due to the effect of conduit sizing on the point during the flood event that the dam breaches. For instance, sometimes failure occurs near the peak of the flood inflow hydrograph and sometimes well past the peak during the falling limb. In the case of the latter (failure well past the peak inflow), the PAR is understandably lower as the dam break pulse is superimposed on a lower “background” flood flow.

9.4 Background information: dam break analysis

Additional dam break analysis was completed to determine the dam height at which a hypothetical breach would begin to have implications for public safety. Even though a PAR assessment had been completed, the assessment of changes in flood depth and velocity was considered necessary since hazard to public safety is not necessarily directly proportional to PAR. For instance, even if the same people are exposed to a dam breach flood as the no breach flood, velocities and depths could be higher and more hazardous for the dam breach scenario.

The additional dam break analysis incorporated the proposed coffer dam height and finalised diversion culvert size and configuration (rather than the range of embankment heights and culvert sizes considered in the PAR assessment). A coffer dam with a crest level of 154.6 mRL was selected based on frequency of overtopping with regard to construction nuisance as will be discussed in subsequent sections. The rating curve associated with the finalised culvert size and configuration, derived based on “Hydraulic Design of Highway Culverts” (FHWA 2012), is presented in Figure 9.3.

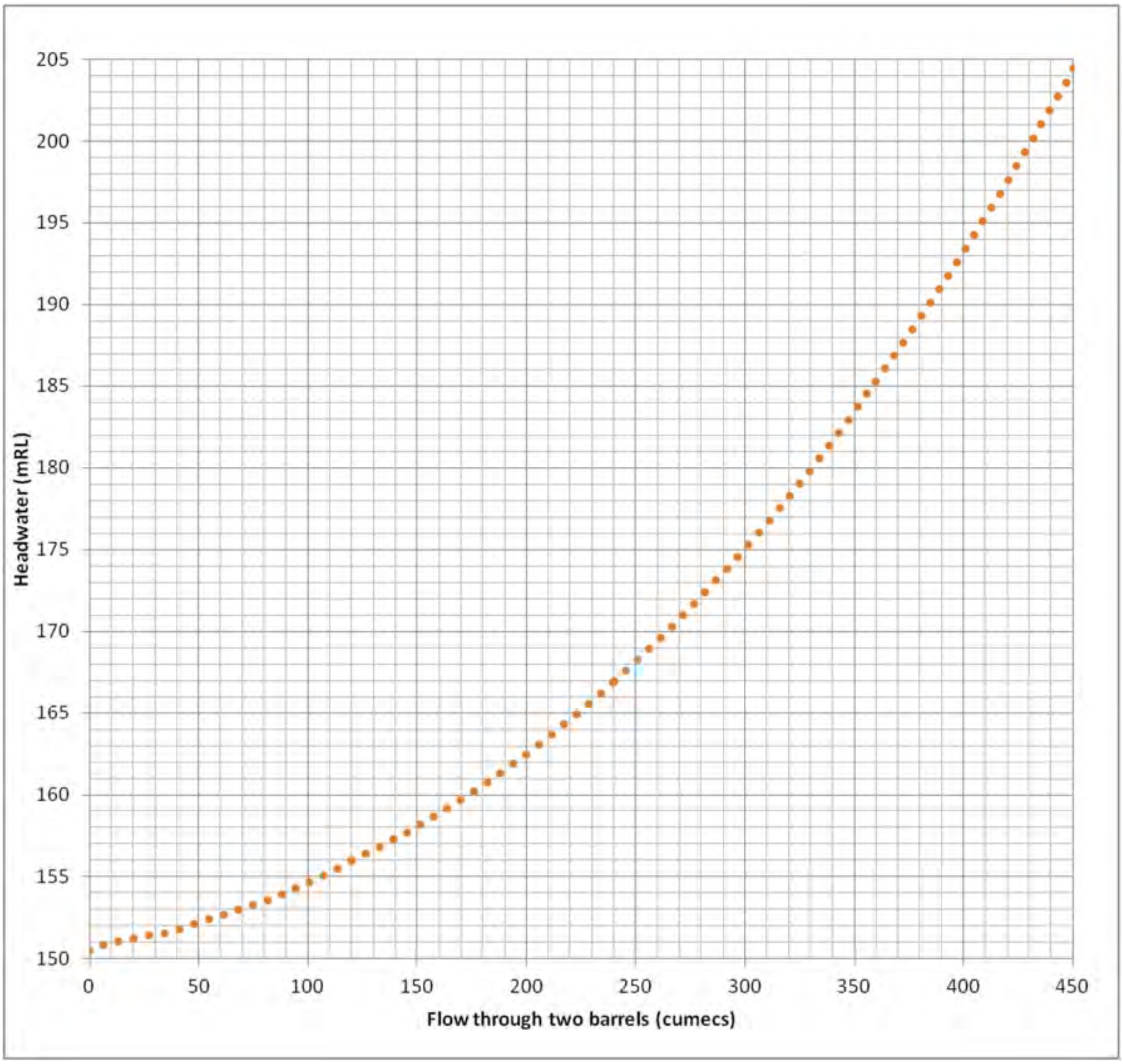


Figure 9-3 Culvert rating curve (twin barrels, each 2.5m wide by 4m high, side tapered)

A hydrodynamic model of the gorge section of the river between the dam and the junction with the Wairoa River was used to assess dam break effects. Changes in flood depth and velocity due to dam break will be greater in the gorge than in the flood plain further downstream because of the confined width available for flow.

A series of routing exercises were completed to identify that a flood with a peak flow of 105 m^3/s would just overtop the crest of the coffer dam at 154.6 mRL at the peak of the hydrograph. This event will be referred to as the “just breaching” flood. As a first step, the water depths modelled for the “just breaching” flood were compared against the water depths for a “no dam” scenario. The change in water depth between the two scenarios is expected to be largest for the “just breaching” flood that just overtops the dam since for larger floods breach will occur during the rising limb of the hydrograph and the dam break pulse will be superimposed on a flow lower than the peak “no dam” flow.

Strictly, the dam breach pulse should be superimposed on the peak inflow rather than peak outflow in order to model the greatest total peak flow. By identifying the “just breaching” flood as described above, the breach will occur at the peak outflow. However, this is considered reasonable because the small storage available at 154.6 mRL provides minimal attenuation and lag between the inflow and outflow peak from the coffer dam. The

minimal attenuation due to the small storage is evident in the routing results at the dam for a "with dam but no breach" scenario presented in Figure 9.4, which shows marginal difference between the inflow and outflow hydrographs. Furthermore, the "no dam" scenario will be essentially the same as a "with dam but no breach" scenario because of the lack of attenuation.

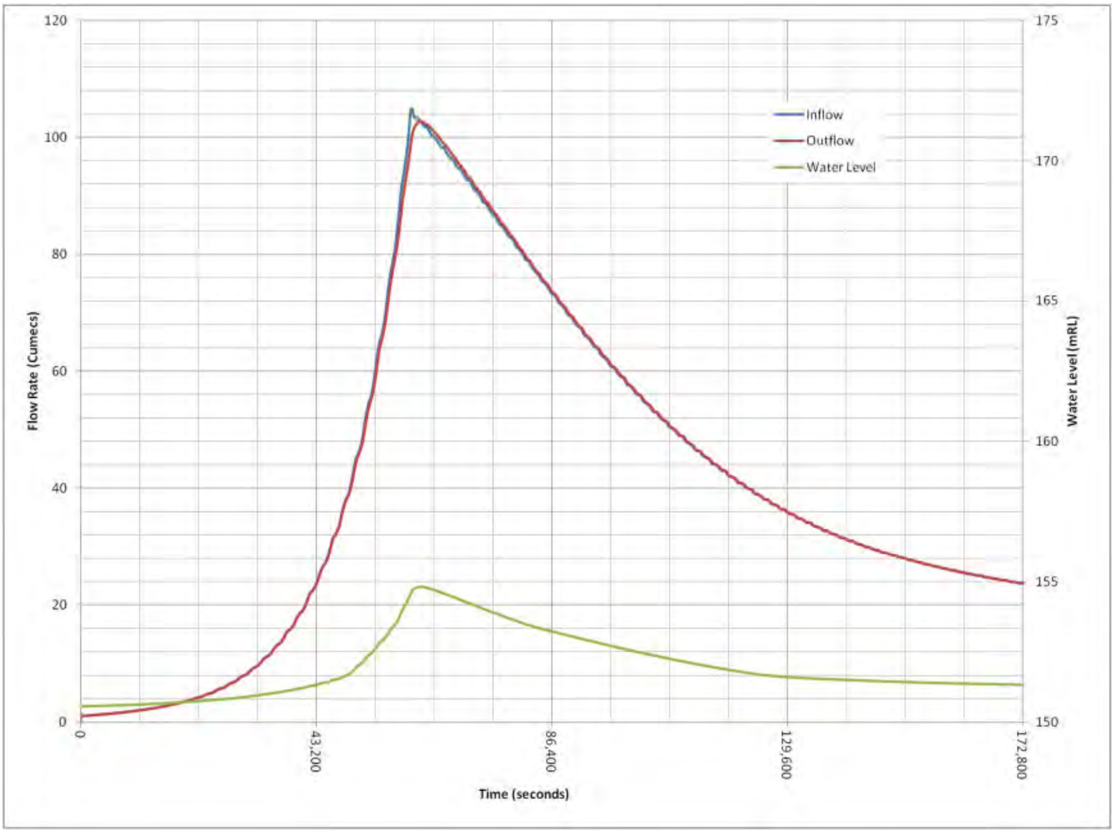


Figure 9-4 Inflow outflow and stage relationship at dam for 105m³/s flood with no breach

The flood depth versus time relationship at the most upstream building (a quarry at model Chainage 7250 m) for the "just breaching" flood and for "no dam" scenario is illustrated in Figure 9.5. The figure shows an increase in peak flood depth at this property of 0.16 m.

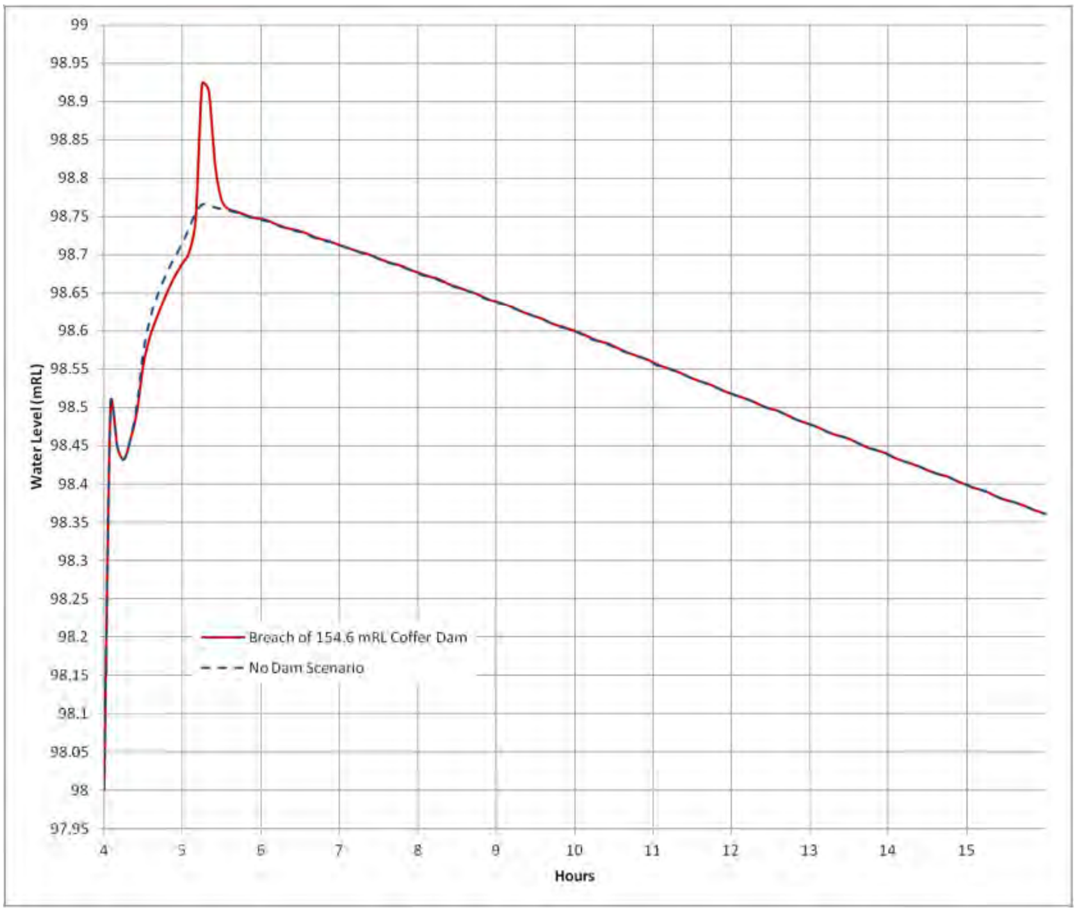


Figure 9-5Water levels modelled at closest building downstream of dam (Quarry with floor level 123 mRL) (Chainage 7250m)

Figure 9.6 presents the modelled water levels over the section of gorge where buildings and dwellings are located. The water levels for the breach and no dam scenarios are too similar to distinguish in the top chart of Figure 9.6. Therefore, the lower chart in the figure presents water level difference on a larger scale at each dwelling/building.

As observed in Figure 9.5, Figure 9.6 also presents the 0.16 m increase in flood depth due to dam breach at the closest building to the dam (Chainage 7250 m). The greatest increase in flood depth due to dam breach is 0.18 m, which occurs at the next dwelling downstream. Further downstream, the flood depth increase due to dam breach generally attenuates, and is estimated to be less than 50 mm upstream of the junction with the Wairoa River.

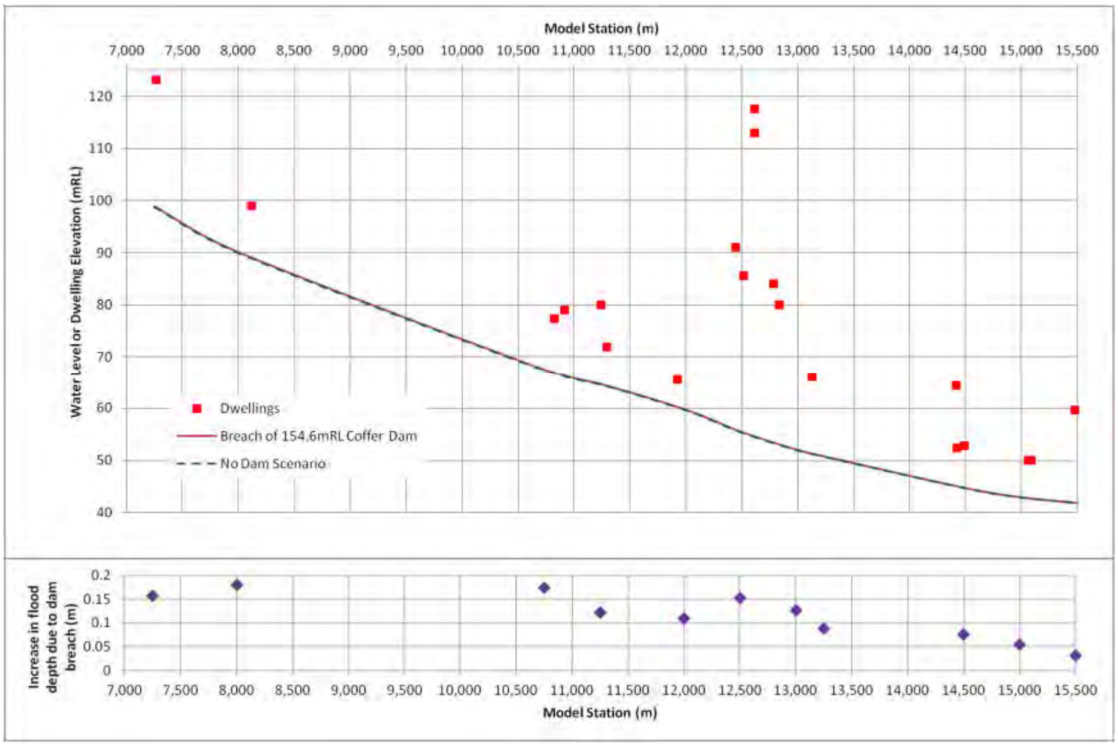


Figure 9-6 Modelled water levels for section of gorge with dwellings

Because the coffer dam is relatively small at 154.6 mRL (when the headwater has risen to the crest of the coffer dam the tail water immediately below the dam will be only 4.4 m lower at 150.24 mRL), the dam break pulse is also relatively small and the "just breaching" flood is a relatively frequent and small flood event. These factors make it unlikely that buildings will be inundated during the "just breaching" flood. Indeed, no buildings (including the quarry at Chainage 7250 m) in the modelled reach between the dam and the junction with the Wairoa River are inundated either by the "just breaching" case or "no dam" scenario as shown in Figure 9.6.

However, even if buildings were to be inundated downstream of the Wairoa River (for instance due to a large flood in the Wairoa River accompanying the "just breaching" flood on the Lee River at the dam) the increase in flood depth due to a coffer dam breach (attenuated to <50 mm by the junction with the Wairoa River) is much less than the recommended threshold for detailed assessment of dam break effects (300 mm ANCOLD 2003, 610 mm Federal Energy Regulatory Commission 1993). The very small increase in flood depth due to dam break is unlikely to be significant in relation to flood depths generated by a large flood on the Wairoa River. Furthermore, the increase will attenuate with distance downstream, and reduce further where the gorge opens up into a flood plain above Brightwater. Investigation into velocities and loss of life due to a coffer dam break is considered unnecessary given the very small increase in flood depths, as supported by the guidelines referenced. In conclusion, because the increase in flood depth due to coffer dam break is extremely small, a coffer dam with a crest level of 154.6 mRL is not considered to have implications for public safety.

Furthermore, because the increase in flood depth due to coffer dam breach with a crest at 154.6 mRL is significantly less than the recommended thresholds, there may be an opportunity to increase coffer dam height further if the contractor identifies advantage in doing so. However, further dam break analysis by the contractor (once appointed) will be

necessary to show that a coffer dam higher than 154.6 mRL will not have implications for public safety.

9.5 Description of proposed diversion staging

The key components of the diversion strategy are:

- A low height upstream coffer dam with a crest at 154.6 mRL and a diversion wall able to retain flood water to the same elevation. This allows the river flow to be shifted between the right and left sides of the existing channel during construction of the components described below (starter dam and culvert). The upstream coffer dam would be expected to fail if overtopped. The height of the upstream coffer dam is restricted so that it will not cause any increase in hazard to public safety above the pre-dam condition (as discussed in Section 9.4)
- A concrete culvert with two rectangular barrels, each 2.5 m wide by 4 m high, providing a combined cross sectional area of 20 m^2. The culvert will be founded on rock towards the true right side of the existing river bed. The face and throat of the culvert are oversized so that the size of the barrel controls hydraulic capacity rather than inlet configuration (FHWA 2012). The culvert will be approximately 165 m long with an inlet invert elevation of 148.8 mRL and outlet invert elevation of 148.4 mRL. The culvert will eventually house the permanent operational outlet works
- A starter dam, comprising conventional concrete, in the upstream shoulder of the permanent rockfill embankment with a crest level of 154.6 mRL. The starter dam and the upstream coffer dam have the same height and can play similar roles during river diversion. The starter dam, in addition to diverting the river, provides a convenient base/enclosure/interface for construction of the culvert and face slab and provides room for the upstream fill cofferdam and conduit entrance. The starter dam will also eventually form part of the permanent plinth line at the base of the concrete face. A crest width of 3 m, upstream batter of 1V:0.5H and downstream batter of 1V:1.5H have been adopted. The relatively flat downstream batter has been adopted to minimise the potential for cracking of the permanent concrete facing due to differential stiffness and settlement in the vicinity of the starter dam.
- A main coffer dam with a crest at 173.4 mRL, 6 m wide, located in the downstream shoulder of the permanent rockfill embankment. This main coffer dam is described as the “downstream stage” and will comprise reinforced rockfill (also described as “meshing”) designed to enable large floods to flow over and through the embankment without failure. An alternative that may be considered is the use of anchored gabions based on precedents in Tasmanian CFRDs. An indicative illustration of the “downstream stage” arrangement is shown in Figure 9-7 (mesh and single bar system rather than gabion system shown).
- A 7 m high “quick rise berm” with a 6 m wide crest at 180.4 mRL, an upstream batter of 1V:1.5H and downstream batter of 1V:2H. The base of the berm would be at the level of the crest of the downstream stage, and would be located within the upstream third of the dam footprint but far enough downstream to avoid complications with Zone 2A and 3A filters. The quick rise berm is located in the upstream third of the dam footprint rather than further downstream so that if the berm is overtopped in a flood event larger than the design event, rockfill from the berm is less likely to be transported down (potentially damaging) the reinforced face of the downstream stage. The height and width of the quick rise berm has been limited to allow rapid construction. The berm will be constructed from the

abutments towards the centre of the valley leaving a 65 m wide gap that can reasonably be closed within a few days.

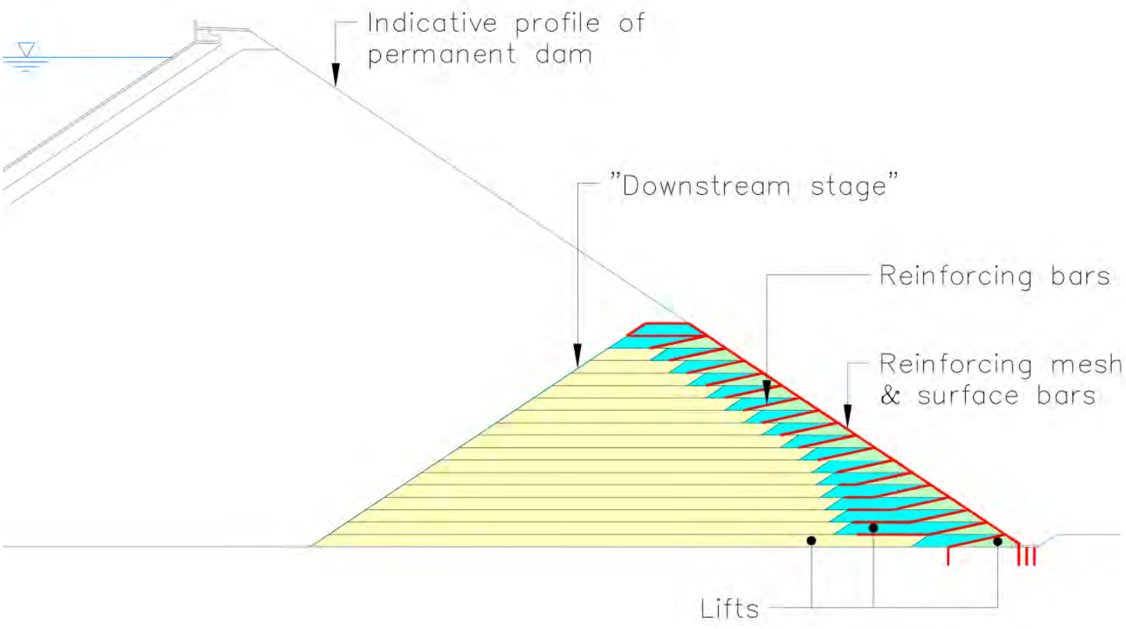


Figure 9-7 Indicative arrangement for reinforced "Downstream stage" of dam construction

The diversion sequence is presented in Drawings 27425-DIV-01 to 03. The sequencing is identified in this report as Steps A, B and C and comprises the following:

1) Step A (River on true left side of channel during starter dam and culvert construction):

A1 Construct a diversion wall dividing the existing channel into left and right sides and tying in to the right bank at the downstream end.

Excavate the left bank of the river to provide a large enough channel for flow between the diversion wall and left bank to prevent overtopping of the diversion wall and upstream coffer dam.

Construct the upstream coffer dam between the diversion wall and right bank of the river, hence diverting the river to the true left side of the existing channel.

A2 Excavate the foundation for and construct the culvert intake and the section of culvert through the starter dam.

Excavate an approach channel to the culvert intake between the upstream coffer dam and starter dam.

A3 Construct the concrete starter dam between the diversion wall and the right bank.

Excavate the foundation for and construct the remainder of the culvert.

2) Step B (River through culverts on true right side of channel during completion of starter dam on left):

B1 Remove the upstream coffer dam between the diversion wall and true right bank of the river, and complete excavation of the approach channel to the culvert.

B2 Construct the upstream coffer dam between the diversion wall and true left bank of the river, hence diverting river flow through the culvert.

B3 Complete the concrete starter dam between the diversion wall and true left bank.

3) Step C (Starter dam complete, river remains diverted through culverts on the true right side of channel):

C1 Construct the main rockfill embankment including the reinforced rockfill downstream stage to manage overtopping of the starter dam.

A description of risks at the different diversion steps is developed in Table 9.2.

Table 9.2 Risks during diversion

Risks and Design Criteria at Different Steps	**Configuration Affecting Performance of Stage**	**Risk and Design Criteria**	
		Relating to Public Safety	**Relating to Cost Risks**
A	Height of diversion wall Height of upstream coffer dam to right of diversion wall Extent of excavation of left bank	Height of diversion wall and upstream coffer dam to be limited to avoid any increase in hazard downstream (e.g. <154.6 mRL as per section above. The assessment described in Section 9.5 is likely to be conservative for Step A because flow capacity through the gap between the wall and left bank is likely to be greater than the culvert capacity leading to a smaller difference in headwater and tailwater at the time of a hypothetical breach. However, Step B is likely to govern component heights.)	Excavation of left bank to be sufficient to avoid overtopping of diversion wall in mean annual flood. The consequences of overtopping comprise inundation of the starter dam and culvert construction area, damage to the upstream coffer dam, and the need to pump out the works area and reconstruct the coffer dam. The cost of additional excavation is balanced against the cost of building a higher diversion wall and upstream coffer dam (but limited to a maximum height for public safety requirement). However, the minimum height of diversion wall is likely to be governed by Step B requirements.
B	Height of diversion wall Height of starter dam Height of upstream coffer dam to left of diversion wall	Height of diversion wall and upstream coffer dam to be limited to avoid significant increase in hazard downstream (e.g. 154.6 mRL as Section 9.5 above)	Capacity of culvert to be sufficient to avoid overtopping of diversion wall, coffer dam and starter dam too frequently. The consequence of overtopping the starter dam is inundation of the area between the starter dam, right bank and diversion wall. The works in this area should be complete at this step so there

	Culvert cross sectional area		should be minimal damage cost. The consequences of overtopping the diversion wall and upstream coffer dam are inundation of the left hand side starter dam construction and the need to reconstruct the upstream coffer dam. The cost of providing a larger culvert is balanced against the cost of building a higher diversion wall, upstream coffer dam and starter dam (but limited to a maximum height for public safety requirement).
C	Height of concrete starter dam Culvert cross sectional area Height of downstream stage & quick rise berm	Top of quick rise berm to extend high enough so that large floods (1 in 1000 AEP [NOTE 1]) can be routed through culvert without overtopping the main embankment beyond the reinforcing. Culverts must be large enough that depth of overflow over the downstream stage will not exceed the limits of reinforced rockfill (interpreted as 4.5 m upstream head (ICOLD 1993)) at any time during the construction of the downstream stage.	Size of culverts and height of starter dam to be sufficient to avoid too frequent overtopping of the starter dam. The consequence of overtopping the starter dam is inundation of the area between the starter dam and downstream stage, or wetting of the upstream face of the main rockfill embankment once the main embankment is above the level of the starter dam. The acceptable threshold for overtopping has been considered by looking at change in diversion capacity with rise of the embankment versus historical floods in the available 52 year flow record.

NOTE 1: Formal guidelines on selection of construction floods are limited (ANCOLD Mar 2000, ICE 1996 and CDA 1999). However, NSW Dams Safety Committee (Demonstration of Safety for Dams – DSC2D Section 6.17) advise that they will accept a flood capacity, during those phases of construction with public safety at risk, in the range of the AEP 1 in 500 to 1 in 1,000 flood discharge on the basis of world practice provided the risks are as low as reasonably practicable.

The following sections present the calculations and assessments completed to show the design criteria described in Table 9.2 are fulfilled.

9.6 Height of downstream stage and quick rise berm

The NSW Dams Safety Committee (Demonstration of Safety for Dams – DSC2D Section 6.17) advise that they will accept a flood capacity, during those phases of construction with public safety at risk, in the range of the AEP 1 in 500 to 1 in 1,000 flood discharge on the basis of world practice provided the risks are as low as reasonably practicable (ALARP). NZSOLD (2000) do not specify flood diversion standards and note that there "*... appears to be no universally accepted standard for selecting the size of flood for construction diversion facilities and the choice is generally based on risk or optimisation of diversion capacity versus construction costs and damage costs.*" This is consistent with what is adopted here and presented in the Stage 1 Design Report (T&T 2011).

The ability to safely pass a flood of this magnitude can be provided more economically by the incorporation of a reinforced downstream stage able to withstand overtopping plus a quick rise berm rather than by enlarging the culvert.

As the embankment rises during construction, the head available to drive flow through the culvert increases, thus increasing the capacity of the culvert. As shown in the routing presented in Figure 9.8, once the embankment reaches an elevation of 180.4 mRL the 1 in 1000 AEP flood can be passed entirely through the culverts without any overtopping. Therefore, the quick rise berm provides protection to a crest level of 180.4 mRL. Meshing is only continued up to an elevation of 173.4 mRL, at which level a flood between the 1 in 100 AEP and 1 in 200 AEP event can be passed entirely through the culverts without any overtopping. This leads to a downstream stage in the order of 26 m high and a quick rise berm 7 m high.

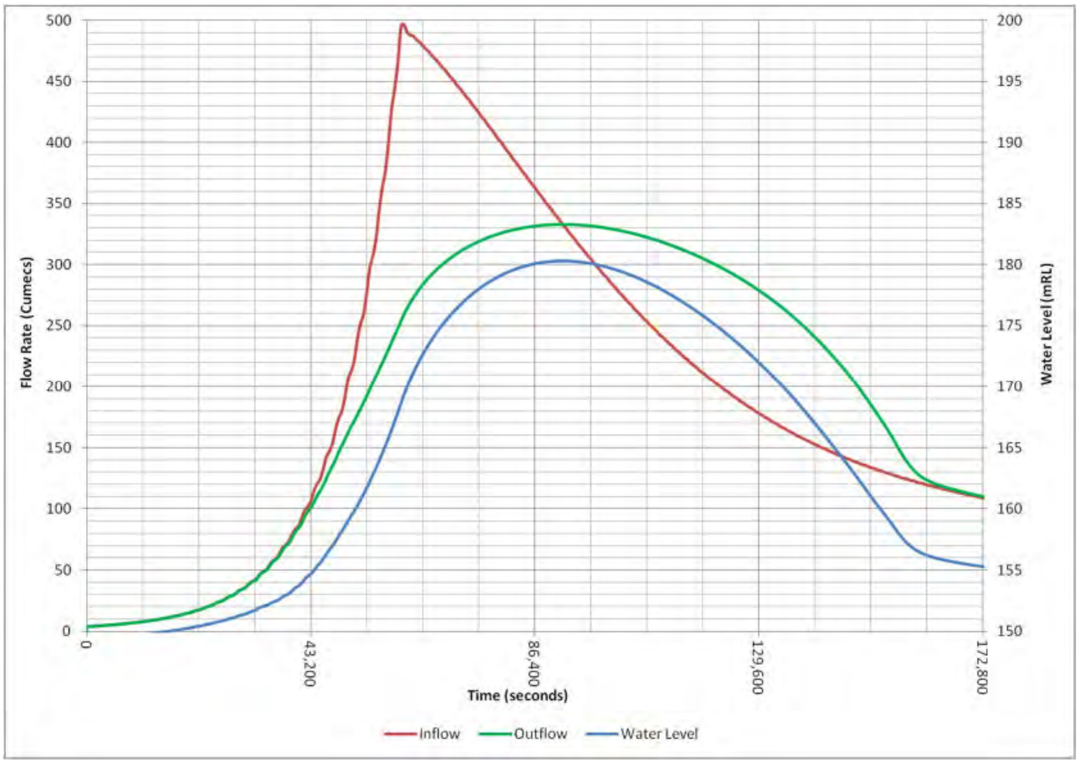


Figure 9.8 Flood routing during construction

Stability of the downstream stage and quick rise berm under flow through conditions has been assessed and findings are presented in Section 12.

9.7 Depth of flow over meshing

The crest width available for overflow ranges from 65 m (starter dam width) to 215 m (main embankment width at completion of construction).

Ignoring the significant beneficial effects of routing and flow through culverts (e.g. assuming no attenuation of peak flows due to storage and assuming the culverts are blocked and all flow passes over the top of the downstream stage), overflow depth for the smallest width available of 65 m would cause an upstream flow depth of 2.9 m for the 1 in 1000 AEP event (497 m^3/s) above crest level. (An overflow coefficient suitable for a broad crested weir of 1.55 has been assumed.) While noting that there is limited performance data available, ICOLD (1993) suggests flow in excess of an upstream head of 4.5 m (equivalent to a critical depth of 3 m) "may be considered dubious, although overflow depths up to 10.5 m have been sustained without damage".

Once the effects of routing and flow through culverts is taken into consideration, there is likely to be scope to restrict the width of overtopping while still remaining within ICOLD (1993) overtopping depth guidelines. Restricting the width of overtopping would require additional meshing (channel side walls) and introduce complexity into lift sequences, which is considered likely to outweigh any benefits relating to restricting the width of overtopping. As a result, meshing across the full width of the embankment has been assumed for costing purposes. However, it is noted that design of reinforcement for the downstream stage is a contractor design element, and the contractor selected may choose to submit a design for approval that involves restricting overtopping to only a portion of the embankment width.

9.8 Detailing of meshing / downstream stage

Aside from the height and width of meshing discussed above, the following details are also relevant to the performance of the downstream stage:

- Meshed rockfill placement to commence from the abutments and proceed towards the centre of the dam
- The extent of rockfill placed ahead of mesh protection to be limited to ensure unexpected inflows do not overtop an uncompleted lift line
- Unreinforced fill levels to be kept below the level of the anchored mesh and all rockfill to be compacted prior to overtopping (to prevent unreinforced fill being washed down the face of the mesh and damaging the mesh)
- Embedment bars to be sloped to prevent progressive erosion of the layers of rockfill and reinforcement. Progressive erosion refers to the process described by ICOLD (1984) whereby the top layer of unreinforced rockfill becomes eroded, exposing the mesh which in turn becomes unconfined allowing erosion of the next layer of rockfill and so on. The sloping of embedment bars allows immediate placement of rockfill up to the level of the top of reinforcement to provide confinement
- A small number of rockfill gabions to be kept on site to secure partly completed mesh if overtopping is anticipated
- Specific face layer of larger size rock to avoid rockfill moving through mesh during flow through
- Mesh to be constructed tightly against rockfill to prevent movement during flow through. Containment concrete used to fill any large gaps
- Fill placement that channels overtopping flows to be avoided (except where channel sides have been reinforced and flow depth of channelized flow has been assessed)

- Embedment length of reinforcing to be sufficient to prevent deep seated failure
- Mesh to be anchored into abutments and the abutment groins to be protected from erosion by concrete.

9.9 Detailing of meshing / downstream stage with regard to debris

Significant quantities of felled timber have been abandoned on steep slopes in the catchment. The possibility that the timber could mobilise during a construction flood event and need to be passed down the downstream face of the dam without damaging the mesh has been considered. Logs could potentially be mobilised by the following mechanisms:

a Logs being inundated in the area immediately upstream by water ponded behind the downstream stage. This would be low velocity water but may cause logs to float downstream

b Logs being floated by high velocity in the river due to an extreme inflow, substantially larger than recent river flows

c Local landslips into the storage in areas where the logs are stacked.

Standing trees and felled logs that will be inundated by the final reservoir are expected to be removed for water quality purposes as part of the reservoir clearing works, and this should negate the potential for the mechanism listed as “a” above. The mechanism listed as “c” above is also expected to be negated through a process involving inspection of slopes immediately surrounding the storage for potential zones of instability and removal of any logs that could be affected by the unstable zones identified.

A literature search of historic meshing failures identified three instances of failure due to debris:

1. Bridle Drift Dam (1966) as reported by Pells (1978)
2. Cethana Dam (1968) as reported by Fitzpatrick (1977)
3. Paloona Coffer Dam (1969) as reported by Fitzpatrick (1977) and HEC (1969).

Only in the Paloona Coffer Dam failure was the debris timber. In the remaining two failures incomplete layers of rockfill (yet to be reinforced) were washed down the face and failure was due to rocks impinging on the mesh and breaking it to form a hole through which the rockfill piped. These two failures would have been avoided by the provisions described in Section 9.8 and being adopted for the Lee Dam. Moreover, these failures involved relatively light mesh, specifically chain link fencing mesh at Paloona and Cethana and weldmesh comprising 7 mm diameter wires horizontally at 150mm spacing and 5 mm diameter wires down the face at 225 mm spacing at Bridle Drift Dam. The mesh proposed for the Lee Dam is 333 mesh (6.3 mm diameter bars at 75x75 mm spacing).

A well-documented case where debris was passed over reinforced rockfill with only minor damage is provided by Googong Dam. As noted by Pells (1978) “the surface mesh was quite considerably heavier than that used at Xonxa. Thus a small break in the mesh would not easily ‘run’ to cause a major hole as at Bridle Drift”. A detailed description of the overtopping at Googong Dam is provided by Fokkema et al (1977). The rockfill was faced with F81 mesh (8 mm wires at 100 mm centres each way) and held in pace by 20 mm bars at 0.5m centres which pass through the mesh to provide anchorage back into the rockfill. The reinforced rockfill was overtopped for 33.5 hours in total (two peaks). Debris was observed catching on the crest early on, but was carried over as the water level rose. Once the water depth over the crest was greater than 1.5 m, debris generally did not catch and

flow was relatively smooth except at abutments. Subsequent to the flood, it was observed that in many places the horizontal wires of the mesh had broken away from the sloping mesh and been pushed downstream, but in all cases the mesh overlap and anchor bars had restricted the extent of this occurrence and the sloping mesh wires were effectively held in place. The worst damage was from a haul road washing away. One sheet of mesh was installed with the horizontal strands on the underside, and this suffered very little damage, suggesting this could have been a better way to place the mesh. At the crest there is a danger with sheets placed this way that all sloping wires of a sheet could be stripped off by debris caught in the horizontal wires. Therefore, the authors (Fokkema et al 1977) concluded that the positioning with the horizontal strands on the outside was preferable. Other guidelines (ICOLD 1993 recommend placing downslope bars on the outside to minimise damage by logs and loose rocks, as was done at Clarrie Hall Dam.

Aside from heavier mesh, another defensive measure that has been adopted historically by the Hydro-Electric Commission (H.E.C) is the use of a gabion system rather than an individual bar system. Some of the key features of the gabion system with regard to debris resistance have been cited (Fitzpatrick 1977) as:

- A second line of defence in the form of the grid mesh on the upstream, and buried, side of the gabion
- Isolation of a local failure in the face mesh from adjacent superior and inferior gabions by the horizontal grid mesh in between them.

Notwithstanding the comments above, in the mesh and individual bar system the 20 mm diameter bars restraining the mesh provide additional protection not offered by the gabion system.

Paloona Coffer Dam was rebuilt using the gabion system, and survived three subsequent floods, the third lasting for 35 hours. Some 70 logs were passed without damage (HEC 1969).

The mesh and bar system is considered appropriate for the Lee Valley Dam based on the precedent at Googong and the reservoir clearing expected. The assumption that the gabion system would be significantly more expensive than the mesh and bar system should be confirmed by the final contractor selected.

In conclusion, the following defensive measures with respect to debris loading should be considered in the contractor's design of reinforcing in the downstream stage for the Lee Dam:

- Heavier than standard mesh
- Detailing regarding order in which downslope bars and mesh strands are laid on the rockfill relative to the transverse horizontal bars
- Consideration of the gabion system if additional cost / programme issues are similar or better than for the mesh and individual bar system.

9.10 Starter dam height (also height of diversion wall and upstream coffer dam on left hand side)

The starter dam height has implications for the number of times construction works are inundated and the cost associated with this construction nuisance. The starter dam height does not have implications for public safety since it comprises concrete and can be overtopped without unravelling.

In order to keep the starter dam as small as possible while ensuring inundation of works is not unreasonably frequent, the number of times the starter dam would have been overtopped if construction had commenced in any year of the 52 year flow record has been assessed. Commencement of construction in different months has also been considered to provide some indication of the variation in overtopping frequency with season, though this was not expected to be significant based on the assessment presented under the section on hydrology above.

The assumptions involved in the starter dam overtopping assessment are as follows:

- There is expected to be negligible risk of overtopping until the river is diverted through the culvert during the second half of Step B when constructing the concrete starter dam on the left side of the channel. This is because the excavation to widen the channel on the left hand side during Step A can be oversized at relatively little cost to provide a high level of confidence that the diversion wall and right hand side upstream coffer dam will not be overtopped. (Channel widening currently sized to avoid overtopping the starter dam up to the mean annual flood.) Therefore the starter dam overtopping assessment has only considered a period of approximately five months between diversion into the culvert and completion of main embankment rockfill.
- The height of the top of the main embankment during this five month exposure period has been estimated:
    - Assuming two weeks (after diversion into the culvert) to complete the starter dam on the left hand side (Note the upstream coffer dam during Step B is assumed to be the same height as the starter dam)
    - Assuming a rate for rockfill placement of 20,000 m^3 per week (For comparison, Cruz (2010) states that for high dams it is normal to place 125,000 m^3 per week)
- The peak inflows from the 52 year flow record have been converted to flood rise (assuming no overtopping e.g. all flow through culvert) by reference to a rating curve (of flood rise versus peak inflow) developed by completing a series of routing exercises floods with incrementally increasing peak flows.

The height of the main embankment over the five month period has been based conservatively on constructing the main embankment concurrently with the downstream stage such that the top of the lift for the downstream stage is the same as the elevation of the top of the lift for the upstream portion of the main embankment. In reality, the downstream stage should be constructed in advance of the upstream portion of the main embankment to increase embankment height as quickly as possible and thus minimise frequency of overtopping. Construction of the upstream portion of the main embankment would commence when the downstream stage approaches its maximum elevation of 173.4 mRL and the work rate possible on the downstream stage decreases (due to reduction in crest width of the downstream stage with elevation). However, there would then be a delay in increase of overall embankment height while the upstream portion of the main embankment is raised to 173.4 mRL prior to construction of the quick rise berm, such that the *overall* exposure time until the embankment is increased to 180.4 mRL would

be similar to that assumed in this analysis. Nevertheless, the frequency of overtopping predicted by the analysis is conservative because the analysis predicts more frequent overtoppings than likely when the embankment is below 173.4 mRL. This conservatism is offset by not allowing for any close down period over Christmas.

Figure 9.9 illustrates the comparison between embankment level and flood rise over the 52 years assuming diversion had started in December or April in each year. The starter dam and upstream coffer dam level is shown as 154.6 mRL as selected for the final configuration.

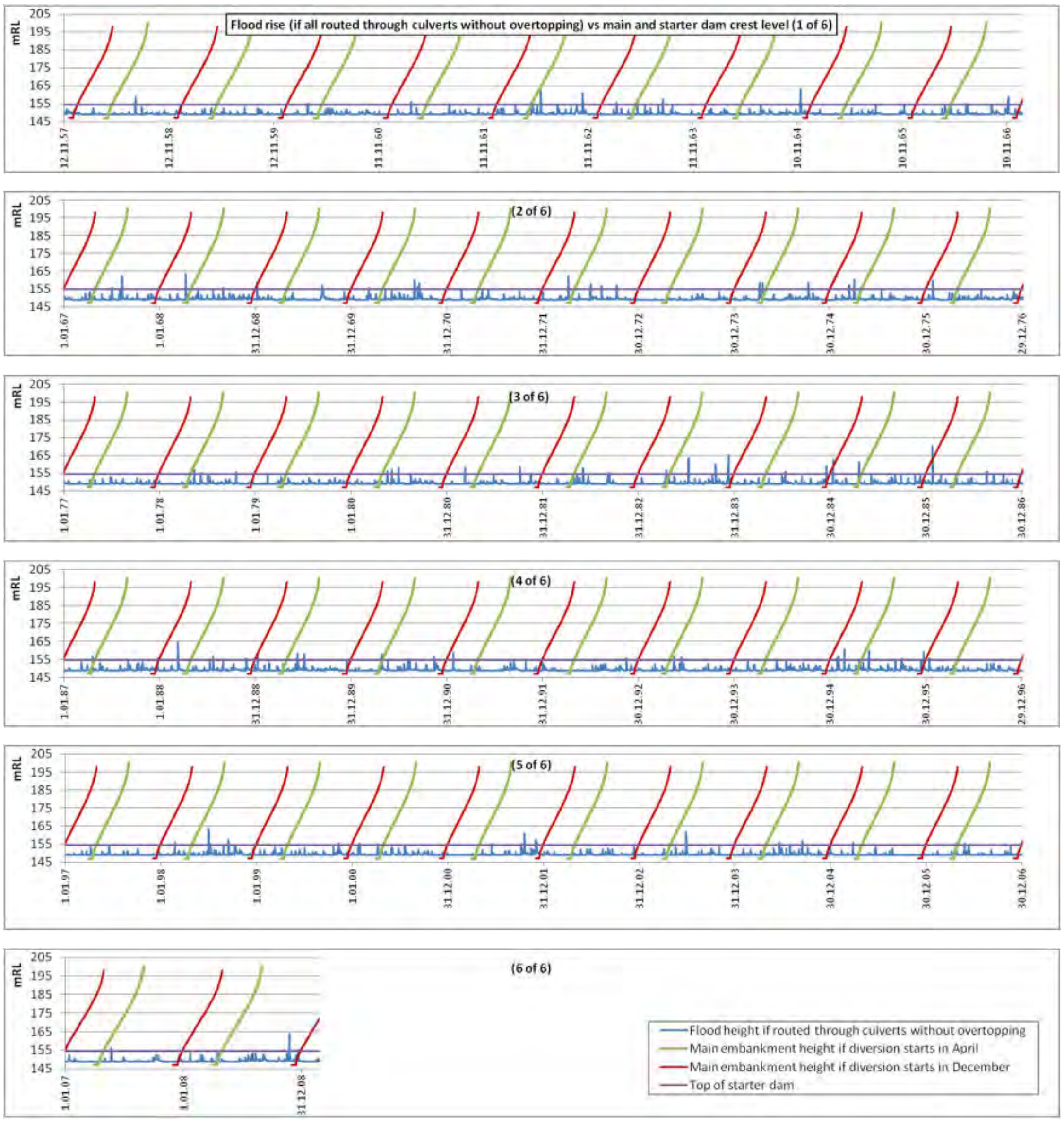


Figure 9.9 Comparison of embankment level and flood level rise during diversion.

Figure 9.10 consolidates the 52 years of inflow flood rises shown on Figure 9.9 into a single year starting on the 1st December e.g. all events occurring on the 8th of December during the 52 years are shown against 8th of December in the figure. Therefore the events shown as blue dots are much more numerous (representing 52 years of data) than could be expected to occur in any one construction year. (Note that the horizontal axis has been limited to the five month construction period, so floods occurring between May and November (inclusive) are not represented.)

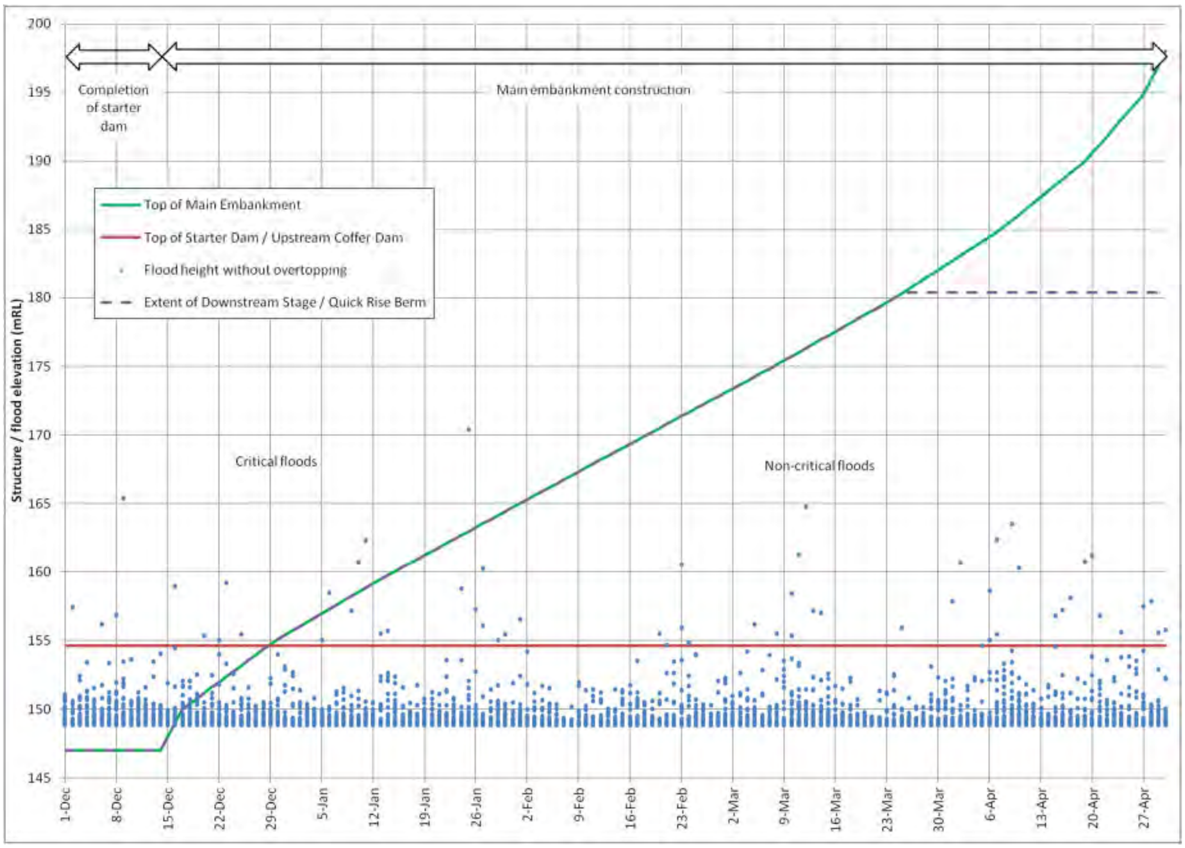


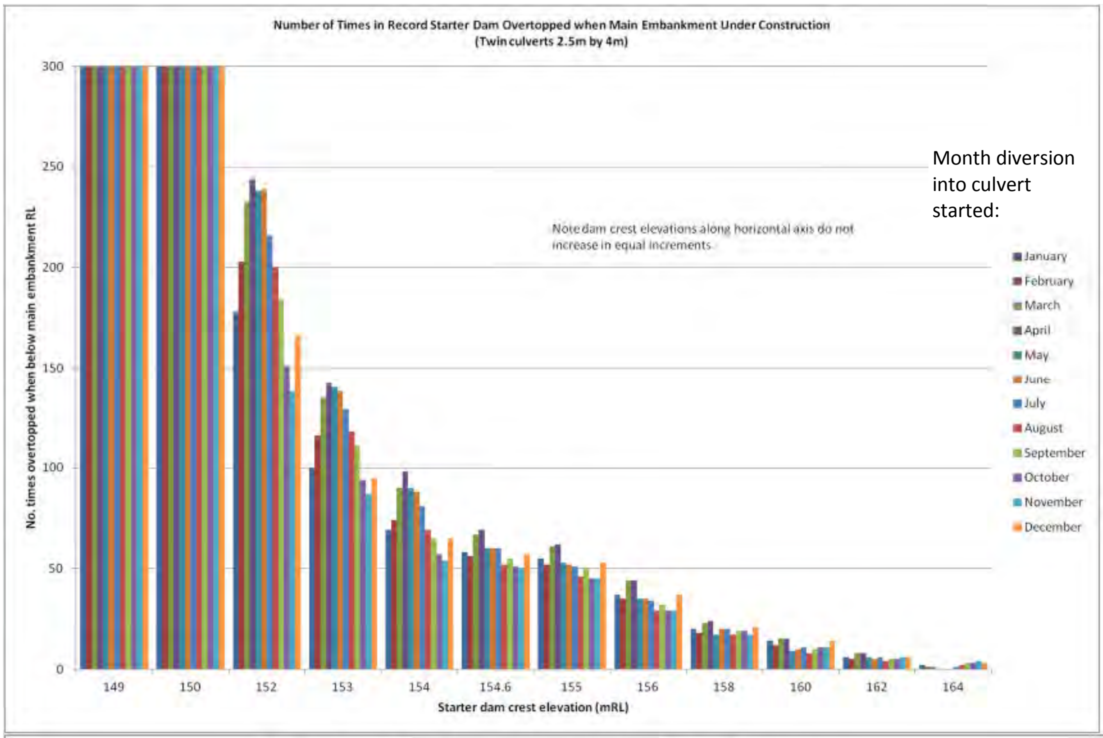
Figure 9.10 Historical flood events within 52 years overtopping the starter dam

Figures 9.11 and 9.12 summarise the variation in number of times the starter dam is overtopped with different starter dam and upstream coffer dam heights and with different months of diversion commencement.

Based on these plots, a starter dam and upstream coffer dam elevation of 154.6 mRL has been adopted, which corresponds to the following:

- The starter dam / upstream coffer dam is overtopped 50 to 69 times (depending on the month diversion starts) during the 52 year flow record over the five month construction period considered. This corresponds to overtopping the starter dam 1.0 to 1.3 times on average for construction starting in a typical year. Some of the time the main embankment is higher than the starter dam, so overtopping only results in inundating the upstream face of the main embankment which will result in negligible damage since the Zone 2B will be protected by the concrete kerb construction
- The starter dam / upstream coffer dam is overtopped 6 to 17 times (depending on the month diversion starts) during the 52 year flow record when still higher than the main embankment e.g. thus inundating main embankment works. This corresponds to overtopping the starter dam and inundating main embankment or starter dam

works 0.12 to 0.33 times on average for a typical year e.g. 12 to 33% probability in any one year.



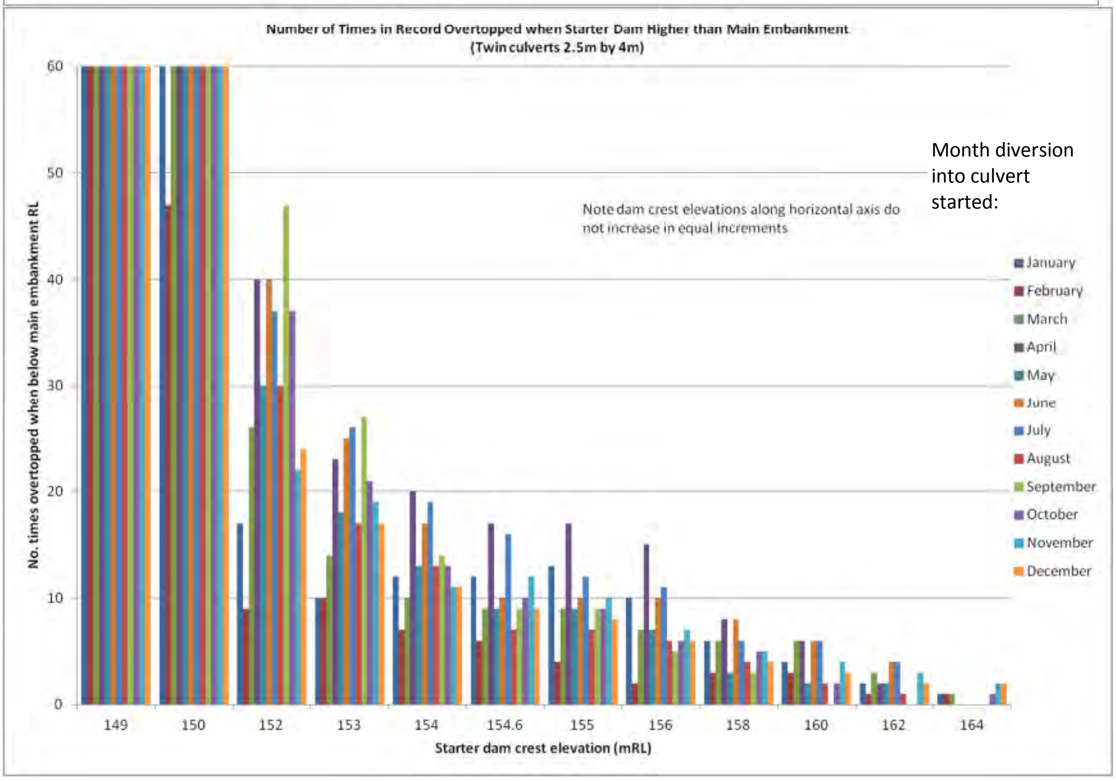


Figure 9.11 and 9.12 Numbers of overtoppings for various starter dam heights

Starter dam stability has been assessed and findings are presented in Section 11.

9.11 Height of diversion wall and upstream coffer dams

The minimum height of the diversion wall and the upstream coffer dam (on the left hand side) is set in Section 9.5 to 154.6 mRL to prevent too frequent overtopping and inundation of works and is also limited in elevation in order to prevent risk to public safety from coffer dam break. However, as discussed in Section 9.4 on dam break analysis, the increase in flood rise downstream for the current coffer dam height is smaller than the thresholds for concern in terms of public safety suggested by guidelines (ANCOLD 2003, Federal Energy Regulatory Commission 1993). Therefore, there may be an opportunity to increase coffer dam height if the contractor identifies advantage in doing so. A small increase in upstream coffer dam makes a significant reduction in the number of flood overtoppings. However, the construction area is very tight and the embankment coffer dam would need to be moved upstream with a longer diversion wall. Further dam break analysis would be necessary to prove that a coffer dam higher than 154.6 mRL will not have implications for public safety.

If the upstream coffer dam on the true right hand side is also set for simplicity to 154.6 mRL then, to prevent overtopping during the mean annual flood (168.5 m^3/s), the minimum width of channel required to the left of the diversion wall is 5.8 m. (This is based on a normal flow depth calculation for a rectangular channel using a Manning's roughness of n=0.033, channel slope of 1V:100H, and channel invert of 148.5 mRL. A width of 8 m has been specified in drawings to recognise the simplistic nature of this calculation method and the uncertainties surrounding temporary work configurations, which will change throughout the construction and are subject to contractor design elements. A channel of this width will require negligible excavation and could be oversized to provide greater security at small cost.

No further optimisation or calculations are intended because relatively small cost is likely to be involved with constructing the upstream coffer dam and channel widening.

9.12 Contractor design elements

Temporary works are typically part of the contractor's scope of work. As such, the diversion strategy contains a significant number of contractor design elements. These elements include:

- Diversion wall
- Debris screening at the culvert intake
- Upstream coffer dam, including review of the crest level with regard to construction nuisance in terms of frequency of overtopping. A preliminary upstream coffer dam and diversion wall height has been specified in this document based on analysis of overtopping frequency as described in the previous sections, and this has been checked to ensure there are no negative implications for public safety. However, if the contractor determines that there is benefit in considering a higher coffer dam, then the contractor will need to assess public safety implications relating to coffer dam break
- Downstream stage reinforcing, including consideration of restricting the width of overtopping over the downstream stage by maintaining a channel with reinforced sides at a slightly lower lift height compared to the remainder of the embankment as rockfill placement progresses. The gabion system should only be considered based on the ALARP principle if cost and programming issues are similar or better than for the bar system

- Quick rise berm to be placed above the downstream stage reinforcing level. This must be designed to ensure it can retain flood water without failure
- Development of a Dam Safety Emergency Plan (DSEP), which details measures to protect the partly completed dam, and measures to warn the population at risk
- Temporary stoplogs for upstream plugging of the conduits.

These contractor design elements should be submitted to the Engineer for approval, especially with regard to implications for public safety.

9.13 Plugging of the diversion conduits

The following general methodology is assumed for sealing the conduits on completion of the main dam and spillway. The actual methodology will be the contractor's:

1. Temporary stoplogs are placed over the left hand conduit. This directs flow into the right hand conduit
2. Works in the left conduit can then commence (penstock construction, valves etc.) When plugging conduits, concrete settlement and grouting between the culvert soffit and top of plug will require close monitoring and suitable quality assurance procedures. Once works in the left conduit are complete; then:
3. Use a temporary 600 dia pipe to direct flow into the left conduit and into the penstock
4. Temporary stoplogs are placed over the left hand conduit. River flow can then be maintained through the 600 diameter pipe
5. Works in the right conduit can then commence. If floods are impounded within the reservoir, then the spillways will start to operate and the intake into the left conduit could be used
6. Once the pipework and intakes are installed then the 600 diameter pipe can be plugged. If desired this pipe could be kept as an emergency low level inlet.

9.14 Optimisation and alternatives

The selection of the heights of the starter dam, upstream coffer dams and downstream stage as described above depends on the selection of a particular culvert size (since flood rise for design events varies with culvert size). In order to demonstrate that the most appropriate combination of culvert size, starter dam and downstream stage has been selected, an indicative assessment of the key components of the diversion strategy has been completed for four different culvert configurations:

- Two barrels, one 2.2 m wide by 3.7 m deep and the second 1 m wide by 1.5 m deep, corresponding to a combined barrel area of 9.64 m^2
- Twin barrels, each 1.75 m wide by 3.5 m deep, corresponding to a combined barrel area of 12.25 m^2
- Twin barrels, each 2.5 m wide by 4 m deep, corresponding to a combined barrel area of 20 m^2 (adopted configuration)
- Twin barrels, each 2.5 m wide by 5 m deep, corresponding to a combined barrel area of 25 m^2.

The selection of height of starter dam, upstream coffer dams and downstream stage for each culvert arrangement has been determined following a similar process as described in earlier sections of this report. This has included the following assumptions:

- The reinforced mesh forming the downstream stage spans the full width of the embankment between abutments and extends up to an elevation where the 1 in 1000 AEP event would be routed entirely through the culvert with no overtopping
- The top of the starter dam is set to avoid too frequent overtopping as determined from considering number of times it would be overtopped based on the 52 year flow record
- The starter dam comprises conventional concrete founded on Class 1 or Class 2 rock.

A tunnel from the left bank of the Lee River discharging into Anslow Creek was identified as an alternative to the culvert. However, this possibility was discarded on the basis that the tunnel would need to be in the order of 300 m long compared to the 165 m long culvert and therefore appeared unlikely to be more economic.

Alternative materials were also considered for construction of the starter dam such as roller compacted concrete (RCC) or rockfill (with a conventional concrete plinth). The rockfill option was discarded on the basis that removing the diversion wall from the rockfill starter dam after it had fulfilled its purpose would be impractical and it would be unacceptable to leave the diversion wall embedded in the rockfill starter dam permanently because of the potential for cracking of the concrete face due to differential settlement.

Similarly, the possibility of using RCC in the starter dam was discarded for the following reasons:

- The working area available for starter dam construction was considered too tight for the widths required by RCC plant and to fit in the two operations required (e.g. bulk RCC placement and placement of grout enriched RCC/conventional concrete on the upstream face)
- Materials testing for RCC mix design can often take in the order of half a year, which would introduce an additional constraint for programming.

9.15 Response to peer review comments

Table 9.3 includes responses to Opus peer review comments.

Table 9.3 Responses to Opus Stage 1 report peer review on diversion strategy

Opus peer review comment	Response
Selection of Construction Design Flood Flood Routing during construction - Tables 6-1 and 6-2 We have difficulty understanding the figures in the “Peak Outflow if Dam Does not Breach” Column. For any crest level of the partially completed dam presuming that the flood water reaches that level then the outflow will be the same for each occasion. The peak outflow will be at the maximum water level reached and thus the flow increases as the level rises.	The peak outflow when the flood water reaches the crest of the partially completed dam will depend on both the conduit area and the flood water level. The conduit area has been sized for each flood event so that the flood water level just reaches the crest of the partially completed dam. Because the inflow hydrograph is different for each flood event, routing effects result in a different conduit area associated with each flood event and crest level, and consequently different peak outflows without breaching.
Section 6 of the report is somewhat confusing and inconclusive. It concludes with the	Thinking regarding this aspect has progressed as presented in this report. Additional dam

statement that "the risk to downstream areas over the period of construction for this diversion option is likely to be an order of magnitude lower than over the nominal service life of the dam". This statement requires community acceptance of the duration time consequence of the hazard exposure to be a simple function; some further debate as to the validity of this approach is needed when comparing risks during construction with risks during the service life of a structure.	break analyses have been carried out to demonstrate there is no increased hazard to public (regardless of duration of exposure) from unreinforced coffer dams (such as the upstream coffer dam with crest at 154.6 mRL) as presented in Section 9.4. Although, the main coffer dam or "downstream stage" will be high enough to cause increased hazard to the public if breached, the intention is to reinforce the downstream stage such that it can withstand overtopping without breaching. As described in Section 9.6, the reinforcing will continue up to a level at which a flood event between a 1 in 100 AEP and 1 in 200 AEP event can be routed entirely through the culverts without overtopping. A "quick rise berm", which can be constructed within several days (and timed to match favourable weather forecasts), will then be used to raise the overall embankment to a level at which a 1 in 1000 AEP event can be routed entirely through the culverts without overtopping.
It is not clear what the selection criteria are for the diversion culvert size. Nor is it explicitly stated what the recommended diversion culvert size is although a sample risk profile is given for a diversion culvert with a cross-sectional area of 24m^2. For this culvert size, what size (magnitude and frequency) floods would just be able to be passed without the partially constructed embankment dam being overtopped at the three intermediate construction levels – 165m, 175m and 185m? What would be the incremental downstream Population at Risk if the partially constructed dam was to be overtopped and fail?	Twin culverts, each 2.5 m wide by 4 m high, with a combined cross-sectional area of 20 m^2 have been adopted based on a cost optimisation process (contingent on specific safety and design standards) as described in Section 9.13. A 1 in 1000 AEP flood has been selected as the construction design flood for the partially constructed embankment, which is consistent with NSW Dams Safety Committee (Demonstration of Safety for Dams – DSC2D Section 6.17) advice that they will accept a flood capacity, during those phases of construction with public safety at risk, in the range of the AEP 1 in 500 to 1 in 1,000 flood discharge on the basis of world practice provided the risks are as low as reasonably practicable. The flood size that is just able to be passed through the culverts without overtopping at the three intermediate construction levels mentioned is of less relevance since reinforced rockfill that can withstand overtopping has been adopted. The partially completed embankment is designed to withstand the 1 in 1000 AEP flood at the three intermediate construction levels without breaching either by allowing flows to overtop reinforced sections of the embankment (within ICOLD 1993 guidelines for safe overtopping depths for reinforced rockfill) or by passing flow through the culverts with no overtopping. No evaluation of the incremental PAR is intended for the partially constructed dam. The evaluation may be of interest but would

	not change the design criteria or standards, and therefore serves little purpose.
Further explanation would be helpful to our understanding on how the diversion is to be plugged and the inlet/outlet flow regulation managed (ref dwg 500)	Refer to Section 9.13.

10 Culvert (conduit) structural design

Estimates of the vertical and horizontal loads applied to the diversion culvert from the rockfill embankment have been estimated in two models implemented in the finite difference package FLAC/2D. The two models represent a long section along the culvert alignment, and a cross section along the dam crest.

The models have been used to assess the static loads that might be imposed on the diversion culvert by the embankment and the seismic deformations that might occur in the dam embankment that could lead to racking of the culvert box.

10.1 Geotechnical analysis

10.1.1 Static rockfill loads

For static models, the detailed stress related rockfill parameters were simplified to allow the models to be implemented with a mohr-coulomb elasto-plastic model. The rockfill was described by the following parameters. Sensitivity studies were carried out to assess the range of values listed below.

- Strength - ϕ=38 to 45 degrees, c=0kPa
- Stiffness - E=13 - 40MPa
- Density - γ=2250 - 2500 kg/m^3.

From these models, absolute loads have been extracted from the model representing the section parallel to the dam crest. The variation of these loads along the culvert alignment (with changes in embankment cover and applied load from the reservoir) has been assessed in the culvert parallel model.

10.1.2 Seismic deformation

Temporary embankment deformations associated with ground shaking may induce additional loads in embedded structures such as the diversion culvert. A critical case for the culvert would be horizontal earthquake motion parallel to the dam crest, potentially resulting in horizontal racking of the box culvert structure.

For the analysis the stiffness parameters controlling the behaviour of the rockfill represent estimates of the small strain stiffness. The small strain stiffness parameters have been estimated using the method of Makdisi and Seed (1984) and a $k_2$max of 120. The use of this value for a compacted rockfill embankment is supported by Lai (1985) and Romo (1980). This yields a maximum small strain E of 200MPa. The maximum small strain stiffness has been degraded to account for the estimated strain level within the embankment based on the degradation curve presented by Makdisi and Seed (1984).

Pseudo static analysis has been used to assess the potential embankment displacements at the location of the top and bottom of the culvert box (a 6m high structure).

Two cases assume peak ground accelerations for the OBE and MDE events consistent with Table 10.1. The horizontal acceleration has been assumed to act parallel to the dam crest (the worst case scenario for culvert racking). Owing to the asymmetry of the cross section and the off centre location of the culvert, analyses consider horizontal acceleration applied from both directions.

Table 10.1 summarises the results obtained in terms of racking. These results have been adopted for the structural design of the culvert, described in Section 10.2.

Table 10.1 - Estimated racking of 6m high box culvert under the highest embankment

Seismic load case	Maximum estimated horizontal displacement at culvert base (mm)	Maximum estimated horizontal displacement at culvert top (mm)	Maximum estimated racking (top of culvert relative to base) (mm)
OBE (0.16g)	2	4	2
MDE (0.48g)	6	26	20

10.2 Structural analysis and design

The results of the FLAC analyses described in Sections 10.1.1 and 10.1.2 have been used as inputs into a linear elastic model using the software package Microstran V9.0. The model analysed is a simple 'stick' model with the following key assumptions:

- The concrete walls and slabs have been analysed using estimates of their cracked section properties. The cracked section properties (Table 10.2) have been estimated using the guidance in the commentary of NZS3101
- The conduit base slab supports have been modelled as a series of springs. The spring stiffness's have been derived using a subgrade reaction modulus of rock of 240 MN/m^3 (derived from Class 1 rock). Sensitivity analyses have also been carried out if the conduit is founded on Class 2 rock. The Class 1 rock analysis results in conservative forces and moments
- The wall/slab joints (joint block regions) have been modelled using rigid off-sets
- The concrete structure has been designed with a nominal ductility ($\mu = 1.25$)
- Maximum bending moments at corners have been re-distributed (reduced) by up to the code allowance of 30%. The mid-span bending moments have been increased by an equal amount. Shear forces have not been re-distributed because they are a brittle failure mode
- Hydrostatic water pressures have been added to the static rockfill loads to represent reservoir pressures
- All wall/slab joints are modelled and designed to be continuous (i.e. carry moment)
- For the seismic design case the maximum estimated rack displacement has been applied to the Microstran model as a horizontal displacement at the top of the conduit.

The resulting bending moments are similar to those derived during the feasibility studies (T&T 2009).

An envelope for un-redistributed bending moments is shown in Figure 10.1.

Steel reinforcing has been determined using simple spreadsheet based calculations at the ultimate limit state. Seismic combinations have been designed using over strength factors (refer to Table 10.2 for a list of design parameters). Sample calculations have been checked using the design software package spColumn v4.60. The derived longitudinal reinforcing requirements have been confirmed using both approaches.

The concrete roof and slab elements (for the full height embankment section) fall within the category of "deep beams" as defined by NZS3101. A simple strut and tie truss analogy has been used to review shear and longitudinal steel requirements for these deep beam

sections. The reinforcing required has been adjusted to take the worst case of the two methods.

The conduit has not been designed as a water retaining structure i.e. crack widths have not been assessed for criteria in NZS3106. This is because under normal operating conditions the conduit will not be retaining water - because it is behind the concrete face.

Joint spacing and final detailing will be established during Stage 4 design. The steel penstock and pedestal design will also be completed during Stage 4.

Table 10.2 Concrete design properties

Description	**Adopted property**
Unconfined compressive strength (28 days) f'c	40 MPa
Longitudinal reinforcing yield strength f_y	500 MPa
Concrete cover	40 mm internal, 50 mm external (assumes shuttered formwork or the use of site concrete)
Concrete ductility (nominal) μ	1.25
Modelled wall stiffness	$I_e = 0.25I_g$
Modelled slab stiffness	$I_e = 0.4I_g$
Seismic Overstrength design	
Unconfined compressive strength (28 days)	f'c + 15 MPa (55 MPa)
Longitudinal reinforcing yield strength fy	1.35fy (675 MPa)
Strength reduction factor (ϕ)	0.75 Shear 0.85 Bending

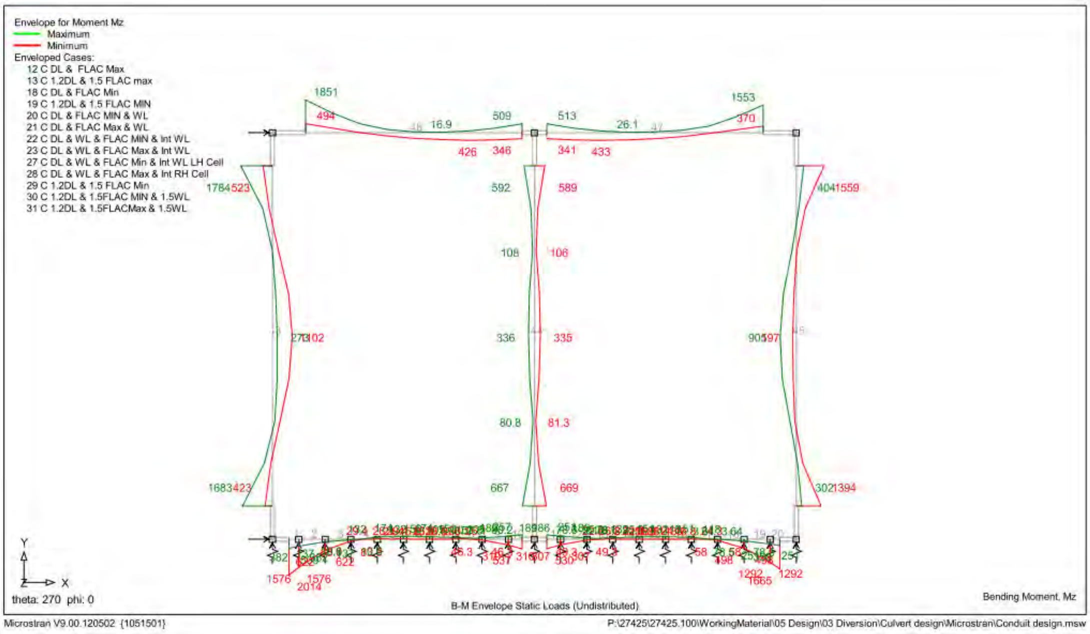


Figure 10.1 Bending moment envelope of all static cases analysed (undistributed) for the maximum height embankment

10.2.1 Response to peer review comments

Table 10.3 includes responses to Opus peer review comments.

Table 10.3 Responses to Opus Stage 1 report peer review on conduit design

Opus peer review comment	Response
Conduit velocities are critical and there needs to be quantitative consideration of erosion / abrasion factors and possible treatments regarding invert and wall damage.	The maximum calculated velocity in the conduits during construction is 16.6 m/s for a 1 in 1000 year return period flood. By comparison, the maximum calculated velocity in the conduits during diversion is 7.8 m/s for the Mean Annual Flood. We have assumed that the contractor will construct temporary screening to the conduit to limit debris and large boulders from entering the conduits. We do not consider it practical to assess the degree of abrasion of the conduit. Instead it will be a contract condition for the contractor to repair any abrasion damage to the conduit prior to commissioning the dam. From a design perspective the concrete strength selected is 40MPa which is expected to better resist abrasion than lower strength concretes.
An indication of the structural design detail for the conduit is yet to be provided, especially the nature of any jointing.	Refer to section10 for design of the conduit. Jointing will be designed during Stage 4.
The nature of the structural interaction between the outlet risers and other elements	The outlet risers will be cast into a concrete thrust block. The design of the outlet risers

and the sealing details needs to be provided, particularly under conditions where settlement or other deformation may occur.	themselves allows for the estimated settlement of the dam during filling.
We are not clear as to the mode of operation of the gated inlet and the means of maintaining the functionality of this feature.	The gated inlet has been deleted in favour of a penstock controlled by valves at the downstream end of the conduit. Further detail can be found in the PB M&E report.
The need for and nature of venting the conduit under normal tail water conditions is unclear to us from the information shown on dwg 500.	Because the penstocks are controlled at the downstream end of the conduits, no ventilation is required. In the unlikely event that the operator of the dam requires to access the upstream portion of the conduit, then breathing apparatus may be required.

11 Starter dam design

The starter dam is assumed to be founded on Class 1 rock. Geotechnical conditions have been adopted from the recommended values presented in Appendix F for Class 1 rock. Partial factors of safety of 1.5 for the frictional component and 3 for the cohesion components were applied. The sliding analysis was also checked assuming no cohesion (i.e. friction only). A factor of safety of greater than 1 was calculated for this conservative situation.

Table 11.1 presents a summary of the design of the starter dam showing that it meets the requirements of NZSOLD guidelines (NZSOLD 2000) extract shown in Figure 11.1.

Other key assumptions made in the design are:

- Uplift water pressures have been assumed to be equal to reservoir water level at the upstream end, reducing linearly to zero at the downstream end
- The concrete face does not provide any passive resistance to sliding or overturning because it would require excessive movement and therefore possible damage to the face slab or joint
- The vertical gravity load acting on the starter dam is equal to self weight plus the vertical weight of rockfill acting over the downstream sloping face.

Table 11.1 Starter dam design summary

Description	Calculation
Class 1 rock/concrete friction angle	49 degrees
Class 1 rock/concrete cohesion	500 KPa
Overturning (PMF)	FoS 1.45
Overturning (PMF + OBE)	FoS 1.40
Overturning (NTWL + MDE)	FoS 1.28
Sliding (PMF)	FoS 3.41
Sliding (PMF +OBE)	FoS 2.88
Sliding (NTWL +MDE)	FoS 2.44

11.1 Shrinkage control

The reinforcing requirements for the starter dam have been considered for two situations:

- Thermal stresses during construction; and
- Long term shrinkage requirements.

11.1.1 Thermal stress requirements during construction

For thermal stresses the method described in BS8007 has been adopted. This approach considers the direct tensile strength of the concrete and the required reinforcing steel to resist the thermal expansion of the concrete during the curing process.

Crack widths of less than 0.1mm are calculated for the starter dam. These are considered to be acceptable. The following assumptions have been made in deriving this crack width:

- A coefficient of thermal expansion of 10.5×10^{-6} per degree Celsius

- A cement content of 350 kg/m^3
- A maximum lift height of 1m has been assumed. This will be a requirement for the Contractor to establish
- 665 reinforcing mesh at each lift height.

11.1.2 Long term shrinkage requirements

In accordance with NZS3101 cl 8.8.2 an allowance of 1000 mm^2/m (HD16-200 each way) on all surfaces of the starter dam has been allowed. This is appropriate because the starter dam design is not controlled by stress considerations.

Control joints

The following additional measures have been undertaken to control shrinkage in the starter dam in accordance with USACE EM 110-2-2200 Chapter 7 (1995):

- Control joints are specified at a maximum of 10 m spacing
- The upstream face of the control joints have a double row of waterstops
- The reinforcing is curtailed at the control joints.

Table B.6.1.
Factors of Safety, Static Assessment.[a]

Loading conditions	Minimum factor of safety	Slope
Steady state seepage with maximum storage pool	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3[b]	Upstream
End of construction before reservoir filling	1.3	downstream and upstream

(a) The factor of safety is that factor required to reduce the mobilized shear strength parameters in order to bring a potential sliding mass into a state of limiting equilibrium, using generally accepted methods of analysis.

(b) Higher factors of safety may be required if drawdown occurs relatively frequently during normal operation

For concrete dams and their foundations, sliding resistance is important to withstand the load combinations that could occur. After extreme loads the dam and foundation must have sufficient stability to safely retain the reservoir. Typical minimum factors of safety for concrete gravity dams are indicated in Table B.6.2.

Table B.6.2.
Commonly Accepted Values for Strength and Sliding Factors for Gravity and Buttress Dams.

TYPE OF ANALYSIS[a] [f]	LOAD CASE			
	USUAL	UNUSUAL (post earthquake)	EARTHQUAKE (MDE) [b]	FLOOD (IDF)
Peak Sliding Factor (PSF) No tests	3.0	2.0	1.3	2.0
Peak Sliding Factor (PSF) With tests [c]	2.0	1.5	1.1	1.5
Residual Sliding Factor (RSF)[d] [e]	1.5	1.1	1.0	1.3
Concrete Strength Factor[g]	3.0	1.5	1.1	2.0

Figure 11.1 NZSOLD guidelines recommended factors of safety for sliding of concrete gravity dams

11.2 Response to peer review comments

Table 11.2 includes responses to Opus peer review comments.

Table 11.2 Responses to Opus Stage 1 report peer review on starter dam design

Opus peer review comment	Response
It is noted that the starter dam is preliminary at this stage and will be detailed in a subsequent design stage. This is potentially an important element in the design of several features of the project, and its scope needs to be determined to progress the other elements.	The starter dam design is now complete.

12 Embankment stability

12.1 Embankment stability under flow-through

The completed embankment may be subject to varying degrees of flow through during its life. The embankment remains adequately stable in these instances.

Situations where significant flow-through could occur are identified as:

- During construction when the embankment is complete but prior to construction of the concrete face slab and in the event a significant flood occurs that results in impoundment during routing
- During operation, following a significant earthquake. The concrete face slab may crack and leak, resulting in flow-through.

The potential impact of flow-through has been estimated by seepage modelling, and subsequent stability modelling. This section describes how the magnitude of flow through has been estimated and what the impacts could be to embankment stability.

12.2 Embankment permeability characteristics

The placement of rockfill in the embankment during construction typically results in segregation of coarse and fine particles in each layer (Janson 1981) such that horizontal permeability is higher than vertical permeability. Fell et al (2005) recommends that all embankments should be designed on the assumption that the ratio of horizontal permeability to vertical permeability is 15 or higher. For large or more sensitive dams, Fell recommends that they are designed such that embankment stability is not sensitive to the ratio.

For this assessment, the main embankment zones have been modelled with a ratio of 100. A summary of the adopted permeability characteristics is presented in Table 12.1.

Table 12.1 - Adopted permeability characteristics

Zone	Permeability	Anisotropy (kv/kv)
2A	$1x10^{-2}$ m/s	100
2B	$1x10^{-4}$ to $5x10^{-3}$ m/s	100
3A, 3B, 3C, 3D	$1x10^{-2}$ m/s	100
Class 1	$1x10^{-7}$ m/s	1
Class 2	$1x10^{-7}$ m/s	1

12.3 Embankment flow-through estimates

Estimates of potential flow, and of the exit level of seepage on the downstream face have been made using the software package SEEP/W. Two cases have been considered, which represent the zone immediately under the concrete slab differently. In both cases the reservoir is assumed to be impounded to NTWL (197.2 mRL).

12.3.1 Construction case - case 1

Case 1 (Table 12.2) represents the construction flow-through case. Zone 2B (the zone the concrete slab is constructed on) is assumed to have a lower permeability than the main embankment zones (this is the intent of the design). However it is assumed the concrete face and any associated kerbing has not yet been constructed.

Table 12.2 Construction case embankment flow-through

Case	Estimated flow-through	Estimated seepage exit RL
Construction case	20 l/s per m width	175 mRL

12.3.2 Post earthquake case - case 2

Case 2 (Table 12.3) represents the post earthquake case. Zone 2B is conservatively assumed to be cracked or disturbed such that there is no permeability contrast between it and the main embankment zones. This is considered to be a very conservative model as it ignores any flow restricting capability from the cracked concrete slab, and assumes significantly disturbed material in Zone 2B. This case is therefore not considered realistic in terms of estimates of flow-through magnitude and has primarily been used as an extreme upper bound for embankment stability assessments.

Table 12.3 Construction case embankment flow-through

Case	Estimated flow-through	Estimated seepage exit RL
Post earthquake case	100 l/s per m width	195 mRL

12.4 Embankment stability results

The global stability of the downstream embankment face during flow-through has been estimated using the software package SLOPE/W and the seepage models described above. The stability models were set up consistent with those described in Section 13. A separate assessment of the potential for seepage to result in unravelling of the downstream face is described in Section 12.5.

The most onerous stability case is the post earthquake case, where embankment seepage is assessed (for the combination of assumptions adopted) to exit the downstream face at 195 mRL. Two stability cases have been considered.

12.4.1 Static post earthquake - case 1

This case (Table 12.4) assesses the static stability of the embankment under the full effect of flow through, prior to any intervention to lower the impounded reservoir.

Table 12.4 Static stability post earthquake case 1

Case	Stability criteria	Estimated F.o.S
Post earthquake case	F.o.S >1.2	1.29

The post earthquake static stability is considered to be satisfactory based on this assessment.

12.4.2 Aftershock stability post earthquake

This case (Table 12.5) assesses the capacity of the embankment under full-flow through to accommodate the ground motions potentially associated with an earthquake aftershock. GNS (pers. comm. with Graham McVerry) suggest that following an MDE event, an aftershock might be expected at one magnitude less than the main shock (in this case a 6.5Mw down from 7.5Mw). In absence of an aftershock spectra, the PGA for the aftershock has been assumed here (very conservatively) to be equivalent to that for the main shock.

The methodology described in Section 13 has been followed in assessing embankment displacement associated with this case.

Table 12.5 Aftershock stability post earthquake case 2

Case	Yield acceleration	Estimated displacement
Post earthquake, aftershock with full flow through	0.13 g	650mm

12.5 Earthquake induced deformations

ICOLD (2010) states that CFRD dams have performed well during large earthquakes. The performance review presented by Cruz et al (2010) conclude that for CFRDs between 50 m and 100 m high only minor fissures or cracks have occurred on the face and these have easily been repaired. Cruz et al (2010) report that there are in excess of 400 CFRDs in existence over approximately 30 m in height, however Swaisgood (2003) lists only seven with measured earthquake induced deformations and ICOLD Bulletin 141 (2010) presents four.

The calculated Earthquake Severity Index (ESI) for the Lee Valley Dam OBE and MDE cases, are 6 and 17 respectively. These are based on a magnitude 7.8 earthquake for both the OBE and MDE events as presented in the GNS report (2011). Figure 12-1 presents the calculated ESIs compared with performance data presented by ICOLD (2010), Swaisgood (2003) and Bureau et al (1985). Swaisgood's published empirical equation was used to estimate the amount of deformation to be expected as a result of the OBE and MDE events. These expected deformations are as follows:

- OBE relative settlement of 0.08%, or < 50 mm
- MDE relative settlement of 0.53%, or < 300 mm

It is worth noting that the deaggregation of the Lee hazard shows nearly 80% of the contribution to the OBE and MDE hazard comes from events of lower magnitude than the 7.8 for which the above ESIs are calculated. The mean magnitude of the contributions to the PGA hazard ranges from about 6.3 to 6.5 for return periods from 150 years to 10,000 years (GNS, 2011). If the average magnitude presented by GNS is used to calculate the OBE ESI, the value reduces down to approximately 1.

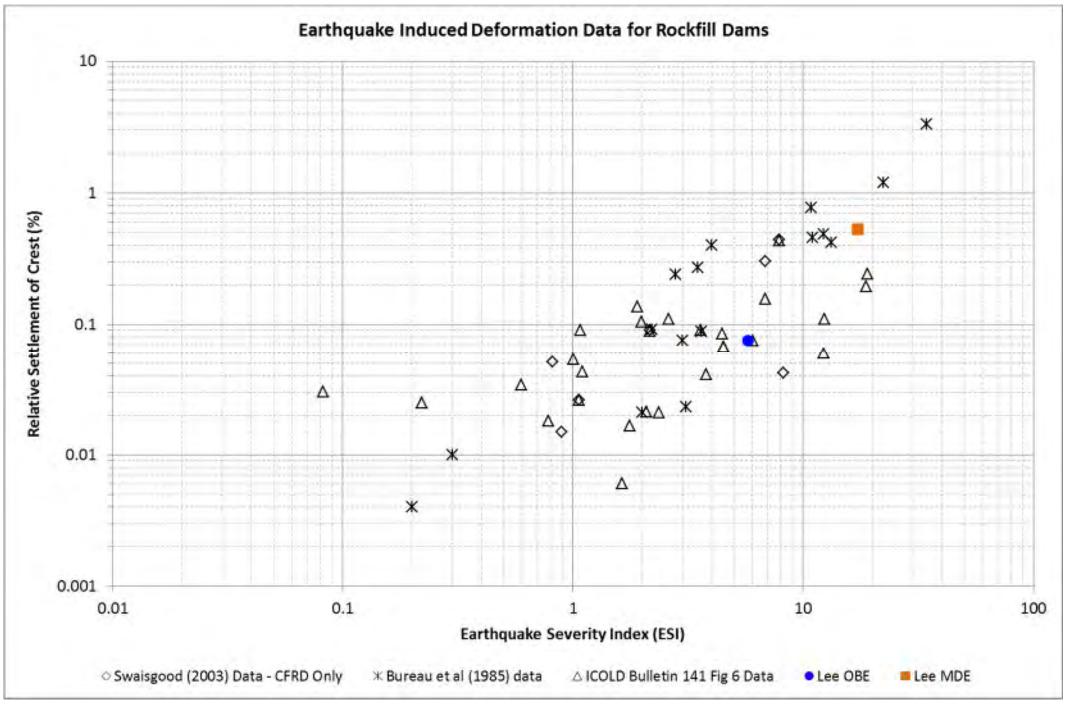


Figure 12-1Earthquake induced deformations of rockfill dams

Bureau et al (1985) report that the 67 m tall Minase CFRD Dam in Japan, settled around 60mm in the 1964 magnitude 7.5 Niigata earthquake. This is of a similar order to the settlement expected for the Lee Valley Dam due to an OBE event. Minase Dam reportedly suffered only minor joint damage and leakage from the dam increased for a period of a few days before returning to normal. The dam has been shaken by several other earthquakes since, but no other damage is reported. Minase dam was constructed from dumped and sluiced rock fill, so we would expect better performance with a modern compacted rock fill dam such as that proposed for the Lee Valley Dam.

New Zealand Dam Safety Guidelines (NZSOLD, 2000) allow for some minor repairable damage at OBE level shaking. Cruz et al (2010) recommend the OBE design criteria as damage that can be repaired whilst the dam is still operational. Thus, for Lee we have not endeavoured to eliminate the risk of minor damage at OBE level shaking. Based on the published information available, the expected Lee OBE and MDE deformations reported above are considered to be within tolerable limits for a CFRD dam.

The concrete face joints have been developed by precedence not by specific design, as is usual for CFRDs. We are not aware of any designers successfully designing (by numerical analysis) the joints for seismically induced movements. However, appropriate detailing of the joints is undertaken to provide some ability to move. Furthermore, the movements expected to occur during first filling of the reservoir are likely to be greater than those caused by an OBE event. These details have been tested in service on other CFRDs and we therefore consider them appropriate for the Lee Valley Dam.

12.6 Unravelling of the downstream face

The potential for seepage to cause unravelling of the downstream face has been investigated using methods developed by Olivier (1967), Stephenson (1979) and Solvik & Skoglund (1995). The construction case described in Section 10.3.1 was used as the worst case scenario. With an unprotected rockfill face unravelling is estimated to occur between 166.8 and 173.3 mRL. This is below the proposed height of meshing of 173.4 mRL (refer Section 9.0) which will constrain the rockfill.

12.7 Stability of the quick-rise berm under flow-through

The unravelling of the quick-rise berm under flow-through (as described in Section 9.0) was also investigated using the methods stated in Section 12.5. Using the hydraulic conductivity value of $k = 1x10^{-2}$ m/s a D_{50} of 90mm is required to ensure stability of the material used on the downstream face of the berm. Therefore the quickrise berm material will be specified such that the downstream half has a D_{50} greater than 90 mm.

12.8 Peer review response

Table 12.6 includes responses to Opus peer review comments.

Table 12.6 Responses to Opus peer review on embankment stability

Opus peer review comment	Response
Loading combinations. The listing of load combinations presented in Section 4.2 appears to be incomplete. For example, given the stated intention in Section 4.13 to design the embankment to remain stable without the concrete facing being intact, the loading cases will need to include for various seepage conditions as identified elsewhere in the report, and the associated degree of stability / deformation that may be considered acceptable. Furthermore the manner of including transient hydraulic loadings onto the structural elements is not identified, nor the nature or degree of how of thermally induced stresses are to be considered. I support the intended use of Makdisi-Seed simplified embankment response analysis in this case, and suggest that the load combinations need to be extended to clarify the seepage conditions to which this analysis is to be applied. This rational analysis approach addressed in Section 4.13 is supported, but no associated specific performance targets / criteria have been defined.	The embankment stability for the case where the concrete facing is absent or highly damaged is addressed in Section 12.4. Stability criteria relating to this case are provided in Section 12.4.1. Makdisi and Seed displacement estimates are provided in Section 13, and displacement estimates discussed in Section 12.3.
Seepage rates: Section 4.5.4 presents several target seepage / drainage capacity parameters, but does not clarify where a facing “failure” involving say damage or deterioration of the perimetric joint may fit within this listing, as it	Seepage rates assessed for large scale facing failure cases are provided in Section 12.3.

is inferred that this information may relate primarily to seepage paths that bypass the facing/plinth system.	
Constructed and Natural Slopes: Sections 4.15 and 4.16 address management and analysis methods to be applied to slope instability risk, but do not present the target performance or design criteria as such. The adoption of a systematic risk based approach is supported, but the threshold levels of acceptable and tolerable risk will need to be established.	Refer to Appendix F and Section 6.3 for the assessment of natural slopes.

13 Seismic performance of the dam embankment

Earthquake ground motion may result in permanent deformation of the embankment, with the deformation magnitude depending on the size of the earthquake (the ground acceleration and magnitude). This section describes how potential for earthquake induced deformation of the dam embankment has been estimated, and what, if any, the potential implications of the deformations might be.

13.1 Seismic response

During an earthquake, fill embankments typically amplify the horizontal earthquake ground motions such that at the crest, accelerations can be significantly higher than at the base.

Makdisi and Seed (1978) present a method for estimating this amplification, and the results of the method for the embankment are presented in Table 13.1 for the OBE and MDE events.

In the method, the following inputs have been adopted:

- Embankment 54 m high, with a 6 m wide crest (marginally higher than the final proposed height and therefore slightly conservative)
- Embankment side slopes 1 vertical to 1.5 horizontal
- Maximum small strain shear modulus (G_{max}) of the embankment of 470 MPa

Table 13.1 - Embankment crest accelerations

Description	150 year ARI (OBE)	5000 year ARI (MDE)
Peak ground acceleration[1]	0.16g	0.48g
Crest acceleration from Makdisi and Seed (1978)	0.64g	1.69g

1 - From GNS (2011)

13.2 Seismic deformation

Estimates of the potential for permanent deformations to develop in the embankment in response to earthquake shaking has been undertaken based on the correlations given in Jibson (2007), adopting the accelerations identified in Table 13.1. The procedure is based on simple limit equilibrium analysis methods and is typically carried out to assess whether significant deformations may be sustained by an embankment, and whether more detailed analyses are required to assess them.

The method estimates the magnitude of post earthquake deformation based on the embankment “yield acceleration”. The yield acceleration is that acceleration that is just large enough to result in development of small permanent deformations within the embankment. When actual accelerations exceed the yield acceleration, deformations increase with increasing acceleration. This Jibson correlations are one of several based on ‘Newmark sliding block’ type analysis of earthquake records that are commonly used to evaluate displacements due to earthquake shaking.

The embankment yield accelerations have been estimated using a limit equilibrium slope methodology implemented in stability program Slope/W. In doing this, it is implied that in an earthquake, slips will form that are discrete masses of soil that move in isolation from material below on a slip surface (a “slip”). In reality, the material deforms in a wide zone,

and hence these analyses provide an estimate of maximum displacement along the theoretical slip plane, not an estimate of the distribution of displacements within the embankment. An earthquake magnitude of 7.5 has been adopted in the estimates based on GNS (2011)

Potential slip surfaces (Figure 13.1) have been assessed at various depths within the embankment, in order to compare the yield acceleration for the particular assumed slip geometry with the maximum average embankment acceleration (which itself varies with depth). Only those theoretical slip surfaces which encompass the dam crest have been included in the analyses, and material strength parameters representing dry material have been adopted (therefore representing the dam in its completed state). A fully specified stress dependant strength function has been adopted for these analyses, consistent with the properties in Appendix F, T&T (2012).

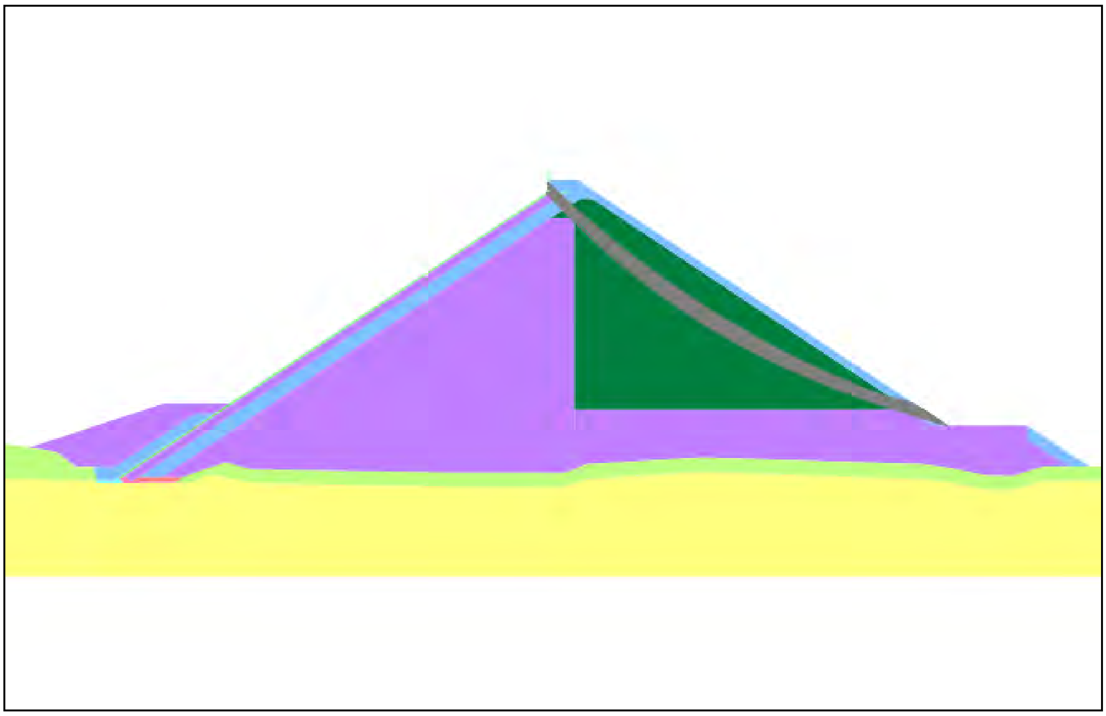


Figure 13.1- Dam embankment showing example failure surfaces (grey lines on downstream face) considered in seismic displacement assessments. Note that only failure surfaces that encompass the dam crest (and hence relate to deformation with the potential to compromise water retention) have been considered

From the analyses, the combination resulting in the highest estimated deformation is reported in Table 13.2.

Table 13.2 - Permanent deformation estimates

	150 year ARI (OBE)	5,000 year ARI (MDE)
Estimated displacement[1]	20mm	400mm

1 - The displacement relating to the "mean plus one standard deviation" displacement Jibson correlation is reported

13.2.1 Sensitivity to other motions and potential conditions

The deformation estimates provided in Table 13.2 assume dry embankment materials and consider the effect of horizontal accelerations associated with earthquake motion. Sensitivity studies have been carried out to assess the potentially larger deformations that might arise if the materials are wet, or if there is a significant vertical component to the earthquake motion, concurrent with the horizontal motion.

13.2.1.1 Permanent deformation including wet embankment material

The potential for wet embankment materials primarily arises in situations where:

i. The incomplete dam (embankment complete but concrete facing is yet to be constructed) experiences a flood event such that the reservoir impounds
ii. The concrete facing is damaged (such as following a seismic event) and the reservoir is full.

For item (i) the probability that a significant earthquake occurs concurrent with the flood event is very low, and is therefore not considered further. For item (ii) it is assumed that damage to the concrete facing would occur during the earthquake, such that initially the materials are dry. Once the facing is damaged, the potential for flow through to occur, and for material wetting is realised. The wet materials are then likely to experience earthquake aftershock motions. Deformation estimates have been made for this potential combination of events and conditions.

As a conservative representation of an aftershock, the peak ground acceleration has been assumed to be the same as experienced in the initial earthquake, with the aftershock magnitude one order of magnitude less (from 7.5 to 6.5).

Table 13.3 - Permanent displacement estimates for aftershock considering flow through

	150 year ARI "aftershock"	**5,000 year ARI "aftershock"**
Estimated displacement[1]	175mm	650mm

1 - The displacement relating to the "mean plus one standard deviation" displacement Jibson correlation is reported

13.2.1.2 Permanent deformation including vertical earthquake motion

The potential effect of vertical earthquake motion concurrent with horizontal earthquake motion has been assessed. In order to include the effects of vertical accelerations, the limit equilibrium analysis described above has been reanalysed, but with vertical accelerations included in the analysis. The effect of including a vertical acceleration on the resulting horizontal yield acceleration can therefore be assessed.

Guidance on the potential combination of vertical and horizontal motion in an earthquake is provided in NZS1170.5 (2004) where it is recommended a maximum vertical acceleration of 21% of maximum horizontal acceleration is assumed to be coincident with the maximum horizontal acceleration. This level of vertical acceleration has therefore been included in the analysis. Additionally a sensitivity analysis has been undertaken where a vertical acceleration of 50% of the horizontal acceleration has been included.

Displacements have been estimated based on these yield accelerations as described above. The results are shown in Table 13.4.

Table 13.4 - Permanent deformation estimates

Description	150 year ARI (OBE)	5,000 year ARI (MDE)
Estimated displacement not including vertical earthquake acceleration	20 mm	400 mm
Estimated displacement including vertical earthquake acceleration at 21% of horizontal acceleration[1]	25 mm	440 mm
Estimated displacement including vertical earthquake acceleration at 50% of horizontal acceleration[1]	35 mm	510 mm

1 - The displacement relating to the “mean plus one standard deviation” displacement Jibson correlation is reported

13.3 Discussion and conclusions

The permanent deformations estimated are of an order that would not be expected to compromise the dam function at the OBE level. The displacements estimated (20 to 35 mm maximum) would be expected to be accommodated by the dam structure and result in little significant damage.

At the MDE level event, the permanent deformations estimated (400 to 510 mm maximum) would likely contribute to damage to the embankment structure, with cracking in the dam face, and in the parapet wall. The damage associated with the permanent deformations would not be expected to be sufficient to compromise the required performance of the embankment immediately following the seismic event. They are likely to compromise the performance of the embankment to the extent that repair, potentially of a very significant nature, would be required for the embankment to remain in service.

The additional permanent deformations that are estimated to result from the adopted aftershock event (a further 175 to 650 mm), are of a magnitude that would not be expected to compromise the performance required of the embankment following and MDE event.

This however does not imply the standard response measures employed to secure the embankment following such an event would not be required.

It should be noted that the aftershock scenario adopted is considered (appropriately) conservative. It is possible that such an aftershock could occur well before steady state flow through develops in the embankment, and equally that such an aftershock occurs after the reservoir has been lowered below the NTWL assumed in the analysis. In both these cases, displacements estimated would be less than those reported in Table 13.3.

14 Fish pass design

The fish pass design principle is unchanged from the Stage 1 design report, namely that the fish pass is designed for climbing species only. Options for the fish pass structure have been considered for the structure. These are discussed below, along with details at the upstream and downstream end of the main structure, and water delivery system to the structure.

14.1 Fish pass structure options

Three options were considered for the structure: these are concrete channel, precast fibreglass channel and natural rock placed to resemble river environment placed at the right bank of the embankment. The options are summarised in Table 14.1 including some of the disadvantages.

Table14.1: Fish pass structure options

Parameter	Concrete Channel	Fibreglass Channel	Rock Riprap
Material	Precast concrete channels	Precast fibreglass channels as supplied by fishladder solutions	Rock riprap most likely to be sourced from the river (150 – 300 mm rock)
Location	Along the downstream face of the dam on access track	Along the downstream face of the dam on access track	Along the right bank abutment
Slope	1V:3H to 1V:6H	1V:3H to 1V:6H	1V:2H
Width	300 mm	300 mm	-
Length (total)	180m to 330m	180m to 330m	130 m
Length (per unit)	1500 mm	2000 mm	-
Depth	300 mm	100 mm	-
Pool dimensions (length, width, depth)	450 mm X 675 mm X 1200 mm	200mm X 150mm X 200mm	-
Shading	No	No	No
Pump	2 l/s pump	2 l/s pump	5 - 10 l/s pump
Accessway width	1500mm	1500 mm	No access way
Water tightness mechanism	Grout	Units are made to provide water tightness as they bolt into each other	Fibre reinforced shotcrete
Angled sections	90 degree angles	10 degree angles	-
Weight	Heavy (heavy machinery)	Light (can be carried by labourer)	Heavy (heavy machinery required)
Foundation	Little is needed due to structure weight	Units are attached to the dam via steel posts founded on a concrete	Fibre reinforced shotcrete

		footing poured into the rock or anchored into the steel meshing	
Constructability	Heavy machinery and labour (manual) required. To be constructed during dam construction phase	Labourer/s can carry unit down the dam face. Can be carried out following dam construction	Heavy machinery required. To be constructed during dam construction phase
Ease of maintenance	No maintenance required provided dam settlement does not affect the structure	Ability to adapt to settlement to the dam face	Heavy machinery required. To be constructed during dam construction phase
Aesthetics	None	Resembles natural environment	Natural looking

A teleconference was held on 4 July 2012 between T&T and Cawthron (M Taylor, S Croft, S Basheer, R Strickland and J Haye) regarding the fish pass and to explore the options. Cawthron (pers. comm.) advised that the preferred arrangement was a rock lined channel on the right hand dam/valley interface. Whilst this is the steepest alignment Cawthron considered that it was adequate for the climbing species. Cawthron commented that a fish pass of similar steepness and around 30m total vertical climb for the same fish species operates successfully at Brooklyn dam.

The rip rap lined channel along the right-hand abutment has therefore been adopted for the Lee Dam.

14.2 Riprap lined channel

14.2.1 Downstream configuration

An important aspect of the selected fish pass design is the inlet at the downstream end. At this location, sufficient flow must be provided in order to attract the fish so that they will find the entrance to the fish pass. Following discussions with Cawthron, Fish & Game and WWAC it was agreed that the exact location of the outlet should be decided once the dam is constructed and monitoring is implemented.

14.2.2 Pump

As the fish pass is designed only for climbing species, the channel need only convey a small flow sufficient to provide a continuous wetted margin. The design flow rate selected for the channel also needs to ensure flow velocities are not too high given the relatively steep channel gradient. The flow down the channel is designed to be 5-10 l/s.

A pump is to be placed downstream of the dam, drawing water downstream of the outlet structure at a rate of approximately 5 l/s. The water is then pumped to a splitter box placed at the crest of the dam (on the parapet wall near the right abutment). Water will be pumped through a pipe placed along the downstream face of the dam.

The water source was chosen to be at the downstream end for the following reasons:

1. The quality of water at this end is considered better than that of the reservoir (given that the water at the downstream end is not still)

2. The constant tail-water level (in contrast with the variable water level in the reservoir) will assist with the operation and efficiency of the pump.

14.2.3 Upstream configuration

The upstream end of the fish pass consists of a splitter box and a structure to provide passage to the reservoir.

The main function of the splitter box is to split the pumped water in two directions:

1. Down the fish pass at the downstream side of the dam
2. Down the fish pass at the upstream face of the dam.

The splitter box is connected to the fish pass rock riprap on the downstream face of the dam embankment and a pipe on the upstream face. The pipe is placed at the upstream face of the dam, providing fish passage to the reservoir. It is important that this pipe is placed at the steepest slope possible to discourage fish from climbing up the pipe.

15 Spillway design

15.1 General and background

This section describes the design spillway arrangement and the basis of the design features shown in the Lee Valley Dam Design Drawings.

Spillway design standards are presented in Sections 1and 2 and the Design Criteria Report (T&T, 2011).

The selected spillway arrangement for the Lee Valley Dam includes the following components:

- 40 m long curved ogee weir on a 100m radius arc with one central bridge pier
- 200 m radius arc horizontal transition to a 20 m trapezoidal chute at 2H:1V grade
- 20 m wide trapezoidal shaped flip bucket with a 20 m bucket radius
- Unlined plunge pool excavated a minimum of 5 m into rock, the base of the pool is approximately 45 m long by 10 m wide.

Additional spillway and energy dissipation structure characteristics are presented in Table 15-1.

Table 15-1 Spillway and energy dissipation characteristics

Characteristic	Value
Chute length (plan – Ogee crest to start of flip bucket)	124 m
Chute width, narrow section	20 m
Chute maximum grade	2H:1V
Chute horizontal transition length	71 m
Chute vertical curve length	21 m
Chute minimum height of concrete lining	2.8 m
Dissipation type	Flip Bucket
Flip bucket radius	20 m
Bucket lip level	156.6 mRL
Flip bucket exit angle to the horizontal	40°

The feasibility design for the Lee Valley Dam incorporated two spillways: a primary spillway with an uncontrolled ogee crest and an auxiliary spillway with a fuseable embankment. During the Stage 3 design process the preferred configuration changed to a single uncontrolled ogee crest spillway, for a variety of reasons, including:

- Reduced consenting and operational risks associated with eliminating the fuseable embankment
- Costs associated with partitioning the fuseable embankment
- Better attenuation of storm peak flows over the full range of storm events
- Reduced long-term maintenance costs
- Improved passage of forestry debris.

As the Lee Valley Dam spillway design evolved, close similarities (design flow range, as well as horizontal and vertical geometry) between it and the spillway proposed for the Tillegra Dam in New South Wales, Australia, became apparent. Although the Tillegra Dam has not been constructed, a physical model study was carried out in 2009 as part of the detailed design process for the proposed spillway and flip bucket energy dissipator.

The Tillegra physical model included the spillway approach channel, the ogee weir, spillway chute and training walls, flip bucket, downstream plunge pool and the downstream channel. The level of detail and instrumentation for the model study was sufficient to enable simulation, observation and measurement of the following:

- Approach channel flow patterns, velocities and drawdown
- Ogee weir discharge rating curve
- Invert pressures on the approach, weir, chute and flip bucket
- Formation, propagation and interaction of contraction and pier shock waves
- Flip bucket jet trajectories and sweepout flow.

Only relatively minor adjustments to the Lee Valley Dam spillway configuration were required in order to take advantage of the measurement data from the Tillegra Dam spillway physical model study.

One of the authors of this report (Phil Carter) was in the design team for the Tillegra Dam and obtained permission from Hunter Water Corporation to use the physical model study findings to assist with the design of the Lee Valley Dam. Considering the advantages gained by having a model study to support the design, the Lee Valley Dam spillway design was therefore adjusted to match the Tillegra configuration as closely as possible given the site constraints. Hunter Water Corporation's cooperation in this matter is acknowledged and greatly appreciated.

The remainder of this section addresses the design aspects associated with the single spillway configuration shown on the Design Drawings.

15.2 Flood routing

Section 5 describes the hydrology for the dam and presents the flood hydrographs adopted for design.

Flood routing calculations were carried out using an in-house developed spreadsheet employing forward difference algorithms to determine the outflow based on the hydrograph inflows. Spillway outflow in any particular time step is determined by the water elevation in the reservoir. The in-house calculations were validated using identical simulations in HEC-HMS software. HEC-HMS is a hydrologic modelling system developed by the Hydrologic Engineering Center of the US Army Corps of Engineers.

Key flood routing results are summarised in Table 15-2. Figure 15-1, Figure 15-2 and Figure 15-3 show plots of the routing results for the PMF, 200 year ARI and Mean Annual Flood (2.3 year ARI) respectively.

Table 15-2 Key flood routing results[1]

Flood Event ARI (years)	Duration (hours)	Peak inflow (m^3/s)	Peak outflow (m^3/s)	Flood Rise (m)	Freeboard [2] (m)	Top WL (mRL)
2.33 (MAF)	48	210	179	1.89	3.74	199.09
5	48	268	239	2.21	3.42	199.41
10	48	314	285	2.45	3.18	199.65
20	48	359	330	2.67	2.96	199.87
50	48	416	388	2.93	2.70	200.13
100	48	457	427	3.09	2.54	200.29
200 (OBF)	48	502	472	3.28	2.35	200.48
1,000	48	601	568	3.67	1.96	200.87
10,000	48	742	708	4.17	1.46	201.37
PMF (MDF)	48	1094	1058	5.33	0.30	202.53

NOTE 1: All routing runs assume an initial reservoir level at NTWL

NOTE 2: 300mm camber (for settlement) excluded from freeboard assessment

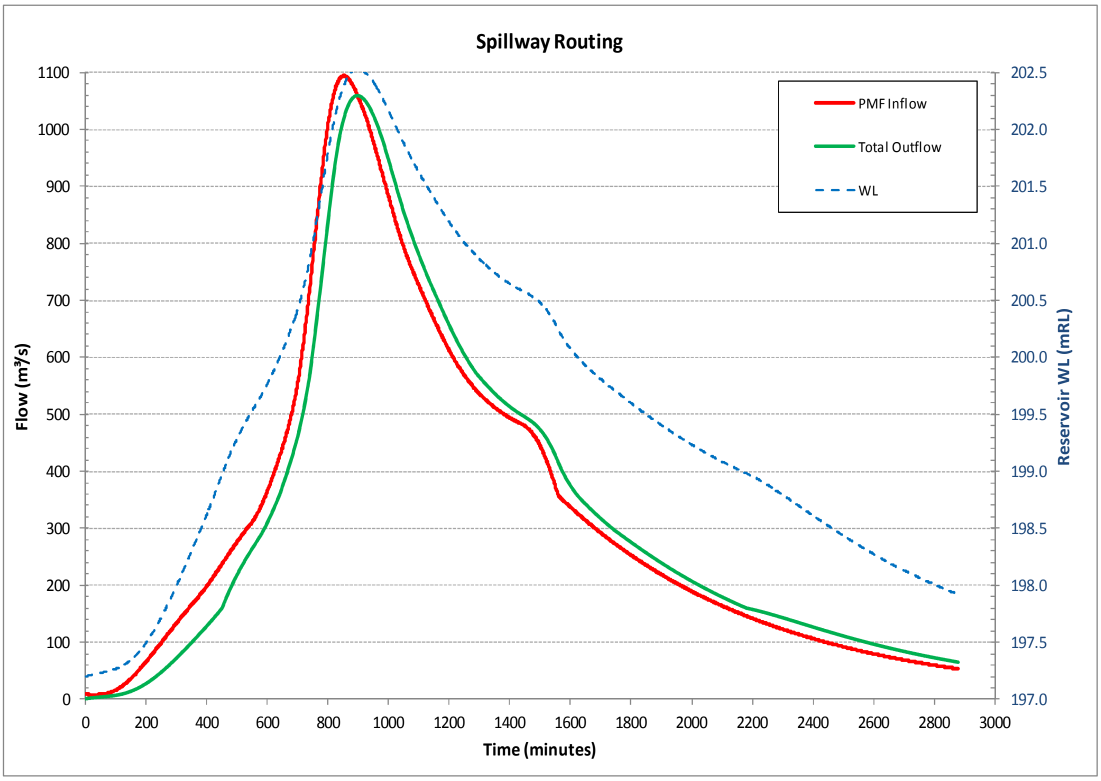


Figure 15-1 PMF routing plot

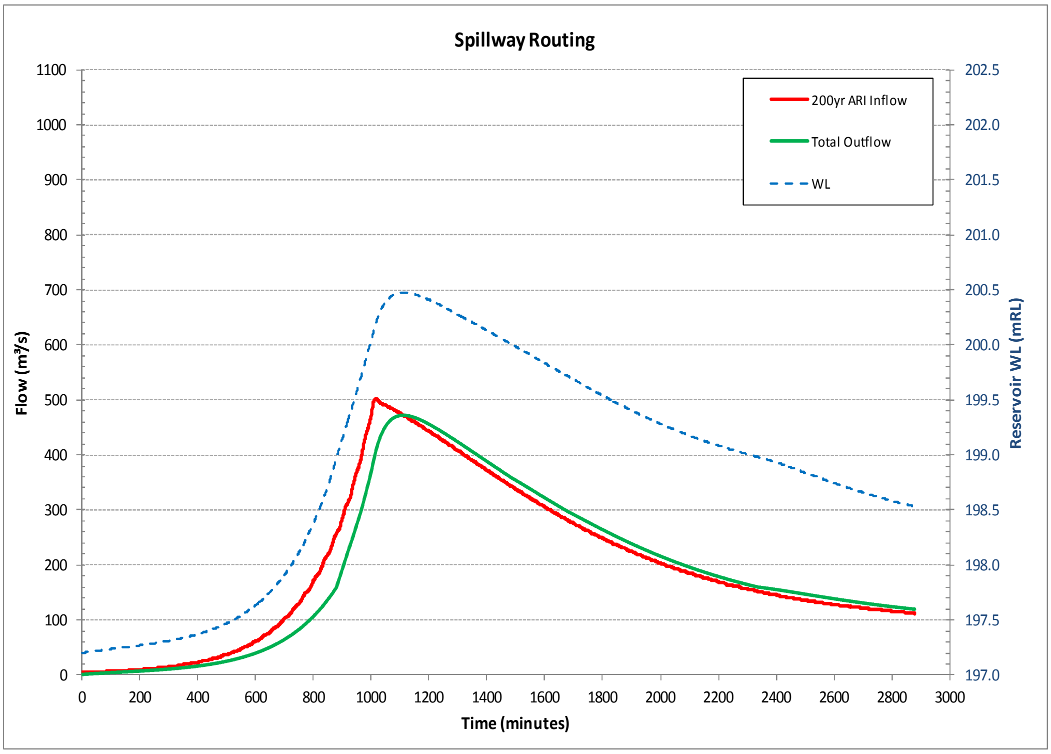


Figure 15-2 200 year ARI routing plot

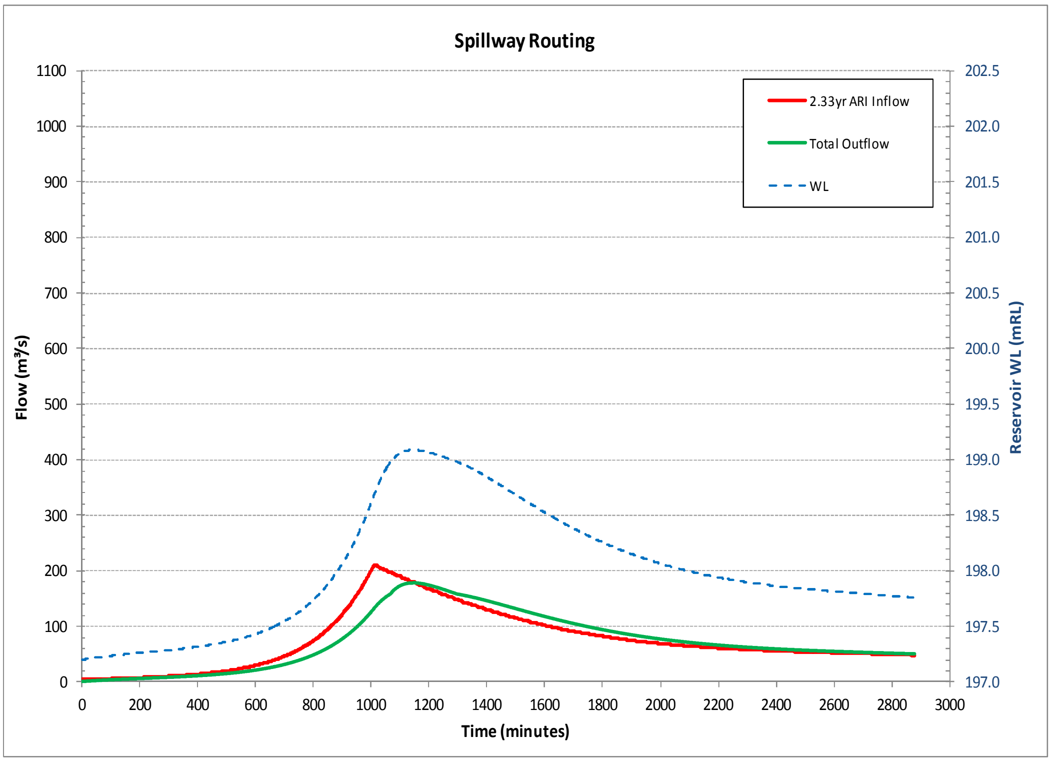


Figure 15-3 Mean Annual Flood routing plot

Design routing runs use the reservoir storage elevation curve shown in Figure 3-1 and incorporate an ogee weir rating curve based on the physical model study results presented

in Figure 15-4. Approach channel velocities are accounted for in the weir discharge rating curve.

The weir rating curve was checked against ogee crested weir equations presented in USACE EM 1110-2-1603, Hydraulic design of Spillways (1990) and found to be in close agreement.

15.3 Spillway approach channel

15.3.1 General

The approach channel is sized to maintain low design approach velocities and head losses upstream of the ogee weir. Similarly, approach transitions are gradual to minimise flow disturbances and contraction losses.

The true left side of the channel has a large radius curve to maintain a large radius of curvature to flow depth ratio (R/y). Khatsuria (2005) recommends that this ratio should be as large as possible but no less than 3. For the Lee Valley Dam operating at peak OBF discharge conditions, this ratio is greater than 13.

Concrete lining of the true right hand side of the approach (between the spillway and the dam) begins approximately 20 m upstream of the ogee weir to improve the approach conditions.

The approach channel floor has a 1V:200H grade to allow drainage away from the weir under low reservoir conditions.

Both the physical model study data and a one dimensional HEC-RAS model of the approach channel have been used to assess the hydraulics of the approach channel. HEC-RAS is a river analysis system developed by the Hydrologic Engineering Center of the US Army Corps of Engineers.

15.3.2 Velocities

Spot velocities were measured across the approach channel, approximately 30 m (ch 970m) upstream of the ogee weir centreline, for a range of flows as part of the physical model study. Interpolation of these measurements predicts the following approach channel velocities for the Lee Valley Dam design flows:

- At peak MDF discharge = 2.6 m/s
- At peak OBF discharge = 1.6 m/s.

The Lee Valley Dam spillway is expected to have somewhat lower approach channel velocities at the equivalent location. This is due to the presence of deep water (reservoir) at the right hand side of the approach and is supported by the HEC-RAS model; which predicts substantially lower velocities. Khatsuria (2005) recommends that the approach velocity for the design discharges should generally be less than 3 m/s but up to 6 m/s has been allowed.

The tapered nature of the approach channel means that velocities increase as flow moves downstream towards the weir. Interpolation of measured velocities on the upstream sloping side of the physical model study weir were 4.5 m/s and 3.6 m/s for Lee MDF and OBF peak flows respectively. Velocities calculated using HEC-RAS in the same vicinity are similar, though slightly less, being 3 m/s to 4.5 m/s and 2 m/s to 3.5 m/s for MDF and OBF flows respectively. These velocities are considered to be appropriate. The lower HEC-RAS computed velocities can be explained by the inability of the software to accurately calculate the brink depth at this location, and also the influence on the model of the deeper water on the right hand side of the approach channel.

15.3.3 Drawdown

Static pressure measurements were made at various locations in the approach channel and on the upstream side of the ogee weir as part of the physical model study. Analysis and interpolation of this data for the Lee Valley Dam design flows results in the following water surface drawdown immediately upstream of the weir:

- At peak MDF discharge, drawdown= 1.0 m
- At peak OBF discharge, drawdown = 0.6 m.

Drawdown calculated using HEC-RAS in the same vicinity are less, being 0.63m and 0.22 m for MDF and OBF flows respectively. The differences may be explained by the inability of HEC-RAS to accurately calculate the brink depth at this location and also the influence on the model of the deeper water on the right hand side of the approach channel upstream of the weir.

Both the above methods predict an MDF water surface drawdown clear of the soffit level of the bridge over the spillway.

15.4 Spillway weir design

15.4.1 Hydraulic design

The weir at the upstream end of the spillway chute is shaped based on the details derived from the hydraulic model study performed by the Manly Hydraulics Laboratory (2009). The weir is an ogee shaped weir, which is commonly used on dam spillways around the world and in New Zealand. The weir is nominally 40 m wide measured along the axis of the dam crest. To access the dam crest, a bridge across the spillway is necessary. To reduce the span length and cost of the bridge a 0.75 m wide central pier is included in the spillway. The spillway chute walls will be sloped at 1.5V:1H. The effective width of the spillway as modelled is 41.89 m. The variation from the nominal width is due to extra width from the sloping chute walls, less pier and abutment losses.

The weir crest level is 197.2 mRL (NTWL) with a minimum approach depth of 2.5 m. During the design flood (OBF) the operating head will be 3.3 m with a Coefficient of Discharge (C_d) of 1.90. The operational design flow will be 472 m^3/s during the OBF. The weir spillway rating curve is shown in Figure 15-4.

The underside of the spillway bridge is set to 202.45 mRL. This level is above the PMF water surface when drawdown and approach velocity affects are accounted for. Pre-camber in the bridge will allow for deadload deflections.

Supercritical flow is maintained once flow passes the crest as the downstream chute has adequate slope to ensure this (10H:1V followed by 2H:1V).

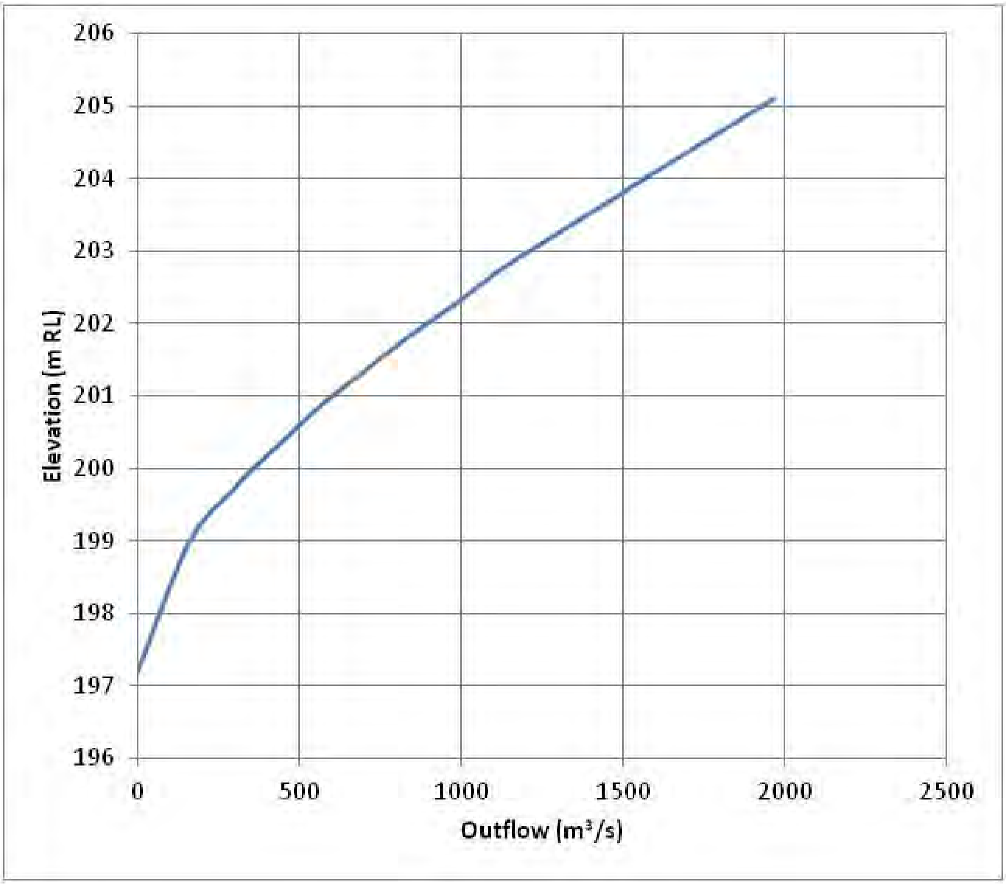


Figure 15-4 Ogee weir spillway rating curve.

15.4.2 Spillway (Ogee) weir structural design

Two load cases were investigated for stability against overturning and sliding. These two cases are:

- A static case with the reservoir at the PMF level
- A seismic case with the reservoir at NTWL during an MDE event (PGA = 0.48g).

The first case allows for the formation of negative pressures on the spillway crest due to operation above the design flow. Table 15.3 shows the results obtained. The resulting factors of safety exceed those recommended in the NZSOLD guidelines and are considered acceptable. Uplift pressures have been incorporated into the analysis. These have been assumed to act as a triangular stress distribution equal to static water level at the upstream end reducing to zero at the downstream end. This is a conservative approach given that the grout curtain extends under the full width of the ogee weir.

Table 15.3 Weir stability results

Load Case	Overturning		Sliding
	FoS	Resultant location	FoS
Static at PMF	1.71	Within base	4.38
NTWL with MDE	1.19	Within middle half of base	6.90

The following two design options were investigated for the weir stability:

- Gravity mass block; and
- Anchored design.

The mass block option is favoured and selected for design as it is not reliant on the mechanical fixings for stability. The anchored option may be adversely affected by long term corrosion of the rock anchors and would require on-going maintenance to ensure the integrity of the anchors is maintained for the life of the dam.

15.5 Spillway chute

Spillway chute details are shown on the Lee Valley Dam Stage 3 Design Drawings.

Design of the Lee Valley Dam spillway is generally based on the Tillegra configuration, for which physical model study data is available, as described in Section 15.1. However, HEC-RAS modelling was undertaken to confirm the results of the physical model study and extract additional infromation required for specific design of the Lee Valley Dam spillway. HEC-RAS modelling was carried out over a range of Manning's roughness values (n=0.008, n=0.014 and n=0.018) in accordance with USBR recommendations (USBR, 1987).

Downstream of the ogee weir the chute contains both horizontal and vertically transitions. In the vertical, the chute grade steepens from 10H:1V to 2H:1V over a 60 m radius convex vertical curve. Horizontally the chute contracts in a fan shape from 40 m at the crest to a minimum of 20 m in the steep portion of the chute (contraction ratio 0.5).

The vertical curve has been designed to maintain positive pressures over the invert and prevent flow seperation.

The horizontal contraction ratio adopted for the Lee Valley Dam is 0.5, which is considered to be relatively modest (ICOLD, 1992). The fan shape avoids abrupt changes in side wall angles and improves flow conditions, meeting USACE recommendations that chute sidewalls be curved horizontally with long radii when Froude numbers are greater than 1.5 (USACE, 1990). The spillway also has straight-lined contracting side walls running downstream from the ogee crest, followed by a curved transition. Experience has shown that this configuration can give better flow conditions than providing curved sidewalls immediately at the crest (Khatsuria, 2005).

Contractions may be subject to choking if local Froude numbers are close to 1 and a hydraulic jump is able to form in the contraction. To avoid this, ICOLD (1992) recommend that designs should be based on a minimum downstream Froude number of 2. HEC-RAS modelling of the Lee Valley Dam chute calculates Froude numbers greater than 2.5 in the contraction at the peak OBF discharge.

15.5.1 Height of concrete lining

The concrete lining for the Lee Valley Dam spillway has been taken to the MDF (PMF) chute water surface profile, accounting for wave action on the side walls and bulking due to air entrainment.

The design lining height was derived using the methodology described below.

A HEC-RAS model was developed of the Tillegra spillway configuration and design flows. The HEC-RAS model compares the measured physical model data with that predicted using one dimensional flow modelling.

The HEC-RAS model was run for a flow rate of 1495 m^3/s (1.4 times the Lee Valley Dam MDF flow) and the computed aerated water surface profile compared to the water surface profile reported for the physical model of the same flow. At this flow the HEC-RAS modelling showed that:

- Over most of the chute length the HEC-RAS model was found to overestimate the water surface profile by an average value of around 0.45 m (0.75 m maximum)
- The HEC-RAS model under predicts the water levels along the side walls to around 22 m downstream of the ogee crest due to the side wall contraction effects
- The HEC-RAS model under predicts the water levels along the side walls between approximately 80 m to 100 m downstream of the crest by an average value of around 0.4 m (0.6 m maximum)

The HEC-RAS model was then run for the Lee Valley Dam MDF using a conservative Manning's roughness value of 0.018 (USBR, 1987) and the aerated water surface profile calculated.

The design lining height was then based on this aerated water surface profile with the height increased in the areas where the HEC-RAS model underpredicted the required heights in the calibration step above.

The approach described above is the method by which the height of the concrete lining was determined. This top of the lining does not reflect the spillway chute freeboard. This is because the chute is contained within a rock cutting. Down most of the chute's length the rock cutting extends well in excess of 1m above the top of the concrete lining. There is one 10 m length on the right hand side of the chute (at approx. Ch1080 m) where this may not be so, the extent of which should be confirmed during construction. If deemed necessary, localised additional freeboard may be created in this area, for example by adding a short length of vertical concrete wall. In this same location dental concrete and rock bolting may be required following excavation. This will be addressed onsite.

15.5.2 Spillway cavitation

ICOLD conducted a survey of dam spillways in 1980. Where erosion of the spillway surface was found to be a problem, most were operating with maximum velocities greater than 30 m/s and specific discharges of over 50 $m^3/s/m$ (Novak, Moffat, Nalluri & Narayanan, 2007). The Lee Valley Dam OBF maximum channel velocity is approximately 26 m/s and the maximum unit discharge is 24 $m^3/s/m$. Both are below the above cavitation thresholds and thus the provision of spillway aeration devices is considered unnecessary.

15.5.3 Spillway floor anchorage and lining

In a typical concrete lined spillway chute the stability of the floor slab depends on multiple design elements including reinforcement, anchorage, joint and waterstop details, and a functioning underdrain system (USBR, 2007).

Damage resulting from hydrodynamic uplift on slabs typically begins at the joints, where offsets or spalling has occurred. Offsets may develop within the concrete lining at joints or cracks as a result of concrete shrinkage, differential settlement, ice pressures, or other loads. Spillway flows over these offsets can introduce water into the foundation, which can lead to structural damage due to uplift or erosion of the foundation material. Complete failure and removal of chute slabs has occurred on some spillways.

The build-up of pressures under spillway slabs due to high velocity flow depends on a combination of a number of relatively low probability events, at least for spillways with modern and well-constructed design details. Nonetheless, it is considered best practice to provide an underdrain system to limit the buildup of hydrodynamic uplift pressure under the concrete lining and design for uplift pressure by providing suitable anchors into the spillway chute foundaton. The Lee Valley Dam spillway chute is considered to be a critical structure given the proximity of the adjacent embankment and thus both defensive design features are provided.

The U.S. Department of the Interior, Bureau of Reclamation (USBR, 1971) recommends that chutes on rock should be designed for minimum hydrostatic uplift heads of 3 m. Other published methods such as that recommended by McLellan (1976) recommend the design uplift should be some fraction (k) of the velocity head ($h = \mathrm{k}\frac{v^2}{2\mathrm{g}}$). McLellan recommends k=0.15 where drains are provided and k=0.3 where there are no drains.

The argument for uplift being proportional to velocity head or stagnation pressure can result in very heavy reinforcement and anchorage. The method can result in anchorage requirements of twice the USBR method for even modest head spillway velocities (say 30 m/s). Velocities for high head spillways of around 45 to 50 m/s would require significant anchorage. However, large South American spillways with these velocities (such as Areia and Xingo) use relatively modest anchorage designs of around 120% - 150% of that derived using the USBR method. It is acknowledged that these are very large spillways and generally include aeration devices in the chute that may have some effect.

Cognisant of the above, the hydrostatic uplift head selected for design of the Lee Valley Dam spillway chute is based on a proportion of velocity head with k=0.15 but capped at 3 m.

Chute anchors are designed to hold down the slab, to resist the uplift pressures less the normal weight of the slab and depth of water in the chute. A load factor of 1.2 was applied to the uplift load and a strength reduction factor of 0.9 on the yield strength of the anchor bars.

The minimum compressive strength of the anchor bar grout is 30MPa at 28 days with the following ultimate bond strengths adopted for design:

- Anchor bar grout to rock of 4MPa with an applied factor of safety of 3.0, based on competent rock with an ultimate compressive stress greater than 20MPa (from BS8081 Table 25)
- Anchor bar to grout of 2MPa (from BS8081 Cl.6.3.2).

The anchor design methodology takes account of overlapping pull out cones for adjacent anchors, assumes a submerged unit weight of rock and ignores side friction effects for the mobilised rock mass.

The floor lining is formed of 300 mm thick reinforced concrete with all longitudinal and transverse joints provided with PVC waterstops. Reinforcing steel is provided throughout to control shrinkage and thermal cracking. Drainage provisions are discussed in Section 15.5.4.

15.5.4 Spillway floor drainage

The USBR carried out a study in 2007 (USBR, 2007) to investigate uplift pressures and resulting flows into cracks and joints caused by high velocity spillway chute flows. The generation of such uplift pressures and flows relies on a break in the continuity of the lining and some feature that transfers a portion of the velocity head below the lining. These breaks in continuity can be at joints or cracks that may develop as a result of concrete shrinkage, differential settlement, ice pressures and other loads, or due to age deterioration.

The transmission of pressure and flow beneath a chute lining depends on a number of factors, including gap width, offset height, orientation to the flow direction and a variety of other geometry and flow related features. The transmission of flow through a properly designed joint with a PVC waterstop would also require a lack of integrity of the waterstop.

As mentioned in the preceding section, providing spillway under-drainage is common practice. This can include pipe drains or drilled eductor drains. To be effective, eductor drains need to be close to the crack or defect that introduces high pressure and typical spacings may generally be too wide for this to occur. Also, the USBR study (USBR, 2007) demonstrates the considerable flow that may be generated by even a small gap into the subsurface drainage system.

Cognisant of the above, perforated underdrains have been selected for the Lee Valley Dam spillway. These drains are located under each of the spillway's transverse contraction joints and discharge into longitudinal collector drains running down either side of the chute.

The transverse drains for the Lee Valley Dam are sized based on the following methodology:

- Potential unit flow rates into the defect calculated based on the HEC-RAS model spillway velocities and the USBR study findings (USBR, 2007)
- Drains sized to convey flow from a defect with a 3 mm (1/8 inch) gap and 3 mm offset with a vented cavity extending over 25% of the chute width
- Drains assumed to be full flowing pipes with a maximum head of 3 m, corresponding to the hydrostatic uplift head for which the anchors are designed
- Drain flow losses account for exit losses into the longitudinal drains as well as 'screen' entry losses and friction losses based on well screen research (Barker & Herbert, 1992a) (Barker & Herbert, 1992b)
- An allowance has also been made for a provisional number of drilled eductor drains should these be considered necessary in some locations during construction of the spillway.

15.6 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. The flip bucket design is considered to be the most economical type energy dissipator commonly used in spillway design. It is therefore appropriate for the Lee Valley Dam.

The bucket radius of 20 m from the Tillegra hydraulic model study has been adopted. This value was checked against methods developed by Mason (1982), USACE (1990), USBR (1987) and Varshney & Bajaj (1970). Using the MDF flow 1060 m^3/s these methods recommended a range of radii from 14 m to 20 m.

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° – 40° (Khatsuria, 2005). An angle of 40° was adopted to match the parameters used in the Tillegra hydraulic model study. This allowed the use of the results from the model study to estimate the location of the impact zone and provided greater certainty in design. The lip height is set to 156.6 mRL which is the approximate MDF tailwater level.

The water pressures developed within the flip bucket whilst operating at the PMF have been investigated using three methods. Theoretical pressure distributions were calculated using methods developed in USACE (**1**987) and USACE (1994). The results from the Tillegra Hydraulic Model Study (Manly Hydraulics Laboratory, 2009) included measurement of static pressures within the flipbucket. These results were linearly interpolated to the PMF case for Lee Valley (1060 m^3/s) and produced comparable results to the theoretical methods. The results from these methods are shown on Figure 15.5.

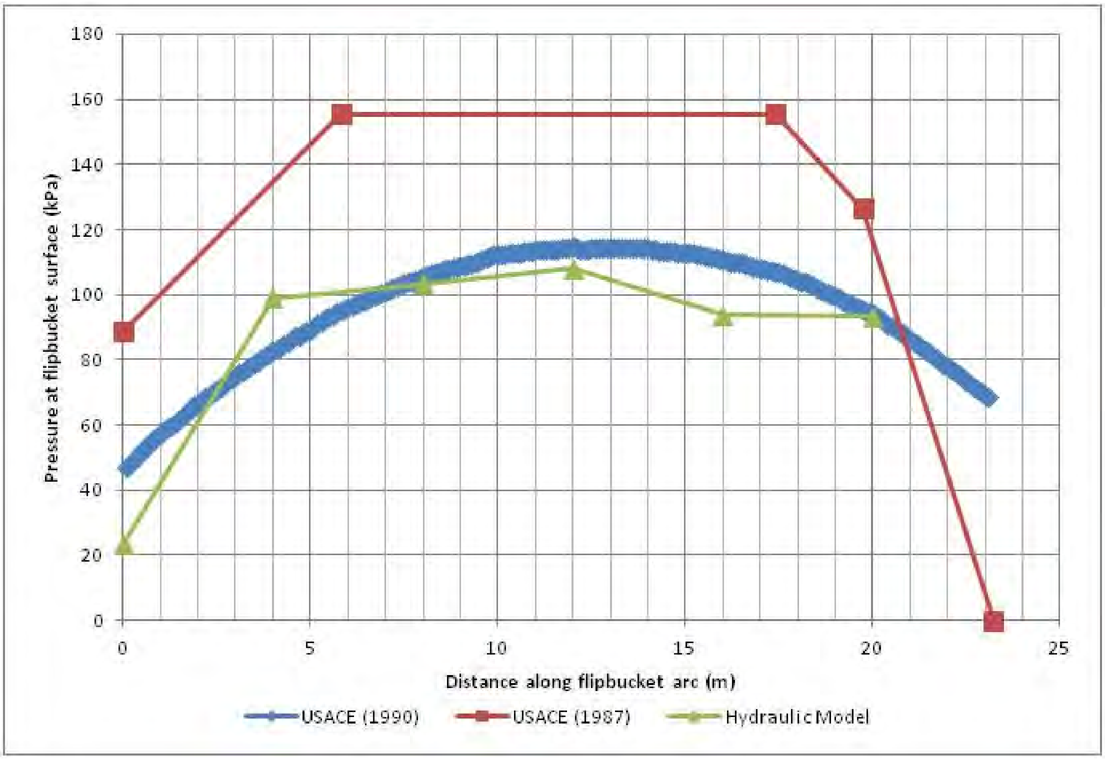


Figure 15.5 Comparison of methods for estimating hydraulic pressures in the flip bucket

Two load cases were investigated for stability against overturning and sliding. The PMF flow without seismic acceleration and with the reservoir at NTWL coupled with an MDE

event. This analysis allowed for the formation of negative pressures on the spillway crest due to operation above the design flow. Table 15.4 shows the results obtained.

Table 15.4 Flip bucket stability results

Load Case	Overturning		Sliding
	FoS	Resultant location	FoS
PMF	1.74	Within middle third of base	20.6
NTWL with MDE	2.39	Within middle third of base	12.4

The estimated maximum pressure exerted on the bedrock supporting the flip bucket is 80 kPa. The resultant is located at the downstream edge of the flip bucket foundation. The maximum bearing capacity of the rock was estimated at 30 MPa using a method described by Bowles (1996). Thus, notwithstanding unforeseen conditions, there is adequate capacity in the rock to support the pressures exerted by the flip bucket.

The area immediately downstream of the bucket will be subject to frequent flows, lower than the design level, that do not sweep out and become airborne. During these low flows there is potential for erosion and undermining of the flip bucket. As such it will be protected with a 0.3 m thick layer of concrete downstream of the flip bucket.

15.6.1 Flip bucket structural design

The flip bucket has been analysed using SAP2000 software for the following Ultimate Limit State load combinations:

- 1.2 x Dead Load & 1.3 x PMF dynamic water pressure
- 1.2 x Dead Load & 1.5 x Hydrostatic water pressure
- Dead load & Seismic (MDE).

The following serviceability limit state load combination has been analysed to calculate crack widths:

- Dead load & PMF dynamic water pressure.

Concrete crack widths have been calculated to be limited to approximately 0.3 mm under the serviceability load combination.

The base and true left wall of the flip bucket are assumed to be founded on Class 1 Rock. The true right wall is conservatively assumed to be freestanding (cantilevered). This is because of uncertainty of rock conditions under the true right wall. The true right wall is tapered from the top to its base to reduce dead load induced moments. It is likely that dental concrete or backfill with compacted rockfill will be placed under the true right wall. The requirements for backfill will be confirmed onsite.

15.7 Plunge pool

The plunge pool comprises a trapezoidal unlined channel downstream of the spillway and flip bucket as shown in Drawing 27425-SPL-06. The base of the pool is approximately 45 m long by 10 m wide and excavated 5 m into Class 1 rock based on the current geological model. At the downstream end of the pool the channel invert rises back to river level

(147.18 mRL), ensuring the pool is at least 3 m deep at the upstream end and 5 m deep at the downstream end, even at extreme low flows.

To identify the location that scour is likely to occur and the extent of pre-excavation of a plunge pool that will provide the most benefit, it is necessary to predict the trajectory of the jet from the flip bucket during a range of design flows. The physical model study data from the design of Tillegra Dam was used to calibrate the Kawakami (1973) method for calculating trajectories of a free jet from the spillway. The effective lip angle and air resistance parameters were changed to replicate the real jet trajectories found in the physical model study, and these parameters were interpolated /extrapolated for the velocities and flow rates for the mean annual flood, OBF and PMF at Lee Valley Dam.

The likelihood and extents of scour have been estimated using a variety of empirical methods. Some of these methods consider the strength of the rock (Annandale 1995, Van Schalkwyk 1994, Khatsuria 2005), while others do not (Mason 1985, USBR 1987, Yildiz & Uzucek 1994, Damle 1966, Chian Min Wu 1973, and Martins 1975). As noted in Large Brazilian Spillways (2002), the methods considering rock mass quality are “relatively recent” and “sufficient experience has not yet been accumulated regarding the representativity of the proposed systems”.

Mason (1985) reviewed a significant number of the empirical methods from the school of thought that scour extent is largely independent of rock mass quality (as espoused by Mason 1993). The predicted scour depths at Lee Valley Dam based on Mason’s equation derived from his collation / review are summarised in Table 15.5. The table also presents the smallest scour depths predicted (Damle 1966) from the methods considered for the Lee Valley Dam. The predictions from all the methods that did not consider rock mass quality that were considered for Lee Valley Dam are bracketed by the predictions for Damle (1966) and the Mason (1985) upper bound estimates.

Table 15.5 Scour depth estimates

Flood	**Mason (1985) Typical**	**Mason (1985) Upper Bound**	**Damle (1966)**
Mean annual flood	11.6 m	23.2 m	7.5 m
OBF	18.7 m	28.1 m	12.1 m
PMF	27.8 m	41.7 m	17.9 m

NOTE: Scour depths are measured from water level (rather than existing bed level) to bed level after scour.

The scour profile was projected upstream from the point of maximum scour (represented by the tabulated estimates above) towards the flip bucket and dam embankment using guidelines provided by Mason (1993), Bollaert (2004) and Taraimovich (1978). Even using the most conservative estimate (Mason’s 1985 upper bound estimate) the predicted scour profile does not extend back to the flip bucket or dam embankment.

A nominal amount of pre-excavation has been allowed for in the plunge pool design, which correlates to a plunge pool depth of 6.6 m at the upstream end of the pool and 8.7 m at the downstream end of the pool during the mean annual flood (water level 150.85 mRL). This is close to the lower bound of scour estimates for the mean annual flood (Damle 1966). It is likely that scour beyond the pre-excavation extents could occur for the mean annual flood, and scour beyond pre-excavation extents is highly likely for the larger events. However, the additional scour is considered acceptable since the scour estimates and profiles assessed indicate that it will not affect the flip bucket and dam embankment stability.

15.8 Peer review comments

Table 15.6 includes responses to Opus peer review comments.

Table 15.6 Opus peer review comments

Opus peer review comment	Response
Spillway Capacities It is not clear from the report what the overall philosophy is with respect to proposed spillway capacities and operation. When is the auxiliary spillway proposed to be first activated? What is the discharge capacity of the primary spillway at this time? What is the headwater level at the dam? What flood magnitude does this correspond to? When is the second part of the auxiliary spillway proposed to be activated? What are the discharge capacities of both the primary and auxiliary spillways at this time? What is the headwater level at the dam? What flood magnitude does this correspond to? What is the overall design capacity of both the primary and auxiliary spillways? What is the design headwater level at the dam? What flood magnitude does this correspond to?	The auxiliary spillway has been deleted in favour of a large single spillway. Refer to Section 15 for detail.
Spillway Hydraulics The sharp side-wall contractions shown in the spillway chute layout will give rise to the formation of cross-waves which will be reflected back and forth across the chute at they are conveyed downstream by the high velocity super-critical chute flow. The side-walls will need to be high enough to contain these cross-waves. The magnitude of the cross-waves will be exacerbated if the width of the ogee crest is increased.	Spillway completely redesigned and supported by physical model study.
Under PMF conditions, the flow velocities in the flip bucket (ski jump) at the bottom of the spillway chute are expected to approach ~27m/s. For flow velocities of this magnitude (> 20m/s), the potential for cavitation in the spillway flip bucket is significant and needs to be addressed. Cavitation mitigation measures may be required. The jet projecting from the end of the flip bucket at the end of the spillway chute will spread laterally before impacting in the area of the plunge pool downstream, At the present time the plunge pool geometry shown on drawing no. 27425.100-100 does not show any divergence from the flip bucket structure to	Refer to Section 15 for consideration of the flip bucket and plunge pool design

accommodate this spread of the spillway jet. Erosion of the left bank adjacent to plunge pool is therefore likely to be a significant issue. The area of the plunge pool needs very careful attention to minimise the risk of bank erosion. This may require excavation of a wider plunge pool area and adjustments made to the spillway chute and flip bucket geometry to direct the spillway jet to fall more in the centre of the river. The effects of river bed erosion by the falling spillway jet in the plunge pool area needs to be addressed.	

16 Bridge design

There are two bridges required at the Lee Valley Dam:

- Lower bridge - at the toe of the dam over the primary spillway on the toe access road. This bridge is to provide access to the outlet works and power station (if it is to be constructed); and
- Upper bridge - at the top of the dam on the crest access road. This bridge is to provide access to the crest of the dam and the primary irrigation intakes.

16.1 Dimensions of the bridges and key data

Table 16.1 summarises the key dimensions and design criteria for the bridges. This information is consistent with the Design Criteria Report (T&T 2011). The width of the bridges has been limited to keep construction cost to a minimum.

Table 16.1 Bridge summary

Description	Upper Bridge (Dam crest road)	Lower Bridge (Dam toe road)
Deck width - overall (between kerbs)	4.4 m (4.0 m)	4.4 m (4.0 m)
Design vehicle	6 wheel, 11 m long truck with 8.2 tonne axles	6 wheel, 11 m long truck with 8.2 tonne axles
Bridge length	Single 25 m clear span (26.2 m bearing to bearing	Two 25 m clear spans (26.2 m bearing to bearing)
Bridge type	Steel beam sub structure with composite concrete deck	Steel beam sub structure with composite concrete deck

16.2 Design vehicle and loadings

Because the bridges are on a private road, there is no New Zealand Standard to define the loadings for the bridges. For this project we have adopted some of the provisions of New Zealand Transport Agency's (NZTA's) Bridge Manual. Not all provisions and criteria in the Bridge Manual have been adopted because they are intended for State Highways that have high volumes of traffic. Furthermore, to adopt the bridge manual requirements in their full extent would result in a more conservative and therefore expensive design.

Therefore a pragmatic design vehicle has been selected (Design Criteria Report, T&T 2011) which is a 6 wheel, 11 m long truck with an 8.2 tonne axle load. This size of vehicle would be suitable for transporting materials that might be required for most future maintenance of the dam, outlet works and power station e.g. aggregate, valves, portable generators, compactors, small excavators (around 1- 8 tonne size) etc.

Whilst the design vehicle is a 3 axle, 11 m long truck with an 8.2 tonne design axle, we have also considered a single HN (maximum legal weight limit vehicle) vehicle (not acting concurrently with a UDL) on any given span of the bridge. The bridge design has adequate capacity for this vehicle loading. Figure 16.1 shows the key dimensions of the design vehicle.

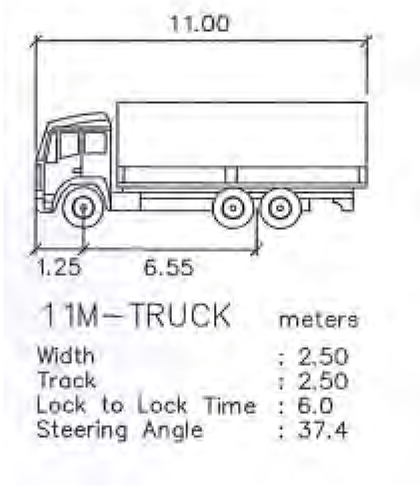


Figure 16.1 Lee Valley Dam design vehicle for bridge design

In the unlikely event that larger or heavier vehicles are required to gain access across either of the two bridges, then temporary support could be provided to the bridges. The temporary support would need to be designed appropriately for the loads under consideration.

An alternative live load to the design vehicle has also been considered. This is a uniformly distributed load (UDL) of 5 kPa. This UDL is greater than what is specified for the UDL portion of HN loading in the Bridge manual, but is consistent with NZS1170 for UDL's in car parking buildings (bridges fall outside the scope of NZS1170). We therefore consider it appropriate for these bridges.

A dynamic load factor of 1.22 has been applied to the design vehicle to account for the impact of the vehicle moving across the bridges. The factor has been derived using the approach outline in the Bridge Manual. A dead load factor of 1.2 and a live load factor of 1.5 have been adopted in analysis and design.

No overload element has been considered in the design of these bridges.

16.3 Bridge type

The bridge type selected for both locations is a steel beam sub structure with composite concrete bridge deck. The design is based on the deck providing 75% composite action with the steel beams.

The primary reason that this bridge type has been selected is for constructability. The other alternative that was considered was a concrete bridge (e.g. Super T's). Modern concrete bridges in New Zealand are normally constructed in pre-cast segments and then transported and lifted into position. Given the poor condition of the access road to the dam site it is considered that there would be difficulties in transporting bridge beams in excess of 26 m long to the site.

The advantage of transporting steel beams is that they can be manufactured in segments and spliced together onsite before being lifted into position.

16.4 Bridge deck

The concrete deck is designed to act compositely with the steel beams. This is more economical than designing the steel beams to carry the weight of the bridge deck (commonly referred to as dead load) and the live load (either vehicle or distributed loads) on their own. The bridge decks have been designed in accordance with NZS3101.

The nominal deck thickness is 180 mm. Traydec is a proprietary galvanised steel formwork that has been specified as temporary formwork for the bridge deck. Traydec is commonly used as permanent formwork in building structures in New Zealand. It is specified as a temporary measure here because it requires limited support during construction. However once cured (after approximately 28 days) the concrete will not require the galvanised steel sheets. The Traydec steel will corrode without reducing the design strength of the bridge. We do not consider it cost effective to apply corrosion resistant coatings to the Traydec. The long-term design therefore ignores the benefit that the Traydec may provide.

The concrete deck spans perpendicular to the bridge span between the longitudinal beams.

No formal drainage for the bridges is to be provided. Surface water will be shed via cross fall to regular gaps or drainholes in the upstand kerbs.

The bridge decks are intended to be left uncoated (i.e. no running coarse). This is to keep construction costs to a minimum. The concrete finish will be specified in Stage 4 design.

16.5 Bridge beams

The beams are 1000WB249 custom welded steel beams. These beams are readily manufactured by New Zealand Steel in Glenbrook (Greater Auckland). However an experienced steel fabricator could manufacture the beams if this is more cost-effective.

The beams are specified to be pre-cambered (upwards). This is intended such that the beams deflect to be approximately straight under the full dead load of the bridge.

16.6 Bridge pier and abutments

The abutments of both bridges are reinforced concrete beams. The beams are nominally shallow foundations approximately 0.6 m wide and 4.5 m long. The left hand abutment of the bridge will be founded on Class 2 rock. The right hand side of the upper bridge will be founded on the dam mass concrete abutment.

Based on the site investigations to date we expect that there will be overbreak of the spillway excavation. This overbreak may result in an unsuitable foundation for the bridges. We have therefore allowed to thicken the spillway walls locally under the bridge abutment beams to compensate for this circumstance. The exact detail under the bridge abutment beams will need to be reviewed on site following the excavation of the spillway.

No significant settlements of the foundations are expected.

The alignment of the upper bridge may be refined in the Stage 4 design to transition the dam crest and bridge approaches. Skewing the abutment beams will also be considered.

16.7 Fall protection and guardrails

Industrial handrails are provided to the bridges because the bridges are:

- In an isolated location
- On private land
- Not accessible to the public;

i.e. they are not suitable for protection from falling for children. This may be challenged by the regulatory authority (Tasman District Council) when applying for building consent. However in accordance with WWAC's aspirations to keep costs to a minimum the basic industrial handrail has been adopted.

Upstand kerbs are provided to prevent vehicles from falling off the bridge. The design of the upstands and their capacity will be defined in Stage 4. Guardrails are provided at the approach to the bridges. The design of the guardrails will be undertaken in Stage 4.

16.8 Assumed construction sequence

The following construction sequence is assumed for the bridge construction (similar for the two bridges). The design of any temporary support is the responsibility of the Contractor:

1. The bridge concrete abutment beams and central bridge pier (upper bridge only) are cast insitu
2. Bridge beams are fabricated off site and pre-painted (with shear studs and web stiffeners welded onto the beams)
3. Bridge beams are transported to site in 8-12 m long segments
4. The beam segments are spliced together to form 26.2 m long beams
5. The beam pairs are connected using the permanent equal angle cross bracing (the Contractor may need additional temporary bracing to prevent racking of the beam pairs)
6. The beams are lifted into position in pairs (maximum single lift weight is approximately 12 tonnes). The beams are required to be placed in pairs to prevent buckling of the beams by wind or construction live loads during erection
7. The remaining cross bracing connecting the beam pairs is bolted into position (this is required to restrain the beams during concrete placement)
8. Once all four beams are in position the Traydec is placed
9. Deck reinforcing is fixed into position
10. The concrete decking is poured and cured
11. Handrails are fixed into position
12. The steel beam paint system is touched up as required.

17 Debris boom

The catchment above the Lee Valley Dam is generally forested with commercial exotic forest species and pockets of indigenous vegetation. Although the reservoir area is to be cleared of vegetation, in the long term, localised or widespread mobilisation of forestry debris, associated with heavy rainfall events, needs to be managed. It is considered likely that large debris rafts could form on the reservoir at some stage and the risk of compromising spillway capacity needs to be appropriately mitigated.

The dam design thus includes a debris boom to provide protection from this hazard and to facilitate safe maintenance of debris in the long term. The debris boom will require regular clearance of accumulated debris by the dam owner.

The recommended debris boom is a Worthington Products Incorporated (WPI) 300 m long TUFFBOOM waterway barrier with 610 mm debris screens and an in-water mooring buoy to alleviate load and maintain stability. T&T has not designed the boom or anchor arrangements. We recommend this design work be undertaken by or in conjunction with the supplier WPI, who offer this service.

18 Outlet works

18.1 Outlet works summary

The design of the outlet works is described in detail in Parsons Brinckerhoff (PB) Report "Lee Valley Dam Mechanical Design Report Preliminary" (PB 2012). A summary of the key features is given below whilst the full report is contained in Appendix G.

A HAZOP workshop attended by the designer, the peer reviewer and representatives of WWAC was held at T&T on 27 March 2012. The workshop concentrated on the health and safety aspects of the operation of the outlet works in the conduits. The outcomes of this meeting were documented and the design has considered these aspects.

The requirement for permanent ventilation should be considered during Stage 4. We note that this is currently outside T&T's design scope. Given that the contractor will likely require a ventilation duct along the conduit for construction, for the permanent works, a fan can be brought to the site or stored at site and installed when access into the conduit is required.

The outlet works comprise intake screens, pipework and valving which have to manage the flows summarised in Table 18.1 below.

WWAC should be aware that PLCs, telemetry and power supply are outside the scope of T&T's engagement and are therefore not covered here.

The penstock design has been carried out to Stage 3 level in accordance with our engagement, but the Stage 4 design of the penstock is outside T&T's design scope.

The outlet works are also based on the expected requirements for environmental flows, i.e. a minimum residual flow at the base of the dam of 511 l/s and provision for flushing flows of 5000 l/s (Cawthron, 2009).

Table 18.1: Outlet Works Summary

Duration	Reservoir Level	Min Residual Flow (l/s)	Additional Downstream Flow Demand (l/s)	Flushing Flows (l/s)	Outlet flow (l/s)
Continuous (Note 1).	Min Operating to Max Flood	511	0	0	511
Depends on demands and inflows (Note 2).	Spillway to Max Flood	511	Met by spill flows	0	511
Depends on demands and inflows (Note 3).	Min Operating to Spillway	Included in Downstream Flow Demand	511 to 2230	0	511 to 2230
Limited duration and frequency in summer (Note 4).	Min Operating to Spillway	Included in Flushing Flows	Included in Flushing Flows	5000	5000

Notes :

4. Consent may allow for occasional non release of flow during critical maintenance activities. Outlet works flow is assumed to continue when the dam is spilling.

5. Outlet flow > 511 l/s is generally when the dam is not spilling though there are occasions of minor spill when the outlet flow is > 511 l/s (from model outputs highest bottom release flow of 823 l/s with 235 l/s of spill).

6. Irrigation demand is in January to April but other demands continue throughout the year. Outlet flow > 511 l/s is generally when the dam is not spilling although there are occasions of minor spill when the outlet flow is > 511 l/s.

7. The consent will set the frequency and duration of flushing flows.

There are two intake pipes, one a high level intake at RL 185 m and the other a low level intake at RL 166.5 m based on the recommendations for water quality detailed in the "Cawthron report No. 1701, Dec 2009". The main features of the outlet works are summarised in Table 18.2 below.

Table 18.2: Outlet works summary

Parameter	Value	Comment
Number of outlets	2	Two outlets required high level and low level.
Maximum Design Flow	5000 l/s	Flushing flow (occasional releases) may be required through a single outlet.
Normal Upper Design Flow	2230 l/s	Maximum forecast downstream release. May be through one or both outlets.
Minimum Design flow	511 l/s	To meet minimum residual flow. May be through one or both outlets.
Minimum operating water level, upper intake	RL 185.0 m	Based on requirements for upper intake in Cawthron report No. 1701, Dec 2009.
Minimum operating water level, lower intake	RL 166.5 m	Based on requirements for lower intake in Cawthron report No. 1701, Dec 2009.
Intake Screen	-	To protect the downstream pipework and valving by preventing debris from entering the pipework. Also to provide protection to aquatic life by limiting the bar spacing and approach velocity. The intake bellmouth level is set below the minimum operating water level to prevent vortices forming and air being drawn into the pipework. The removal of the screens can be achieved by winching the intake structure up the face of the dam.
Inclined Intake Pipework	1200 mm diameter steel	The pipework design is based on epoxy coated and lined spirally welded steel pipework, bends and fittings. Steel pipework has been selected as it has less specialised manufacturing processes, and also provides

	8, 10 or 16 mm thick depending on location.	the additional flexibility of being able to weld components together, either on-site or in the factory using routine techniques. The minimum diameter of the pipeline was determined by consideration of the maximum velocity through the primary isolation valve (see below) and to minimise erosion in the long radius bends at the base of the dam / inlet to conduit. For simplicity the diameter of the inclined intake pipework has been sized to be the same as the long radius bends and to reduce the long term internal erosion of the, difficult to access, pipework. Minimum wall thicknesses are recommended based on requirements of internal pressure, shipping, handling, buckling, impact loads and robustness. The removal and adjustment of the inclined intake pipework can be achieved by use of divers and winching the individual pipe lengths up the face of the dam.
Primary Isolation Valve	1200 mm diameter gate valve. 160 m pressure rated (PN16)	Required to isolate the pipeline and valves in the conduit to allow maintenance of these items. This needs to be located as far upstream as possible in the conduit to minimise the risk to the conduit and dam caused by the pressurised pipework. A wedge type gate valve is the most secure and robust valve option as it has two separate sealing faces and the physical arrangement does not allow the gate to be dislodged. The valve sizing is based on the recommended maximum velocity from a reputable valve supplier. The valves are recommended to be electrically actuated to allow the valves to be remotely opened and closed without the need to access the upstream end of the conduit. The electric actuators will also provide a method of shutting the primary isolation valves in an emergency, should a major leak downstream of the valve prevent safe access to the valve actuator. The valves will also be capable of manual operation. The primary isolation gate valves will be provided with a small bypass valve to balance the upstream and downstream pressures on the gate valve to aid the operation of the gate valve. The valve is in a difficult to access location and will be difficult to maintain and remove (if needed). It is therefore critical that a high quality valve is installed and thoroughly tested and witnessed at the factory.
Conduit Pipework	1000 mm diameter steel 6 or 8 mm thick depending on location.	The pipework design is based on epoxy coated and lined spirally welded steel as for the inclined intake pipework. To minimise the downstream pipework costs, a smaller internal diameter pipework is proposed downstream of the primary isolation gate valves. This is an area that

		can be more easily maintained through the use of the primary isolation valve. The pipework will be provided with air valves so that air can be released during filling and drawn in during emptying and to ensure that vacuums are not formed.
Fixed Cone Valve	800 mm	The fixed cone valve is required to discharge the downstream releases in a controlled and adjustable manner. Other valve options are possible but tend to be more expensive. The valves are sized to pass the flushing flow under the minimum gross head, i.e. at the minimum operating level. Both fixed cone valves are proposed as the same size to ease maintenance and operation. The valves are capable of operation over a wide range of opening and therefore allow for a good range of flow mixing from either intake. The valves are sited at the downstream end of the conduit and may need a hood to ensure the discharge envelope lands within the downstream channel.

18.2 Dewatering capacity

The low level outlet works are designed to meet multiple requirements which include:

- Project operational releases such as minimum residual flow, irrigation discharge and environmental flushing flow
- Diversion releases
- Controlling the rate of reservoir rise during first filling
- Dewatering the reservoir if emergency conditions occur, or inspection, maintenance and repair of the dam and appurtenant works that are normally submerged is required.

USBR ACER Technical Memorandum No. 3 (1982) (hereinafter referred to as USBR TM3) provides criteria and guidelines for determining suitable reservoir evacuation or dewatering rates. Determining the evacuation period requires routing flows through the outlet facilities in conjunction with recommended reservoir inflows as follows:

- **Reservoir filling**
  Inflow during filling should assume an average of the mean monthly inflows for the selected filling period as well as a flood with a recommended frequency of approximately five times the duration of the filling period

- **Reservoir evacuation**
  Reservoir inflows should be based on the highest consecutive mean monthly inflows for the duration of the evacuation period

The High PIC status of the Lee Valley Dam roughly equates to a High hazard classification in USBR TM3, though the risk status is subjective and more difficult to classify. We have not carried out an assessment to determine the dam’s risk status in terms of the categories described in TM3. However, we consider it unlikely that it would have a High-Risk status, although it could conceivably be given a Significant-Risk status.

USBR TM3 general guidelines for determining High-Hazard dam emergency evacuation times are presented in Table 18-. These values are based on USBR experiences and endeavour to reflect a balance between risks, hazards and costs. USBR TM3 states that the values are considered to be conservative and may be adjusted.

Table 18-3 General guide for determining emergency evacuation time (days)

Evacuation Stage	High-Hazard High-Risk	High-Hazard Significant-Risk	High-Hazard Low-Risk
75% Height*	10-20	20-30	30-40
50% Height*	30-40	40-50	50-60
25% Height*	40-50	50-60	60-70
10% Storage**	60-80	70-90	80-100

Note: Table reproduced from Table 4 in USBR TM3

*For Lee, the height is considered to be measured from the NTWL to river bed level

** For Lee, the storage is considered to be between NTWL and river bed level

The Lee Valley Dam outlet facility has two distinct draw off levels. These can be adjusted by removing or adding pipes on the face of the dam using divers. To achieve a minimum draw off level, it is possible to disconnect the inclined pipes completely, connecting the screens directly to the thrust block above the starter dam.

For the purpose of evaluating the filling scenario, an outlet rating curve was developed assuming the two distinct draw off levels shown on the design drawings. To evaluate a reservoir evacuation scenario, the outlet rating curve assumes the outlet pipes are removed to achieve the maximum drawdown possible. Both scenarios assume:

- The pipework within the concrete conduits and the fixed cone valves (FCVs) remain in place to control the outflow
- The discharge rate through each of the 800mm diameter FCVs is limited to a maximum of 7.5 m^3/s so velocities through the valves are kept within manufacturer's recommended limits.

The Lee Valley Dam design inflows are largely based on the 52 years (1957 to 2009) flow record of the Wairoa at Gorge/Irvine gauge. This Wairoa record has been scaled to produce a synthetic record for the Lee Valley Dam based on correlations with the new Lee River gauge above Waterfall Creek (established in April 2007).

An analysis of the synthetic inflows has been carried out to determine the dam's mean monthly inflows and the highest consecutive mean monthly inflows. Table 18- presents a summary of the monthly data over the 52 year record.

Table 18-4 Lee Valley Dam synthetic record monthly inflow summary

Month	Mean Monthly Inflow (m^3/s)	Maximum Monthly Inflow (m^3/s)
Jan.	2.7	13.1
Feb.	2.0	9.2
Mar	2.5	11.1
Apr	3.5	14.8
May	3.4	10.8
Jun	4.1	10.7
Jul	4.3	17.2
Aug	4.2	13.2
Sep	4.7	15.7
Oct	4.6	15.1
Nov	3.8	10.9
Dec	3.3	14.3
NOTE: Monthly flows derived from synthetic daily record		

18.2.1 Reservoir filling

Routing was carried out to assess reservoir filling rates and determine a design frequency storm to apply during filling. The two lowest consecutive mean monthly inflows occurred in August 1997, with a mean inflow value of 3.6 m^3/s. Routing shows the reservoir could fill in less than two months using this rate with an allowance of 0.51 m^3/s environmental release. Based on this relatively short filling duration, the synthetic design MAF hydrograph (without climate change) was adopted as the frequency storm to apply during filling.

The reservoir filling routing analysis assumes:

- The inclined pipework on the dam face is in place with two distinct draw off levels as shown on the design drawings
- The outflow is limited to 15.1 m^3/s to keep the velocities through the FCVs to within the manufacturers' recommended limits.

Routing a MAF event during filling results in a maximum reservoir level maintained below 90% reservoir depth (75% storage) assuming a hold point of around one third the reservoir depth (just above the elevation of the low level intake). The reservoir is able to be lowered back down to the hold point level in around 27 days with the upper level intake operating for around 6 days.

USBR TM3 recommends that the outlet works should have sufficient discharge capacity to maintain the reservoir levels reasonably constant for elevations above 50% of the reservoir depth for the established inflow conditions. At 50% reservoir depth only the low level outlet is available for release. However the discharge capacity of the low level outlet is

greater than the mean monthly inflows presented in Table 18-, thus we believe the outlet works will pragmatically meet the objectives of USBR TM3.

18.2.2 Reservoir evacuation

The highest consecutive mean monthly inflows were determined by finding the maximum of a two month and three month moving average. The record shows that highest two month inflow period starts in October 2001, with a mean inflow value of 14.0 m^3/s. The highest three month inflow period also starts in October 2001, with a mean inflow value of 11.8 m^3/s.

Table 18- presents routing results for reservoir evacuation assuming mean monthly inflows. The results show that the dam could be dewatered in around 15 days assuming mean monthly inflows.

Table 18-5 Reservoir evacuation - mean monthly inflows

Evacuation Stage	Time (days)
75% Height*	7
50% Height*	12
25% Height*	15
Minimum (21% Height*)	15
10% Storage**	13

*Height is measured from the NTWL to river bed level
**Reservoir storage between NTWL and river bed level

Table 18- presents routing results for reservoir evacuation assuming the highest two consecutive mean monthly inflow (14.0 m^3/s). The routing analysis for this scenario assumes inflow goes back to the mean monthly inflow at the end of the two month period. The results show that the dam could be dewatered in around 68 days if the outflow is limited to 15.1 m^3/s. Comparing this to the USBR TM3 guidelines for a High-Hazard Significant-Risk dam, only the time to 25% Height criteria is met. However, based on an EV1 distribution, the highest two consecutive mean monthly inflow scenario equates to an event with a 90 year ARI. Therefore there is a relatively low probability that this scenario could occur during a dewatering. Further discussion is presented below in relation to routing with the FCVs fully open (Refer results in Table 18-).

Table 18-6 Reservoir evacuation - highest two consecutive mean monthly inflows with limited outflow

Evacuation Stage	Time (days)
75% Height*	61
50% Height*	66
25% Height*	68
Minimum (21% Height*)	68
10% Storage**	66
*Height is measured from the NTWL to river bed level **Reservoir storage between NTWL and river bed level	

Table 18- presents routing results for reservoir evacuation assuming the highest three consecutive mean monthly inflow (11.8 m^3/s). Based on an EV1 distribution, the highest three consecutive mean monthly inflow equates to an event with a 100 year ARI. The routing analysis for this scenario assumes inflow goes back to the mean monthly inflow at the end of the three month period. The results show that the dam could be substantially dewatered (50% Height or 10% Storage) in around 39 to 41 days if the outflow is limited to 15.1 m^3/s. We consider this to be consistent with USBR TM3 guidelines for a High-Hazard Significant-Risk dam.

Table 18-7 Reservoir evacuation - highest three consecutive mean monthly inflows with limited outflow

Evacuation Stage	Time (days)
75% Height*	23
50% Height*	39
25% Height*	90
Minimum (21% Height*)	90
10% Storage**	41
*Height is measured from the NTWL to river bed level **Reservoir storage between NTWL and river bed level	

Faster dewatering in an emergency could be achieved if necessary by either:

- Exceeding recommended velocity limits in the FCVs and accepting they may be damaged (Refer Table 18-)
- Removing the FCVs completely (provided the upstream gate valves are operational and enable the FCVs to be removed) and accepting less flow control and the potential for increased downstream erosion.

Table 18- presents results for the same scenario as Table 18- (highest two consecutive mean monthly inflows) but assuming the FCVs are fully open. The results demonstrate that the dam could be substantially dewatered in around 18 to 23 days but velocity limits on the

valves will have been exceeded for virtually the entire period. We consider this scenario to meet USBR TM3 guidelines for a High-Hazard Significant-Risk dam.

Table 18-8 Reservoir evacuation - highest two consecutive mean monthly inflows FCVs fully open

Evacuation Stage	Time (days)
75% Height*	8
50% Height*	18
25% Height*	61
Minimum (21% Height*)	61
10% Storage**	23
*Height is measured from the NTWL to river bed level **Reservoir storage between NTWL and river bed level	

19 Roading

19.1 Introduction

Permanent road access is to be provided to the dam crest as well as to the outlet works at the toe of the dam. Access to the forest block is to be maintained as far as practicable during construction and after completion of the dam. There will be periods when this access is restricted while the new roads are being formed. We recommend that the WWAC communicate this to the forestry operator. Access to the crest and toe of the dam will require two bridges across the spillway.

The roads that have been designed for this project are as follows:

Upper bypass road - this is an existing forestry track that will be affected by the spillway cuts. We have realigned this road. A possible cost saving measure would be for this road to be constructed by the forestry operator either on this alignment, or elsewhere.

Crest access road - this is the realignment of the existing forestry access road that links the Lee Valley Road to the upper reservoir along the true left of the Lee river. The road also includes a turn off over the ogee weir and onto the dam crest itself.

Dam toe road - This is a new access track to provide access to the toe of the dam. This includes a bridge over the flipbucket.

Figure 19.1 shows the three access tracks that have been designed.

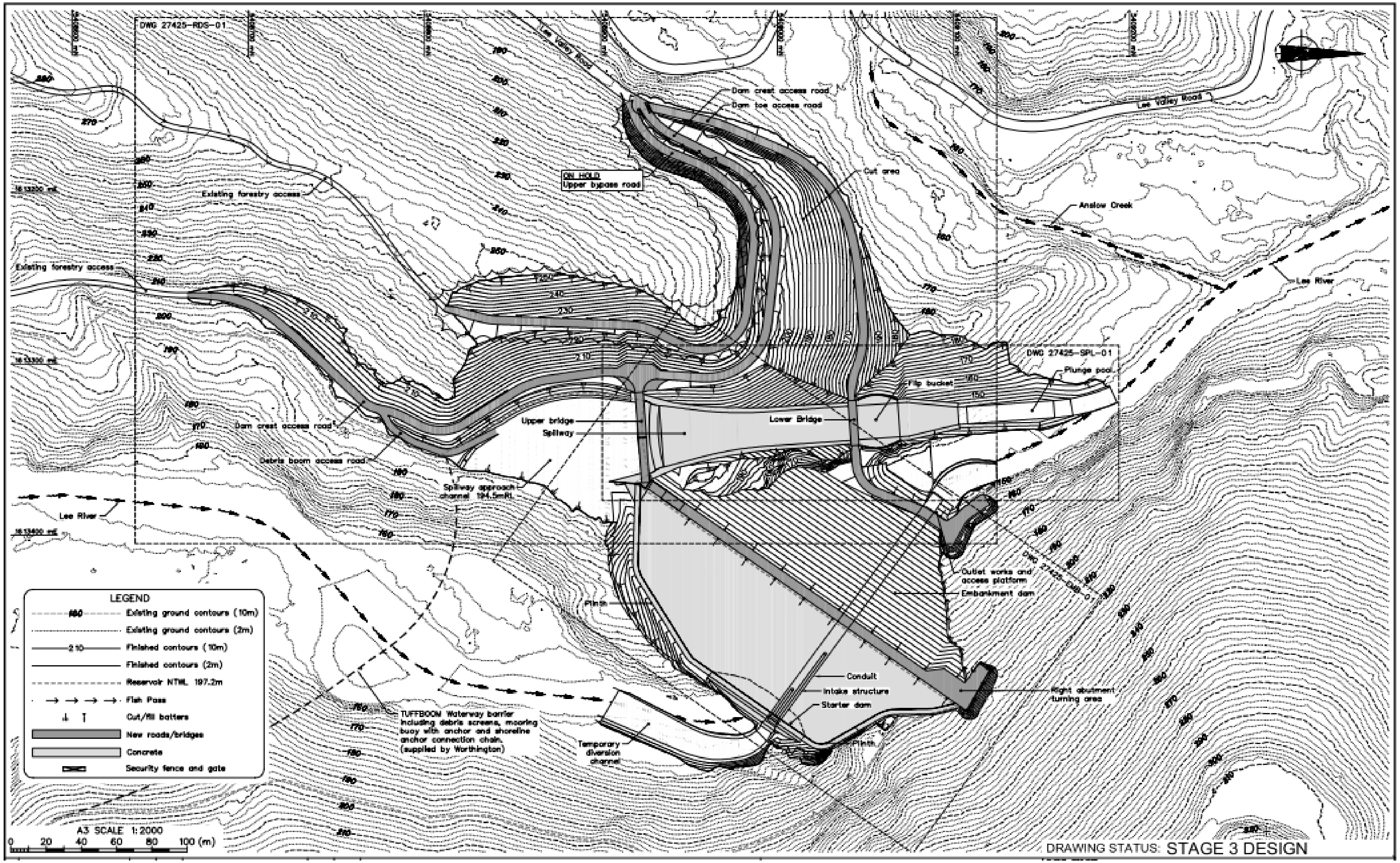


Figure 19.1 - Permanent access tracks shown shaded solid grey (Refer to Drawing 27425-GEN-11)

19.2 Site access

The main access to the site for all construction traffic is from Lee Valley Road, approximately 13.6 km south of the River Terrace Road/Lee Valley Road intersection in Brightwater. While access to the forestry block is to be maintained during construction, access to the dam should be controlled by appropriate security gates and fences. Any such fences and security are outside T&T's design scope.

T&T's design scope does not extend to the site access along Lee Valley Road. We do however highlight that construction access is critical to the successful construction of the dam. Our assessment does not allow for upgrade, maintenance or providing alternate access to the dam site for construction or for the permanent case. Fletcher has brought this risk to our attention and therefore the WWAC should consider this aspect to prevent delays to the construction of the dam from occurring.

19.3 Design criteria

The permanent road carriageway width has been set at 4.5 m to accommodate a 6 wheel, 11 m rigid truck (8.2 tonne standard axle). The typical road cross section also allows for a 0.5 m wide table drain when in cut and a 1 m wide shoulder when in fill.

The carriageway width will allow for one-way traffic for standard construction vehicles with occasional passing opportunities on straights. There are no specific passing bays designed, but there are turn-around areas at the right abutment on the dam crest and at the outlet works at the toe of the dam. There is also local widening at the intersection of the dam crest access road and the upper bridge to facilitate turning onto the bridge.

The selection of design parameters for vertical and horizontal alignments has been based on general guidelines for construction traffic and on the existing forestry access roads which the dam access roads replace.

The existing forestry access road widths vary. However, adjacent to the location of the dam crest the existing carriageway is approximately 4.5 m wide.

The maximum desirable longitudinal road gradient has been set at 15% (1 in 6.7) with an absolute maximum gradient of 16.7% (1 in 6). This gradient is consistent with the gradients already in use around the dam site and proposed reservoir.

The minimum desirable horizontal radius has been set at 27.5 m with an absolute minimum radius of 17.5 m. A summary of the design parameters is shown in Table 19.1.

Table 19.1: Road design parameters

Criteria	Value	Comment
Road width	5 m wide subgrade with 4.5 m wide completed carriage way with 0.5 m table drain when in cut and 1 m shoulder when in fill.	
Horizontal curvature	Minimum desirable internal horizontal radius 25 m Absolute minimum radius of 15 m where there is no physical restriction on the inside of the bend.	
Vertical curvature	Minimum vertical radius 120 m	K = 1.2
Longitudinal grade	Preferred maximum 15% (1:6.7) Absolute maximum 16.67% (1: 6)	Over 15%, additional pulling capability required and/or pavement improvements such as sealing may be required – particularly for transport of hydro equipment. Not included in the design.

19.4 Pavement design

During construction of the dam, the typical running surface will be 150 mm of compacted GAP65. Once construction is completed, an additional 100 mm of GAP40 will be placed on the permanent access roads. Only the dam crest will be asphalted or chipsealed.

19.5 Drainage design

The capacity of the road drainage system has been designed for the 5 year rainfall event. Rainfall data is sourced from NIWA HIRDS v3.0 and runoff is calculated using the rational method. Table drains on the roads have been designed, 0.5 m deep and 1 m wide. Runoff is discharged from the drains through culverts in the road. The culverts have been sized for the 5 year rainfall event assuming that water can head up to the top of the table drains. Events larger than this may result in surface water flowing over the roads.

The vee drains are expected to be excavated in mostly Class 2 rock and therefore scour is not expected to be significant. This should be monitored during operation and remedial measures installed if required.

Because the road surface is not to be paved, overland flow will wash the road surface away if the culverts are not maintained.

20 Permanent power supply

Permanent power supply for control of valves, telemetry, pumps etc. is outside the scope of T&T's design. It is assumed that the WWAC will provide power to the site as required.

21 Reservoir clearance

Cawthron (2009) recommended that for water quality reasons the reservoir, dam site, borrow areas, spoil disposal areas and contractor site compound are clear felled of trees and vegetation and that debris is removed from the same areas. We endorse forest and debris removal as a priority, as there is otherwise the risk that the dam could be damaged by debris during construction and river diversion.

An exception to forest clearance is where there are trees that currently cover possible landslides. The geotechnical investigations (Appendix F) conclude that removing trees above reservoir level on landslides may reduce the stability of the landslides. Therefore the trees above reservoir level on the landslides identified in Appendix F should remain insitu.

We note that reservoir clear-felling and debris removal are not part of T&T's scope of work, and have not been assessed.

22 Contractor design works

In design and construction of projects it is common for certain aspects to be designed by a supplier or the contractor. The reasons for this are:

1. A contractor often has a preferred method of working that the designer cannot predict.
2. Many products are available "off the shelf". These are commonly referred to as proprietary items. The manufacturer of these items (for example a pipe valve) will have a standard design and can provide guarantees or warranties for the valves' performance. It would be uneconomical for the client to have the dam designer to design the valve specifically for the dam. Therefore the designer specifies a product that meets the operating requirements (e.g. design pressure and flowrate) and the contractor sources this from a range of suppliers, obtaining the best price for the specification.
3. Temporary structures such as concrete formwork, scaffolding, haul roads are normally designed by the contractor to meet their specific construction requirements.

This section describes the significant contractor design element of the works. In developing full engineering details there will likely be other less significant items that will be contractor design items. These will be specified in the Stage 4 design. Table 22.1 summarises the main contractor design elements.

Table 22.1 Contractor design elements (not exhaustive)

Location	Item
General	Craneage and crane platforms
	Haul and access roads
	Borrow and spoil disposal areas
	Contractor power supply
	Any proprietary item
	Fish pass pump
	Debris boom and anchor blocks
	Temporary slope protection (cut or fill)
	Erosion and sediment control measures
	All temporary works
Diversion	Debris protection
	Coffer dam
	Mesh protection
	Height and extent of quickrise bund
	Diversion wall
	600 dia diversion pipe and inlet
	Temporary slope protection
	Temporary inlet stoplogs
Outlet works	Gantry crane in conduit
	Valves
Bridges	Bridge bearings
	Temporary stability during construction
	Beam splices
Concrete works	All formwork and falsework
	Kerb under the dam concrete face
	Parapet wall if pre-cast

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24 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

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Appendix A: Stage 3 design drawings (Bound separately)

Appendix B: Peer review letters

Appendix C: Minutes of HAZOP workshop and Contract Procurement workshop

Appendix D: Draft Emergency Action Plan (EAP)

Appendix E: Draft Operational Maintenance and Surveillance Manual (O M & S)

Appendix F: Geological Interpretive Report (Bound separately)

Appendix G: M&E Design Report