REPORT

Hawke's Bay Regional Investment Company Limited

Ruataniwha Water Storage Scheme
Project Description

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

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Report prepared for:

Hawke's Bay Regional Investment Company Limited

Report prepared by:

Tonkin & Taylor Ltd

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1 Introduction

This document provides a description of the proposed Ruataniwha Water Storage Scheme ("the Scheme") in Central Hawke's Bay. The Scheme essentially involves surface water harvesting, storage and distribution for servicing irrigable land principally located on the Ruataniwha Plains.

The purpose of this document is to describe all key Scheme elements in order to provide the necessary base information for consideration of the resource consent applications and notice of requirement which this report will form part of, and for assessing the environmental effects of the Scheme. The document includes basic details for the principal infrastructure, construction methods and the proposed operating regime of the Scheme.

The document has been compiled by Tonkin & Taylor Ltd with input and assistance from a wider team of planning, engineering and science personnel engaged by Hawke's Bay Regional Investment Company Limited ("HBRIC Ltd") to develop the Scheme for consenting purposes.

1.1 Design status and project optimisation

The Scheme elements, construction methods and proposed operating regime outlined in this report are derived from the Tonkin & Taylor "Ruataniwha Plains Water Storage Project Feasibility Project Description" (August 2012a). That report was prepared at the end of the "Feasibility Stage", prior to an 'in principle' decision being made to progress the Scheme through the Resource Management Act 1991 ("RMA") consenting process. This report describes the Scheme as refined since August 2012, including changes resulting from further optimisation work signalled in the Feasibility Project Description. In addition, further work has progressed in developing the Scheme concept for what is described as "Zone M" throughout this report.

For the purpose of this report, the term "Application Design" is applied.

HBRIC Ltd as promoter of the Scheme will be undertaking further detailed design in parallel with the RMA consenting process. The final engineering details of some project elements may therefore differ from the Application Design described in this report. Resource consent conditions will be proposed by HBRIC Ltd that require the Scheme to be constructed and operated generally in accordance with the description in this document but with the flexibility to make changes to the Application Design which do not give rise to materially greater or different effects than as assessed in the resource consent applications and notice of requirement. For example, the conditions would allow some flexibility associated with the main dam design and associated appurtenant structures and works, provided the dam stability and structural integrity is maintained.

Furthermore, and for a project of this scale and complexity, it is considered both reasonable and appropriate to accommodate improvements to the Scheme elements, construction approach and proposed operating regime as described in this report within the overall process to implementation and completion. This is particularly important where such improvements will enable the economic and environmental objectives of the Scheme to be delivered more efficiently or effectively; and opportunities of that kind may be revealed through the detailed design phase. In such instances, the assessment of effects of the Application Design in the resource consent applications and notice of requirement documents (including associated technical reports) would be conservative. The objectives, environmental requirements and environmental controls in the proposed consent conditions would need to be achieved for any proposed improvements.

Improvements of this nature are referred to in this report as "Optimisation". Likely areas of Optimisation within the Scheme are addressed throughout this report relative to the Application Design. They are summarised in Table 1.1.

Table 1.1 Areas of future optimisation

Item	Nature of possible optimisation and description	Section Reference
Dam		
Dam axis (dam alignment along the crest centreline)	The dam axis may be optimised during detailed design to suit the dam type, layout of the appurtenant structures, available construction materials and further investigations for excavation/foundation design.	Section 4.1
Dam type	The Application Design is based on a concrete faced rockfill dam. Consideration may be given to alternatives during detailed design. Any proposed alternative will need to meet accepted dam stability and structural integrity requirements.	Section 4.1
Borrow sources	The suitability, location and extent of borrow sources within the areas defined in the resource consent applications will be confirmed during detailed design.	Section 4.7.2
Spoil disposal locations	The suitability, location and extent of spoil disposal areas within the areas defined in the resource consent applications will be confirmed during detailed design.	Section 4.7.2
Dam batter slopes	The batter slopes of the dam will be optimised during detailed design, consistent with accepted dam stability and structural integrity requirements.	Section 4.4.3
Dam crest elevation and design	The level of the dam crest and its general arrangement will be optimised during detailed design, in conjunction with the primary and auxiliary spillways while complying with minimum freeboard requirements.	Section 4.4.2
Primary spillway location and design	The primary spillway location and general arrangement may be optimised during detailed design.	Section 4.4.4.2
Auxiliary spillway location and design	The requirement for the auxiliary spillway will be reviewed during detailed design. If confirmed, the auxiliary spillway location and general arrangement may be optimised.	Section 4.4.4.3
Coffer dam(s)	The location, type and height of the coffer dam(s) within the defined construction footprint will be optimised during detailed design.	Section 4.5
Diversion tunnel(s)	The location, length and diameter of the tunnel(s) within the defined construction footprint will be optimised during detailed design.	Section 4.5
Intake/outlet	The Application Design incorporates two towers with multiple intakes along with provision for aeration. The intake and outlet arrangements may be optimised during detailed design. Any proposed alternative will need to be consistent with or better than the modelled Scenario M1 (NIWA 2013) involving a low level intake only in terms of maintenance of reservoir water quality.	Section 4.4.5

Power station	The location and general arrangement of the power station and associated facilities within the defined construction footprint will be optimised during detailed design.	Section 6.2
Upstream landslide stabilisation	The requirement for and extent of landslide stabilisation will be reviewed during detailed design.	Section 4.2
Replacement roading and alternative access provision	Alternative access arrangements have been shown and used as a basis for community consultation. Optimisation during detailed design will consider whether there are other arrangements that better meet landowner and stakeholder requirements.	Section 4.8
Flow regime	The opportunities for optimisation of the downstream flow regime will be reviewed during detailed design	Section 3.0
Water Distribution Network		
Upstream water intake	The infiltration bund and gallery pipework will be reviewed during detailed design along with by-wash provisions.	Section 5.4
Primary distribution system headrace canal	The length of primary distribution system may reduce as a result of optimisation during detailed design. The optimisation process may also result in the length of headrace canal reducing with a corresponding increase in the length of primary pipeline. The route alignment will be optimised within the corridor defined in the Notice of Requirement during detailed design in consultation with landowners. The batter slopes of the canals will be optimised to balance the earthworks quantities where possible.	Section 5.5 and 5.6
Secondary distribution network	The network layout will be optimised during detailed design to meet end user locations.	Section 5.7
Downstream water intake	The position and arrangement of the Zone M intake within the area identified in the resource consent applications will be confirmed during detailed design.	Section 5.8
Primary Distribution System - Zone M	The Application Design involves using the Papanui Stream (Old Waipawa River) channel to convey water. An alternative piped option may be considered during detailed design.	Section 5.9

1.2 Report structure

Following this Introduction, the report is structured as follows:

Section 2	-	Scheme Overview Presents a summary of the purpose of the Scheme and a summary of the physical components of the proposed scheme.
Section 3	-	Reservoir Operating Regime Sets out proposed environmental flows, the practical maximum live storage and dead storage allowance, and describes the operating regime of the proposed dam and reservoir, including the dam outflow regime.
Section 4	-	Dam Design Summarises the proposed dam arrangements including design criteria, freeboard, spillway, seismic issues and outlet works arrangements; Dam construction programme and construction activity issues are also discussed.
Section 5	-	Water Distribution Network Summarises design criteria, primary distribution system hybrid option, section and alignment, river intake arrangements, intake sediment management, and secondary distribution arrangements.
Section 6	-	Hydroelectric power generation Provides a summary of a hydroelectric power generation plant at the base of the dam.
Section 7	-	Integrated Mitigation and Offset Approach Presents a summary of the four key mitigation projects.
Section 8	-	Remediation of sediment loss to coast Presents a summary of the proposed strategy to mitigate loss of in-river sediment to the coast due to the construction of the dam.

Appendices at the end of this report contain:

- Drawings and figures
- Preliminary construction programme
- Plans of the proposed primary distribution system for Zone M
- A figure showing the approximate extent of long term reservoir erosion

1.3 Glossary of terms and abbreviations used

Term	Definition
320 HP	A type of bulldozer (320 Horsepower)
6t travelling crane	6 tonne capacity crane
7-day annual minimum flow	Refer to 7-day MALF
7-day MALF	7-day mean annual low flow (lowest average flow over 7 consecutive days in a year, averaged over the number of years considered)
AEP	Annual Exceedance Probability (probability of a specified event, e.g. flood flow, being exceeded in any one year)
ANCOLD	Australian National Committee on Large Dams
ARI	Average Recurrence Interval
AS/NZS	Australian/New Zealand Standard
Baulk	Gate/stoplog
Bidim A44	Type of geotextile
Bifurcates/bifurcation	Split into two
CAT 966	A light wheel loader
CCTO	Council Controlled Trading Organisation
CDA	Canadian Dam Association
CEMP	Construction Environmental Management Plan
CFRD	Concrete Faced Rockfill Dam
CHBDC	Central Hawke's Bay District Council
CHBDP	Central Hawke's Bay District Plan
CTMP	Construction Traffic Management Plan
D&C	Design and Construct
Dam	The Proposed Makaroro Dam (on the Makaroro River) or RWSS Dam also referred to as "the dam", "storage dam", "A7" or "the Makaroro Dam". Includes all facets of the dam and associated structures
Dead storage	Portion of reservoir volume unavailable for use (below the bottom intake level)
DWI	Downstream Water Intake. The point and structure at the lower end of the Waipawa River where water released from the dam is taken to supply Zone M
EAP	Emergency Action Plan
EOI	Expressions of Interest
EPA	Environmental Protection Authority
ESCP	Erosion and Sediment Control Plan
FCD	Fixed cone dispersion (a type of valve)
FERC	United States Federal Energy Regulatory Commission
Flushing flow	Flow required to remove algae build up in river
FSL	Full Supply Level

GAP65	Aggregate for road building
GXP	Grid exit point
Hawke's Bay Datum	Local level datum (equivalent to LINZ +10 m)
HBRC	Hawke's Bay Regional Council
HBRIC Ltd	Hawke's Bay Regional Investment Company Limited
HDC	Hastings District Council
HDP	Hastings District Plan
HDPE	High density polyethylene
Headrace	Also referred to as "headrace canal" as part of the PDS
HN/HO loading	Normal/Overload loading for bridge design (Refer to NZ Transit Bridge Manual for definitions)
IEMP	Irrigation Environmental Management Plan
ICE	Institution of Civil Engineers (UK)
ICOLD	International Commission on Large Dams
LHS/RHS	Left/Right hand side (typically looking downstream)
LiDAR	Light Detection and Ranging – a method of surveying ground topography
LINZ	Land information New Zealand
Live storage	Portion of reservoir volume available for use (e.g. available for irrigation, residual flow etc.)
M4	Aggregate for road building
MALF	Mean Annual Low Flow
MDF	Maximum Design Flood
NB	Nominal bore
NIWA	National Institute of Water and Atmospheric Research
NZSOLD	New Zealand Society On Large Dams
OBF	Operational Basis Flood
OHT	Overhead travelling (gantry) crane
OM&S	Operation, Maintenance and Surveillance
Overseer	A computer model that calculates and estimates the nutrient flows in a productive farming system and identifies risk for environmental impacts through nutrient loss, including run off and leaching, and greenhouse gas emissions
PC6	"Plan Change 6" or the Tukituki River Catchment Plan Change. A proposed plan change which inserts Tukituki Catchment specific objectives, policies and rules into the RRMP
PD	Project Description. Provides a description of the proposed Ruataniwha Water Storage Scheme.
PDS	Primary Distribution System. Comprises of the primary headrace canals and pipelines that are connected to the intake structures that provide water to the SDS, see the PD for a full description
PIC	Potential Impact Category
PLUA	Production Land Use Areas. Areas of irrigable land suitable for primary production within the schemes five command zones

PMF	Probable Maximum Flood
Q2 gravel	Pleistocene aged sediments (typically gravels) of Q2 (older than Q1)
RCRRJ	Reinforced concrete rubber ring jointed (pipe)
Reservoir	The body of water stored behind the dam on the Makaroro River
RL	Reduced level (height above datum)
RMA	Resource Management Act
ROR	Reservoir Operating Regime. Describes the procedure, methodology and system for operating the dam relative to reservoir and river water levels for the storage dam to service the overall scheme
RPS Change 5	Regional Policy Statement Change 5 "Land use and freshwater management".
RPS	The Hawke's Bay Regional Policy Statement (which forms part of the RRMP)
RRMP	The Hawke's Bay Regional Resource Management Plan (generally referred to as the "regional plan")
Rubicon	Hydraulically actuated slide gates
RWSS	Ruataniwha Water Storage Scheme (also referred to as "the Scheme" or "RWS Scheme")
SDS	Secondary Distribution System. Comprises of a network of pipelines that are connected to the PDS that deliver water to properties i.e. "the farm gate", see the PD for a full description
SCEMP	Supplementary Construction Environmental Management Plan
Sympathetic foundation movement	Tectonic movement triggered by an adjacent fault movement
Sz1	Shear zone (zone of rock mass weakness)
The Scheme	Ruataniwha Water Storage Scheme
Total/gross storage	Sum of dead and live storage
TRIM	Tukituki River Model. A computer model incorporating models, environmental data and GIS used to manage nutrient inputs , land use and water quality
True right/left bank	The bank of the waterway looking downstream
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
UWI	Upstream Water Intake. The point and structure at the upper end of the Waipawa River where water released from the dam is taken to supply Zones A, B, C and D
WDN	Water Distribution Network. Comprises of the PDS and SDS, see the PD for a full description
Zone	An area labelled A, B, C, D or M that the scheme is providing water to. Also referred to as "command zones". The word zone is also used in relation to zones in planning maps in the CHBDP and HDP and can be used in different contexts.

2 Scheme overview

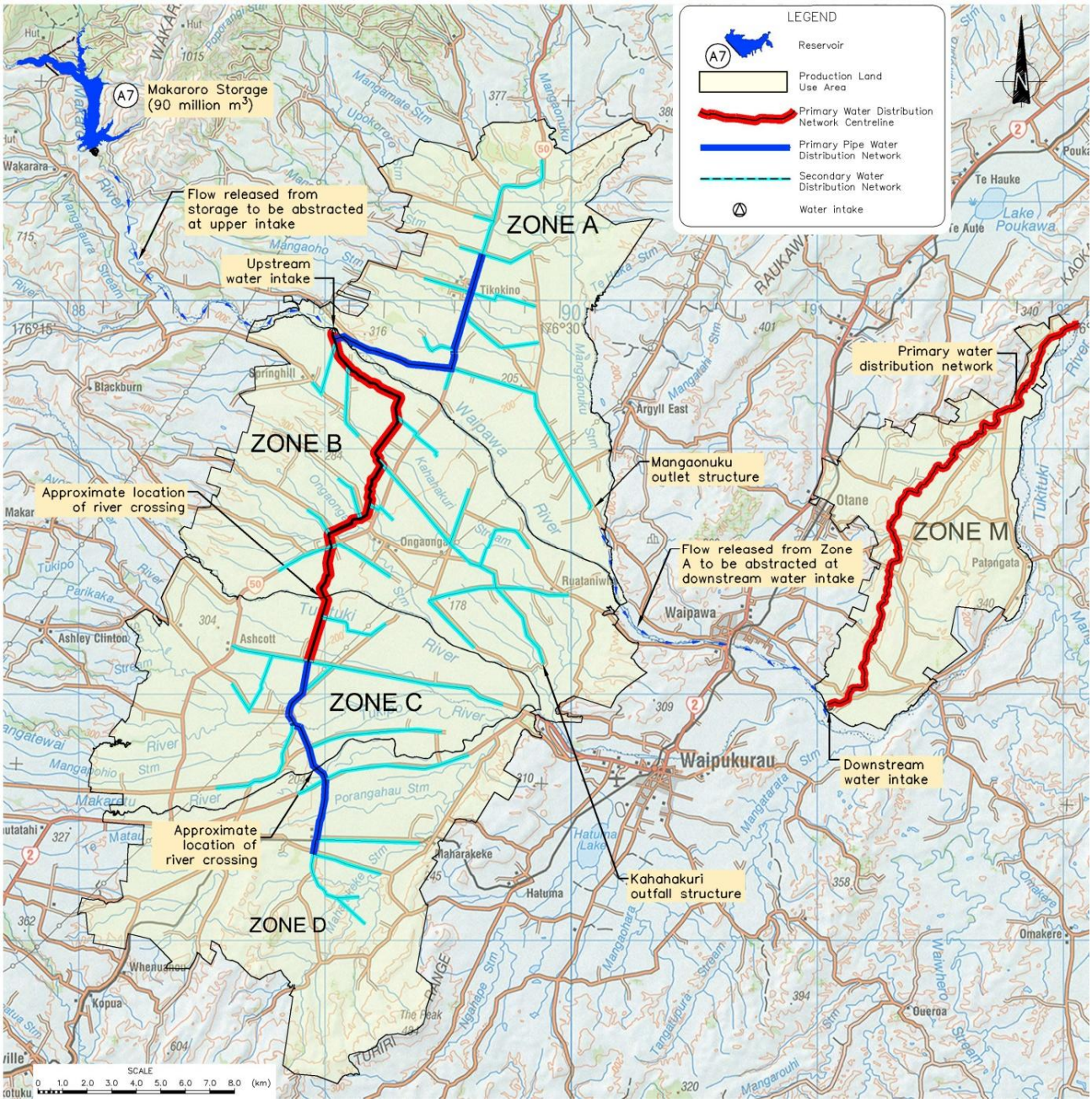


Figure 2.1 Scheme overview showing dam location, reservoir extent, production land use areas, and the proposed water distribution network.

2.1 Dam design

The proposed dam site is located on the upper Makaroro River approximately 1 km east of Wakarara Road and 6.4 km northwest (upstream) of the confluence of the Makaroro and Waipawa Rivers. The dam is located approximately 1 km from the Mohaka fault which also crosses the proposed dam reservoir.

The proposed total water storage requirement of 90 million m^3 at the site results in a dam of approximately 83 m height at the river's deepest point.

The Ruataniwha basin is located within an area subject to significant earthquake shaking and where the geological formations are susceptible to deformation during such events. As a result, a number of geological and geotechnical issues were identified during the feasibility studies that complicate the design. Alternative dam sites within the Ruataniwha basin were considered but were found to have similar or worse foundation issues than the Makaroro Dam site. Following a dam type option assessment that concentrated on the dam's ability to accommodate foundation deformation as a result of extreme seismic events, and the use of locally available construction materials, a Concrete Faced Rockfill Dam (CFRD) was selected during the Feasibility Stage as the recommended dam type for this site. Application Design arrangements for a CFRD type have been developed including zoning, spillways, diversion tunnel, and intake towers, as described in more detail in Section 4 of this report and illustrated in Appendix A.

Due to the potential for sympathetic movement in the foundation during extreme earthquake shaking, an allowance has been made for filters and drainage measures on the left abutment over a zone of rock mass weakness identified during the site investigations. GNS Science has advised that the maximum single event displacement within the dam footprint would be 0.5 m (Langridge et al, 2011b). The Application Design accommodates this movement safely; albeit repairs will be required should this movement occur during the operational life of the dam.

The Application Design allows for irrigation and flushing flows to be drawn from multiple levels within the reservoir through an intake tower into a penstock located in a tunnel beneath the dam. This is an area of potential optimisation as discussed in Section 3. A hydro power station located at the toe of the dam will harness energy before releasing the water into a tailrace canal excavated to the true right of the existing river. A concrete lined spillway will take flood flows through the right abutment and discharge into a stilling basin that joins the tailrace. The discharge point to the river from the spillway is located to avoid a potentially unstable rock mass above the left bank downstream of the dam.

2.2 Water Distribution Network design

The Application Design water distribution network commences with the upstream water intake structure on the Waipawa River. This structure will collect the flows released from the proposed dam and distribute the water via the primary and secondary distribution systems, primarily for use within the production land use areas (Zones A to D and M) as shown on Figure 2.1. The primary distribution system is intended to provide driving pressure for the secondary distribution system where possible, although some areas will require pumping within the secondary pipeline network.

For Zones A to D as shown on Figure 2.1, the primary distribution system headrace canal alignment is constrained by the geography of the area (e.g. the foothills of the Ruahine Range) and the elevation of the upstream water intake site (which acts as a control for the maximum elevation of the primary distribution system). Following consideration of a number of options, an alignment has been selected that balances the advantage of maintaining a high elevation to reduce pumping costs, with an alignment that minimises the canal length and earthworks volumes.

The primary distribution system for Zones A to D is a hybrid headrace canal and pipeline that includes a combination of trapezoidal open channel generally where the design flows are greater than about 3 m^3/s, and buried pipes for lesser flows.

The secondary distribution system is a network of pipes that have generally been located in road reserves where possible, with a layout aimed to provide water to within 2 km of all farm gates within the defined production land use areas.

A tertiary pipe and pump system will be required for the on-farm supply. The required on farm pressures have not been specifically considered in the primary and secondary distribution system design outlined in this report although the Application Design presented in this report is based on some positive pressure being available at the farm gate.

The tertiary system pressure requirements will require further consideration during detailed design, following further consultation with the target water users.

The Application Design water distribution network for Zone M (refer Figure 2.1) commences with the downstream water intake structure on the Waipawa River, located immediately upstream of the confluence of the Waipawa and Tukituki rivers. This will collect flows released from an outfall located adjacent to the Mangaonuku Stream (within Zone A) and distribute the water via the Papanui Stream (Old Waipawa River channel) and a secondary distribution network for use within Zone M.

2.3 Operating regime

The dam's operating regime has been modelled using the GoldSim (Golder, May 2013) modelling platform to simulate the operation of the storage dam to harvest, store and release river flows to meet irrigation demand and an environmental flow regime. A number of "tranches" of water have been identified, which will be released from the dam within the limits of available water in the following order of priority:

- A primary residual flow at the toe of the dam equal to 90% of the 7-day mean annual low flow (7-day MALF)
- A primary flushing flow allocation of 2.0 million m^3. The size, timing and release triggers for the flushing flows are described in Section 3 of this report
- A primary irrigation volume of up to 95.8 million m^3, corresponding to a secure primary irrigation volume of 91 million m^3 delivered at the farm gate at full Scheme uptake. The 95.8 million m^3 accounts for the static storage of the dam and an in-season infill component
- A secondary flushing flow provision of 2.0 million m^3. The size, timing and release triggers for the flushing flows are described in detail in Section 3 of this report
- A secondary irrigation volume of up to 28 million m^3. This volume is to be used to supply water to existing irrigators and to the "spot market".

Section 3 of this report describes the reservoir operating levels, the volumes supplied within each of the above "tranches" of water, and the river flows in the Makaroro River downstream of the dam.

2.4 Hydroelectric power generation

Because the construction of the dam creates a large head difference on the river between the upstream and downstream faces, it is possible to harness the energy from the controlled release of water from the dam. A single power station at the foot of the dam is included in the Application Design.

Preliminary optimisation of the capacity of the power plant and costs against potential energy output has indicated an installed capacity of around 6.5 MW. The corresponding generation design flow is 9.73 m^3/s, consisting of 1.23 m^3/s for the residual flow turbine and 8.50 m^3/s for the main turbine, with both turbines housed in the same power station. An operational buffer of 0.20 m dedicated to hydropower operation, over and above the provision for irrigation storage, is included in the adopted Full Supply Level.

Power generated is proportional to the product of the generation flow and head, the latter of which is given by the water level difference between the reservoir and the river downstream. The height of

the dam and thus the maximum generating head is more or less fixed. The energy output is thus dependent on the size of the installed plant, specifically the maximum design generation flow and, to a lesser extent, the buffering storage (0.2 m) provided within the reservoir. When the reservoir is full or nearly full, this buffering storage allows partial capture of small floods and freshes (which would otherwise be spilled) for generation.

3 Reservoir operating regime

In this section, the following aspects are covered:

- A general description of the reservoir storage, river intake and proposed irrigation command areas
- Reservoir storage modelling approach, including key input data and assumptions
- The proposed operating regime of the dam and reservoir
- The modelling outputs, focusing on the reservoir level and flow regime below the dam.

An assessment of a basic reservoir operating regime was completed by Tonkin & Taylor Ltd (T&T) as part of the Feasibility Stage and described in the Feasibility Project Description (T&T, August 2012a). The reservoir operating regime described in the following sub-sections, including model development, scenario modelling and documentation, has been largely completed by Golder Associates Ltd (Golder). The modelling builds on earlier work by T&T and incorporates an enhanced environmental release flow regime and provision for additional irrigation, called secondary irrigation, which is less reliable than the main (or primary) irrigation supply considered in the Feasibility Stage.

3.1 Key elements

3.1.1 Dead storage allowance

The dead storage allowance caters principally for long term reservoir sediment infill. Nevertheless, before dead storage allowance is used up, there will effectively be extra water storage capacity in the reservoir.

The Sedimentation Assessment (T&T, May 2013) describes the sedimentation assessments and provides an estimate of the anticipated reservoir sedimentation over a nominal 100 year design life. The estimate at this stage is for between 15 and 26 million m^3 of sediment infill over 100 years, with the upper figure known to be too conservative and the lower figure potentially unconservative.

An allowance of 4 million m^3 was set aside for sediment storage for consideration of financial feasibility. Based on a mid-point estimate of 205,000 m^3 of sediment infill per year, this allowance is sufficient for approximately 20 years of accumulation. Over the proposed 35 year resource consent period the sedimentation estimate is 7.2 million m^3 (0.205 million m^3/year x 35 years). This approach also recognises that most of the reservoir space for sediment would be available over the first decade of operation and does not preclude advances in technology and economic incentives in the future to actively manage and extract trapped bedload in the reservoir.

Gravel is a valued resource in the Region and there is an existing rate of extraction activity within the Makororo/Waipawa/Tukituki catchment consistent with the level of sediment predicted to be trapped within the reservoir. The bathymetry of the reservoir would be surveyed over time to determine the need for any gravel extraction or other management actions to maintain the assumed live storage as discussed in the following section, and any resource consents needed for that activity (including on a commercial basis by existing operators) would be sought at the relevant time.

3.1.2 Reservoir evaporation and leakage

Net evaporation is a relatively minor component of the reservoir storage water balance. A preliminary assessment has indicated a maximum net evaporation loss (open water evaporation less direct rainfall on the reservoir surface) of around 380 mm over an irrigation season. This equates to about 1.0 million m^3 based on an average reservoir surface of around 2.6 km^2 over the critical storage drawdown cycle. Note that the full reservoir has a surface area of 3.7 km^2 (see Figure 3.1).

No specific allowance has been made for any leakage from the reservoir floor and seepage under and through the dam and abutments. While these losses are not anticipated to be excessive, it is

anticipated that a large proportion of these flow losses would emerge at the toe of the dam or a short distance below the dam and would count towards the environmental residual flow.

The downstream river flow will be monitored at the following locations:

- At the outlet works (including operation of the turbine and residual flow); and
- The spillway (indirectly via recorded reservoir levels).

During detailed design consideration will be given to monitoring downstream of the spillway to quantify any seepage through the reservoir floor and abutments so that any implications for the proposed environmental flow regime can be assessed.

3.1.3 Practical maximum live storage

The live storage capacity at the Makaroro Dam has been determined considering these factors:

- Crop water demand for irrigation
- Environmental flow releases for maintenance and enhancement of in-stream values
- Production land use areas to be serviced and level of service
- Inter-annual variability of the reservoir inflow volume and the level of drought security desired
- Hydropower generation requirements –as a secondary function of the storage dam
- Reservoir evaporation and leakage
- Conveyance methods, and their associated water losses, to the production land use areas.

There is a range of technical reports that describe the working and modelling assumptions associated with these factors in some detail. This section of the document does not repeat or record that analysis, as these reports will be included with the resource consent applications. Instead, it describes the key reservoir attributes of the Application Design arrived at through application of the various report outputs.

The total (live plus dead) storage volume is 90 million m^3 excluding the hydropower buffer volume of 0.7 million m^3. This live storage is a practical maximum which takes into account the following:

- 90 million m^3 of total storage corresponds with a useable live storage of 85 million m^3, once the provisions for dead storage (4 million m^3 – see Section 3.1.1) and reservoir evaporation (1 million m^3 – see Section 3.1.2) are deducted
- At 90 million m^3, the ponding footprint remains clear of the more environmentally sensitive gorge section of the Makaroro River
- Recognition of the “hydrological limit” of the dam catchment, i.e. a reservoir significantly greater than 90 million m^3 would be considered oversized compared with its refill capacity.

Figure 3.1 shows the reservoir storage-area-elevation relationship for the selected dam site (see Section 4). This relationship has been derived based on LiDAR data supplied by HBRC and incorporates modification to the existing ground surface from dam fill (main and coffer) and excavation of the approach channel to the tunnel intake. In order to provide a gross storage of 90 million m^3, the Full Supply Level of the reservoir is RL 469.27 m (to HBRC engineering datum). This estimate includes an additional allowance of 300,000 m^3 for further modification to the existing ground surface that takes up storage space. Allowing for an additional 0.2 m operational buffer for hydropower operation, a Full Supply Level of RL 469.5 m has been adopted.

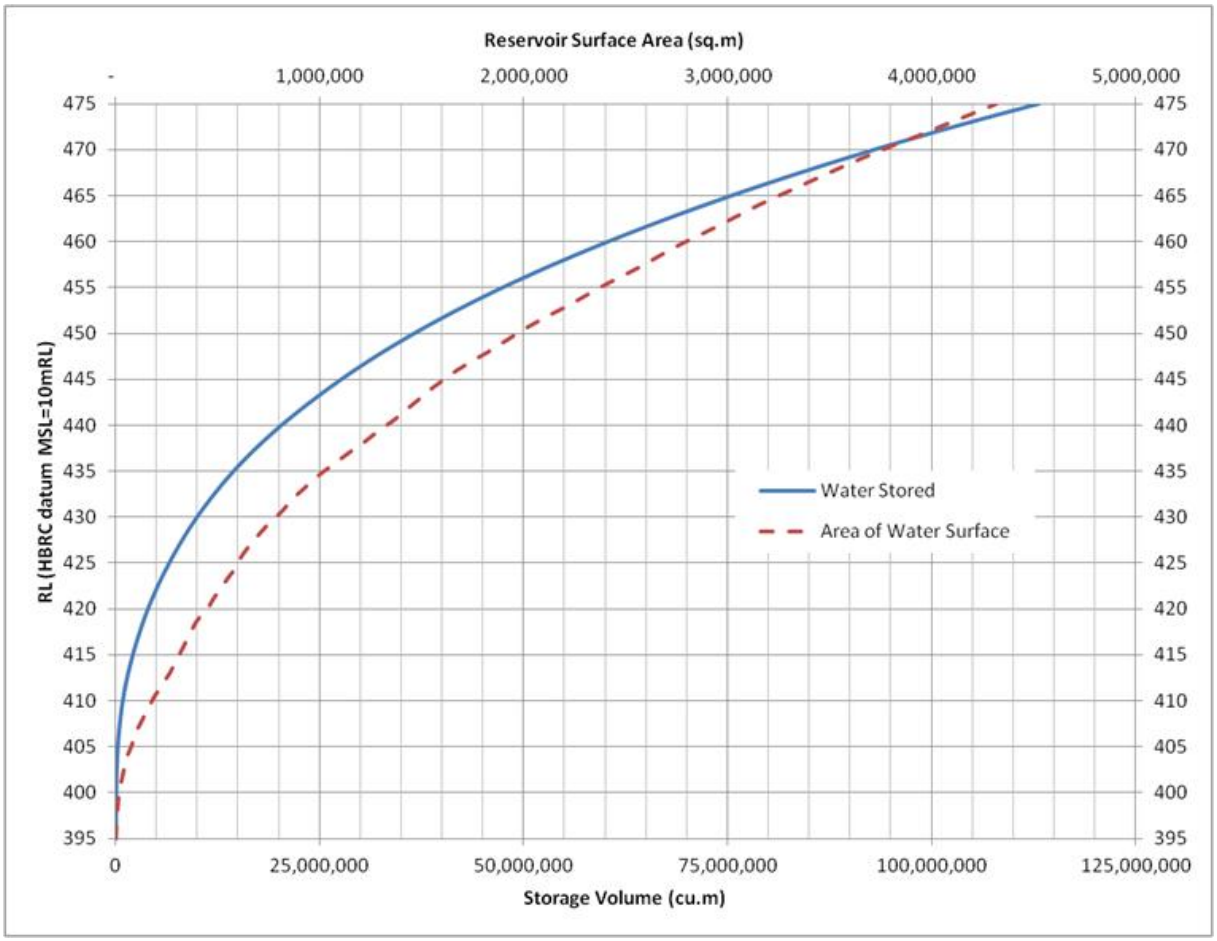


Figure 3.1 Storage – area – elevation relationship for the Makaroro Dam.

3.1.4 Operation of the Waipawa River intakes

Two intakes are proposed in the Application Design. The upstream water intake is located on the Waipawa River below its confluence with the Makaroro River to capture water released from the dam into the Makaroro River for distribution primarily to the production land use areas (Zones A, B, C, D and M, see Figure 2.1). The proposed location and arrangements of this intake are described in Section 5.4. It is noted that the river reach between the dam site and this intake is considered a "stable" or conservative reach i.e. neither "losing" nor "gaining" subsurface flow.

The water destined for Zone M will be carried by pipe through Zone A and released at the point where the Mangaonuku Stream joins the Waipawa River (Figure 2.1).

The downstream water intake is proposed on the Waipawa River approximately 1 km upstream of the confluence with the Tukituki River, to capture irrigation water and a 50 l/s residual flow for Zone M. The concepts supporting channel arrangements and residual flow for Zone M are detailed in EMS 2013.

There is a substantial catchment area of about 150 km^2, with an estimated mean flow of between 5 and 6 m^3/s, which contributes to the river below the dam site and above the upstream intake site. The great majority of this flow is from the upper Waipawa River (127 km^2). Thus, when flows are high in the upper Waipawa, there is a possibility for part or all of the irrigation demand to be met from Waipawa sourced flows, without having to release any irrigation flow from storage (although, flow releases would still be made at the dam to meet environmental flow requirements).

However, for the purpose of assessment of the Scheme, the conservative decision has been made not to depend on any of the flow from the intervening catchment between the dam and the intake (non-storage flow).

On this basis (i.e. no reliance on upper Waipawa water), the operation of the intake may be simplified thus:

> *Daily flow extracted at the upstream water intake = daily flow released from the dam (excluding any spill flow) minus the environmental minimum flow minus flushing flows*

This relationship would apply regardless of the actual flow arriving at the Waipawa intake.

3.1.5 Irrigation Demand

Assessment of the Scheme irrigation demand for the Feasibility Stage, including the rationale and main assumptions adopted for that assessment, is described in the Technical Feasibility study report (T&T, August 2012b). The main features and assumptions of that assessment are summarised as follows:

- The irrigation demand pattern is based on crop water demand for pasture production in the Ruataniwha Plains
- Daily irrigation demand at the root zone has been computed by Plant and Food Research Ltd using SPASMO for the soil types present based on historical climate data (rainfall and potential evapo-transpiration) for the period 1972 to 2010
- For the Feasibility Stage, the accumulated seasonal demand was capped at the 85th percentile seasonal demand volume (453 mm). This means that, on long term average, approximately once every 7 years the on-farm demand would exceed the capacity of the Scheme to supply (assuming full uptake); the highest seasonal demand in the 38 year modelled series (1997/1998) is 30% greater than the 85th percentile value
- The supply rate from the reservoir allows for an on-farm efficiency of 85% (i.e. 15% on-farm loss) and a conveyance efficiency of 95% from the reservoir to the farm gate (i.e. 5% transmission loss).

Reservoir storage modelling in the Feasibility Stage has shown that a reservoir with a gross capacity of 90 million m^3 (inclusive of the 4 million m^3 ring-fenced for sediment infill) can supply an irrigation volume of up to 95.8 million m^3 per season, as required, with a reliability of 19 out of 20 years, i.e. full supply security up to a 20 year return period drought. A minor shortfall would occur once every 20 years on long term average.

For assessment and description of the proposed Scheme, the irrigation demand pattern and volumes from the Feasibility Stage have been retained as the primary irrigation volume to be satisfied. However, the environmental release regime proposed for the Scheme has been refined and enhanced compared with the Feasibility Stage (refer Section 3.2.2). Secondary irrigation, which has a lower priority and thus reliability compared with the primary irrigation, is also proposed to be supplied from the Scheme. Therefore, irrigation reliability and the seasonal volumes able to be supplied from the storage dam (90 million m^3 gross capacity retained) are slightly different to that determined in the Feasibility Stage.

3.2 Reservoir storage modelling

This section focuses on the modelling of the reservoir in relation to the supply of irrigation water and environmental flows which are the primary functions of the proposed dam. The hydroelectric power plant and its functioning are described in Section 3.2.5.

3.2.1 Model Structure

The dam's operating regime has been modelled using the GoldSim modelling platform to simulate the operation of the storage dam to harvest, store and release river flows to meet irrigation demand and an environmental flow regime. A water balance model was developed for the proposed

Makaroro reservoir by Golder using GoldSim Pro (Version10.50) software. GoldSim is a graphical object-oriented modelling environment with the capacity to carry out dynamic probabilistic simulations. GoldSim has been applied successfully as a decision support tool for a range of water balance, water quality and water resource projects both within New Zealand and internationally.

The GoldSim water balance model covers the proposed Makaroro reservoir and extends to assess flows at two locations downstream of the dam (Waipawa River at RDS and the Tukituki River at Red Bridge). The model is a computer-based representation of the essential features of the expected hydrological system post commissioning of the Scheme. The model uses the laws of science, engineering and mathematics to represent the expected system.

The model represents the environmental system simulated to a level of detail suitable to achieve the intended objectives. In this case, the model provides a predictive scientific tool to quantify the effects of the proposed activities on flows in the Waipawa and Tukituki rivers downstream of the reservoir.

There are two main objectives of the Ruataniwha water storage GoldSim model:

1) To provide a water balance for the reservoir which appropriately models the performance of the proposed operating regime for the Scheme to meet irrigation demand, environmental minimum flows and flushing flows.
2) Allow compliance with key performance criteria (supply reliability for water users and achievement of environmental minimum flows and flushing flows) to be evaluated at key sites downstream of the reservoir.

The model has been used to guide development of the proposed operating regime outlined in this section of the report. As with any multi-variable model, solutions are not unique and various operating regimes could achieve the desired environmental outcomes. The adopted regime is aimed at being both practical and effective at meeting the desired environmental outcomes. It is anticipated that the operating regime will be further reviewed, adapted and optimised following detailed design, construction and commissioning of the scheme.

The basis of model development included two key components: a conceptual model and a numerical model. The conceptual model is an idealised representation (i.e. a picture) of our understanding of the key processes of the system. Figure 3.2 shows the conceptual model that underpins the Ruataniwha water storage GoldSim model. A numerical model is a set of equations, which, subject to certain assumptions, quantifies the physical processes active in the system(s) being modelled. While the model itself lacks the detailed reality of the environmental system, the behaviour of a valid model approximates that of the environmental system.

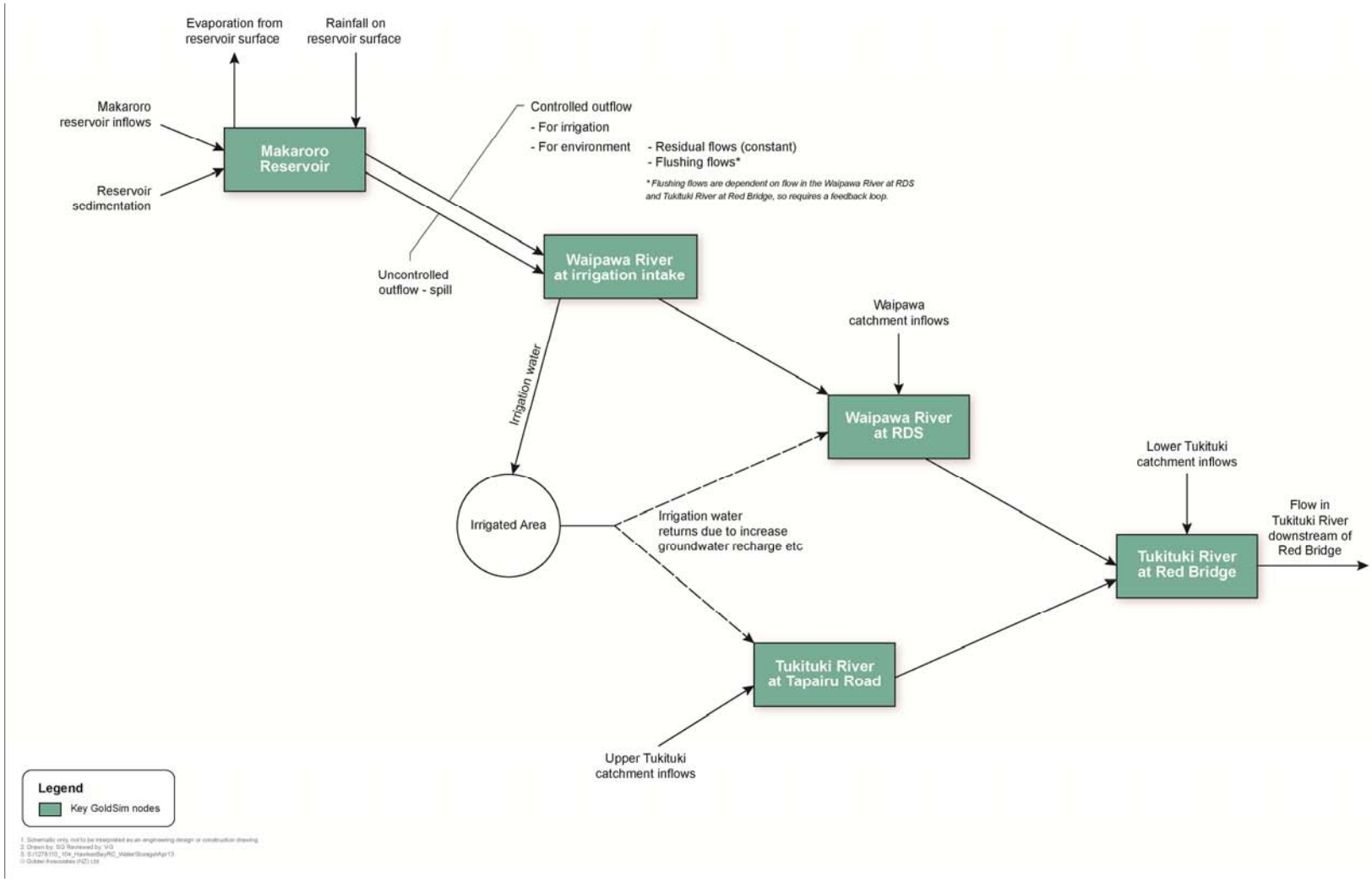


Figure 3.2: Conceptual model of the Ruataniwha Water Storage Scheme.

The main components of the model are highlighted in Figure 3.2 above. The model builds on earlier reservoir modelling work completed during the Feasibility Stage (T&T, 2012) and utilises the results of surface water and ground water modelling work completed by HBRC (2013c). The model, including model conceptualisation, the input data used, all model assumptions, the model build process and limitations are summarised in a separate report (Golder, May 2013) included as Appendix C of the environmental flow optimisation report (Aquanet, May 2013).

A number of "tranches" of water have been identified, which, in the model, are released from the dam within the limits of available water in the following order of priority:

- A primary residual flow at the toe of the dam equal to 90% of the 7-day mean annual low flow (7-day MALF). Refer to 3.2.2.3 for details
- A primary flushing flow allocation of up to 2.0 million m^3 per irrigation season (1 September to the following 30 April), corresponding with the release of up to two flushing flows of up to 30 m^3/s in size and just over 9 hours in duration. The size, timing and release triggers for the flushing flows are described in detail in Section 3.2.2.5 of this report
- A primary irrigation volume of up to 95.8 million m^3 per irrigation season, corresponding to a secure primary irrigation volume of 91 million m^3 delivered at the farm gate (allowing for a 5% conveyance loss from the dam to the farm gate) at full Scheme uptake
- A secondary flushing flow provision of up to 2.0 million m^3 per irrigation season, corresponding with the release of an additional up to two flushing flows of up to 30 m^3/s in size and just over 9 hours in duration (refer to Section 3.2.2.5) of this report
- A secondary irrigation volume of up to 28 million m^3 per irrigation season. Although mainly intended for irrigation purposes, a portion of this tranche of water may be utilised for purposes of river flow supplementation if that was necessary to mitigate any residual effect of the Scheme on low river flows, as described in the environmental flow optimisation report (Aquanet, May 2013).

3.2.2 Key inputs and assumptions

3.2.2.1 Input data

The model is based on a number of time series and data inputs which are based on both the results of various specific studies, and management decisions made by HBRIC Ltd regarding the scale and operation of the Scheme.

Reservoir inflows were estimated during hydrological investigation undertaken by T&T (August 2012b) during the Feasibility Stage. The inflow time series (1 January 1972 to 31 December 2010) is represented by the extended Makaroro at Burnt Bridge average daily flow record scaled down by a factor of 0.952 for the slightly smaller catchment at the dam site (111 km^2, versus 122 km^2 at the flow recording site).

Irrigation demand data was estimated during Feasibility Stage of the project (T&T, August 2012b) and is based on irrigation modelling of local climatic and soil conditions. The basis and derivation of the irrigation demand data is described in Section 3.1.5. Daily irrigation demand from 1 January 1972 to 31 December 2010 was input to the model.

River flow data for the Waipawa River at RDS and the Tukituki River at both Tapairu Road and Red Bridge were those produced by HBRC (2013c) as an output from their flow assessments. The daily time series (30 June 1969 to 30 June 2008) of naturalised flow and water takes (surface water, shallow stream depleting groundwater and deep groundwater)

were included in the model to allow prediction of actual flow in the Waipawa River at RDS and in the Tukituki River at Red Bridge both pre and post Scheme development.

Key dam characteristics and proposed operating rules were initially developed during the Feasibility Stage (T&T, 2012). The proposed operating rules were refined and optimised during the modelling process and model flexibility has been provided to enable the investigation of other combinations of rules if required.

3.2.2.2 Key assumptions

The model is based on hydrological years (1 July to following 30 June) and was set up for the full period of available overlapping time series data i.e. 36 complete hydrological years from 1 July 1972 to 30 June 2008. A key assumption is that future climatic and hydrological conditions are likely to be similar to historic records upon which the input time series have been based. Because of climatic fluctuations the above assumption is only valid if long climatic records are used.

A "live" reservoir volume of 85 million m^3 is assumed, to account for reservoir evaporation and losses of 1 million m^3 per year (refer to Section 3.1.3), and a sedimentation allowance of 4 million m^3.

A daily time step is utilised throughout the model. One of the main consequences is that the peak height of natural flushing flows may be underestimated by the model. Consequently, the model may be releasing more flushing flows than required, which is considered environmentally conservative. The decision to adopt a daily time step for the model was based on the available time series input data and particularly that in the earlier hydrological studies (HBRC 2013c) the process to naturalise the measured flow data was completed on a daily basis.

Full uptake of the primary irrigation tranche is assumed throughout the modelling period. This is considered a conservative assumption, as it is likely that not all of the primary irrigation allocation will be used in most years, particularly in the early years of Scheme operation, with the result that water will remain in the reservoir and be available to meet lower priority uses.

The modelling presented in this report assumes that no flow supplementation water is released from the dam in addition to the primary residual flow and the flushing flows described in Sections 3.2.2.3 and 3.2.2.5 respectively. Additional flow supplementation water may be required to ensure that the Scheme does not result in an increase in the number of days on which river flow is below the proposed regulatory minimum flow in the Waipawa River at RDS or in the Tukituki River at Red Bridge, primarily depending on the level of uptake of Scheme water by existing irrigators. These aspects are discussed in the environmental flow optimisation report (Aquanet, May 2013).

3.2.2.3 Primary residual flow

A permanent minimum residual flow of 1.23 m^3/s is proposed immediately downstream of the dam. This corresponds to 90% of the 7-day MALF (1.36 m^3/s) at the dam site. The 7-day MALF for the dam site was determined by scaling back the 7-day MALF for the Makaroro River at the Burnt Bridge site (estimated to be 1.43 m^3/s) based on catchment area and runoff considerations (T&T, August 2012b).

Figure 3.3 shows a low flow frequency distribution fitted to the 7-day annual (July to June year) minimum flows in the synthetically extended Makaroro at Burnt Bridge record for the period May 1968 to July 2011. Note that the minima for 1978 (large gap), 1979 (anomalous data) and 1985 (anomalous data) have been excluded.

The residual flow requirement is given the highest priority within the model and the proposed operating regime has been developed to ensure that the residual flow is provided 100% of the time during the 36 hydrological years that are modelled. During extreme years when inflows are low and demand high the live storage in the reservoir will be completely emptied. During such times the ability to fully supply the residual flow requirement has the potential to be compromised. To overcome this, the model has been set up to cut back irrigation supplies when the live volume in the reservoir falls below 2.5 million m^3 (i.e. less than approximately 3% of the maximum live storage) thereby ensuring that the residual flow is provided 100% of the time.

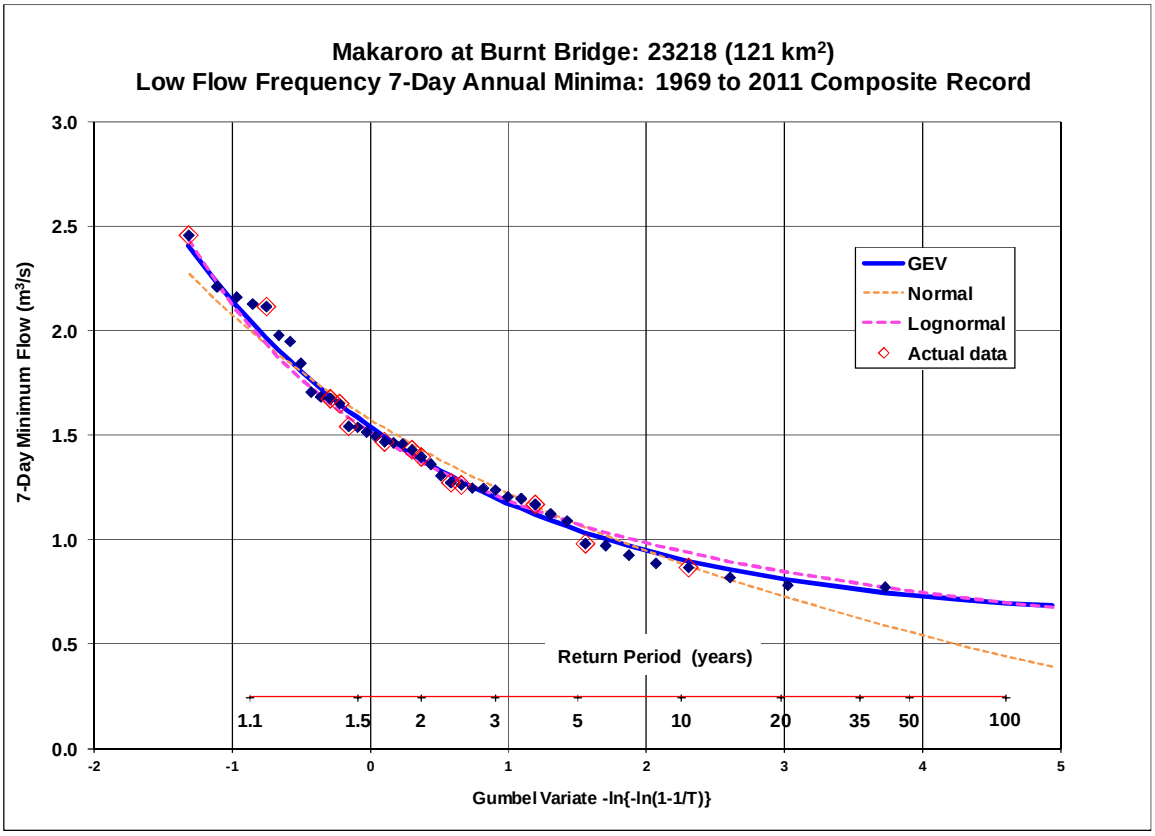


Figure 3.3 *Makaroro at Burnt Bridge low flow frequency analysis – based on the synthetically extended record from 1969 to 2011. Actual data from the recording site indicated by red diamonds.*

3.2.2.4 Primary irrigation water

A primary irrigation volume of up to 95.8 million m^3, corresponding to a primary irrigation volume of 91 million m^3 delivered at the farm gate and an allowance for conveyance/operational losses at full Scheme uptake has been defined.

As outlined in Section 3.1.5 irrigation demand was estimated during the Feasibility Stage of the project (T&T, August 2012b) and is based on irrigation modelling of local climatic and soil conditions. Daily irrigation demand from 1 January 1972 to 31 December 2010 is used in the model.

3.2.2.5 Flushing flows

Two tranches of water have been defined for flushing flows:

- A "primary" flushing flow allocation of up to 2.0 million m^3 per irrigation season
- A "secondary" flushing flow allocation, also of up to 2.0 million m^3 per irrigation season.

The primary flushing flows are given a higher level of priority than the primary irrigation water, and the secondary flushing flows are given a lower level of priority than the primary irrigation water, but a higher level of priority than the secondary irrigation water.

Each flushing flow will have a duration of 9.25 hours, and a maximum volume of 1 million m^3. The total number of flushing flows released under both the primary and secondary allocation per irrigation season is capped at four, with a maximum combined volume of 4 million m^3.

Rules and triggers have been defined in the model to control the release of flushing flows. Flushing flows are released:

- between 15 December and the end of the irrigation season on 30 April
- when the accrual period - between natural flow events that exceed 50 m^3/s in the Tukituki River at Red Bridge - approaches or reaches 30 days
- where possible, the flushing flows from the dam are released to coincide with natural minor freshes in order to maximise environmental benefit. In the model this is achieved by "piggy backing" onto natural flows which are greater than 15 m^3/s but less than 50 m^3/s. When the accrual period mentioned above exceeds 20 days the model looks for potential piggy back flows. If no piggy back flows occur before the accrual reaches 30 days then a flushing flow is automatically released on the 30th day of accrual. The rate of flow released during a "piggy back" event is the lesser of $30m^3/s$ and the difference between 50 m^3/s and the flow recorded in the river (e.g. if a 25 m^3/s natural flow is recorded, only 25 m^3/s is released for 9.25 hours from the reservoir).

3.2.2.6 Secondary irrigation water

A tranche of potentially available secondary irrigation water has been identified, which essentially corresponds to inflows into the reservoir once the other tranches have been provided for. One of the key objectives of the model was to assess the volumes of water that may be available within this secondary irrigation tranche, and how often and when in the season any secondary irrigation water becomes available.

The model achieves the above by determining when in the irrigation season the combined storage remaining in the reservoir and the volume already supplied to the primary irrigators surpasses a certain critical "ring-fenced" volume. Any subsequent inflows into the reservoir, which are not required to meet residual and flushing flow releases requirements, are then potentially available for secondary irrigation. To ensure the supply reliability of the primary irrigators is not compromised by the secondary irrigators a critical ring-fenced volume of at least the maximum allocation assigned to the primary irrigation tranche (namely 95.8 million m^3) is required. To cater for times when reservoir inflows are insufficient to meet the residual and flushing flow releases requirements there is need to further increase the critical ring-fenced volume. The modelling process found that a critical ring-fenced volume of 105% of the maximum allocation assigned to the primary irrigation tranche (namely 1.05 x 95.8 million m^3) was sufficient to ensure the residual flow and

flushing flows requirements were met and the supply reliability of the primary irrigators was not unduly compromised by the supply of secondary irrigation water.

Including the secondary irrigation allocation increases the overall volume of water that is harvested from the river system and as a result leads to lower reservoir levels, which extends the period during which the reservoir is refilling, in turn resulting in lower downstream flows during the re-filling period. To prevent over-harvesting of water a seasonal cap of 28 million m^3 was placed on the secondary irrigation allocation.

3.2.3 Modelling outputs

3.2.3.1 Primary residual flow

Under the modelled scenario, the primary residual flow was continuously supplied throughout the 36 hydrological year modelling period. As outlined above, to ensure that the primary residual flow is provided 100% of the time, irrigation supplies need to be cut back when the live volume in the reservoir falls below 2.5 million m^3.

3.2.3.2 Primary irrigation water

Under the modelled scenario, the predicted cumulative volume of primary irrigation water that is supplied each irrigation season is shown in Figure 3.4. The model predicts the irrigation demand is fully supplied up to the maximum primary irrigation volume of 95.8 million m^3 every irrigation season over the 36 seasons modelled except three seasons (1972-73, 1982-83 and 1997-98). The predicted cumulative volume of the shortfall in the supply of primary irrigation water each irrigation season is also shown in Figure 3.5. The three shortfall seasons are characterised as follows:

- In the 1972-1973 irrigation season, a shortfall of approximately 3.9 million m^3 is predicted. The shortfall occurs over two periods. Approximately half of the shortfall occurs from 6-13 March 1973 with the remainder occurring from 24 to 29 March 1973.
- In the 1982-1983 irrigation season, a shortfall of approximately 10.9 million m^3 is predicted. The shortfall occurs over an approximately two week period from 2 to17 March 1983.
- In the 1997-1998 irrigation season, a shortfall of approximately 6.3 million mm^3 is predicted. The shortfall occurs over a two week period from 23 February to 2 March 1998.

In all three cases the shortfall occurs when the live volume in the reservoir has been drawn down to 2.5 million m^3 at which time irrigation is curtailed to ensure sufficient volume remains to fully supply the residual flow requirements. Low reservoir volumes are predominantly due to drawdown to meet irrigation demand that season coupled with low inflows although the low reservoir volume during the 1982-83 irrigation season was in part due to the reservoir not fully replenishing over the previous winter.

All these shortfalls occur relatively late in the season and, depending on the irrigated crop, irrigators may be able to manage around them without incurring major production losses.

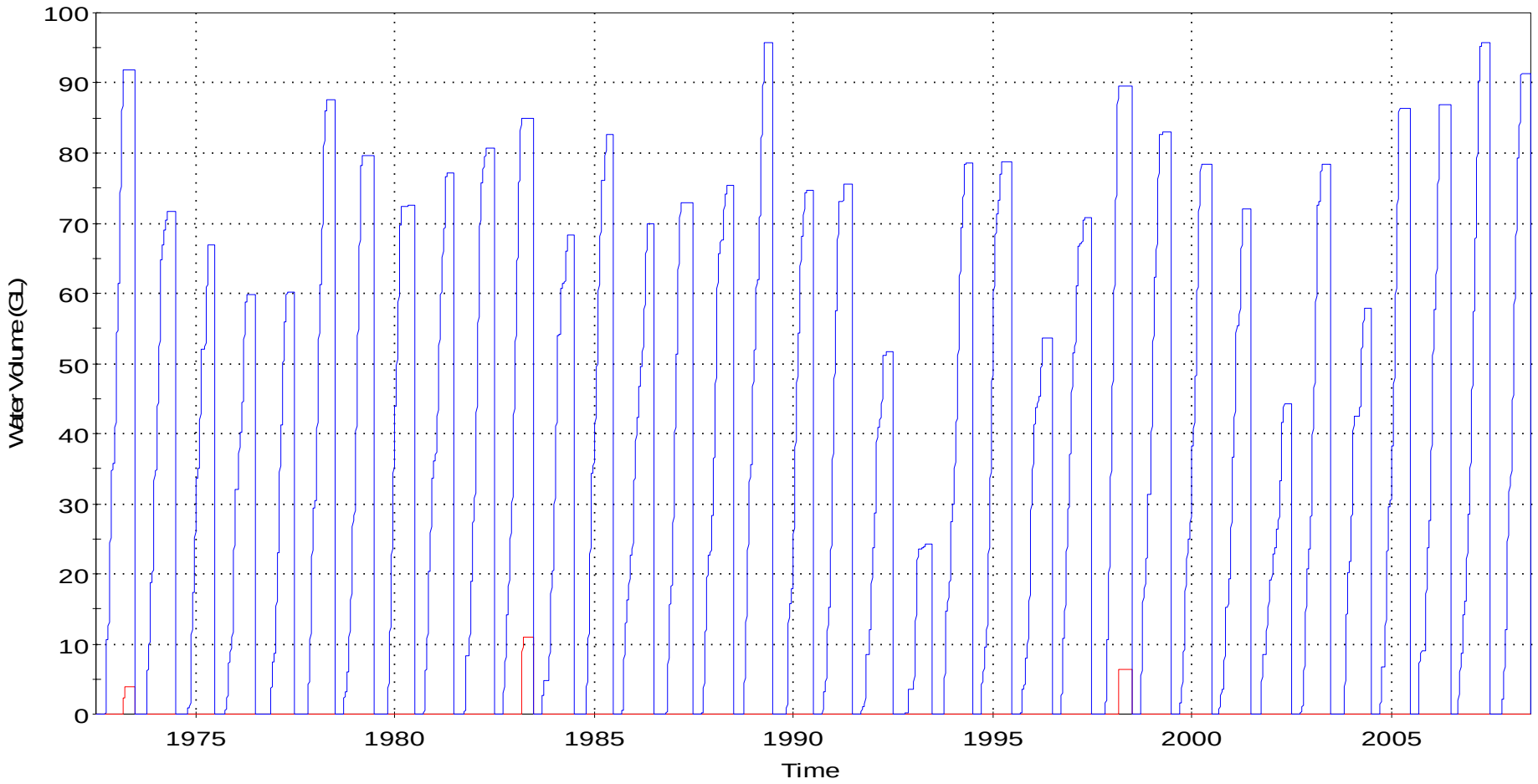


Figure 3.4 Predicted cumulative primary irrigation volume supplied (blue) and shortfall (red) per irrigation season. (Note: 1 GL = 1 million m³)

3.2.3.3 Flushing flows

The model predicts a strong demand for flushing flows, and in all of the 36 irrigation seasons modelled, the primary flushing flow tranche of 2.0 million m^3 is released. The maximum number of four flushing flows is released in all years other than the 1979-1980, 1995-1996 and 1998-1999 irrigation seasons, when only three flushing flows are released. The model does not predict a shortfall of water (i.e. dam does not empty out) in any of the three years 1979-1980, 1995-1996 and 1998-1999 and the need to flush only three times is due to a fourth not being required.

a There were natural freshes in late December 1979, mid February and early March 1980

b There were natural freshes in late January, mid and late February and late March 1996

c There were natural freshes in early December 1998, mid and late January, mid March and mid April 1999

The predicted cumulative volume of flushing flow water released from the reservoir each irrigation season is shown in Figure 3.5 and summary statistics are provided in Table 3.1.

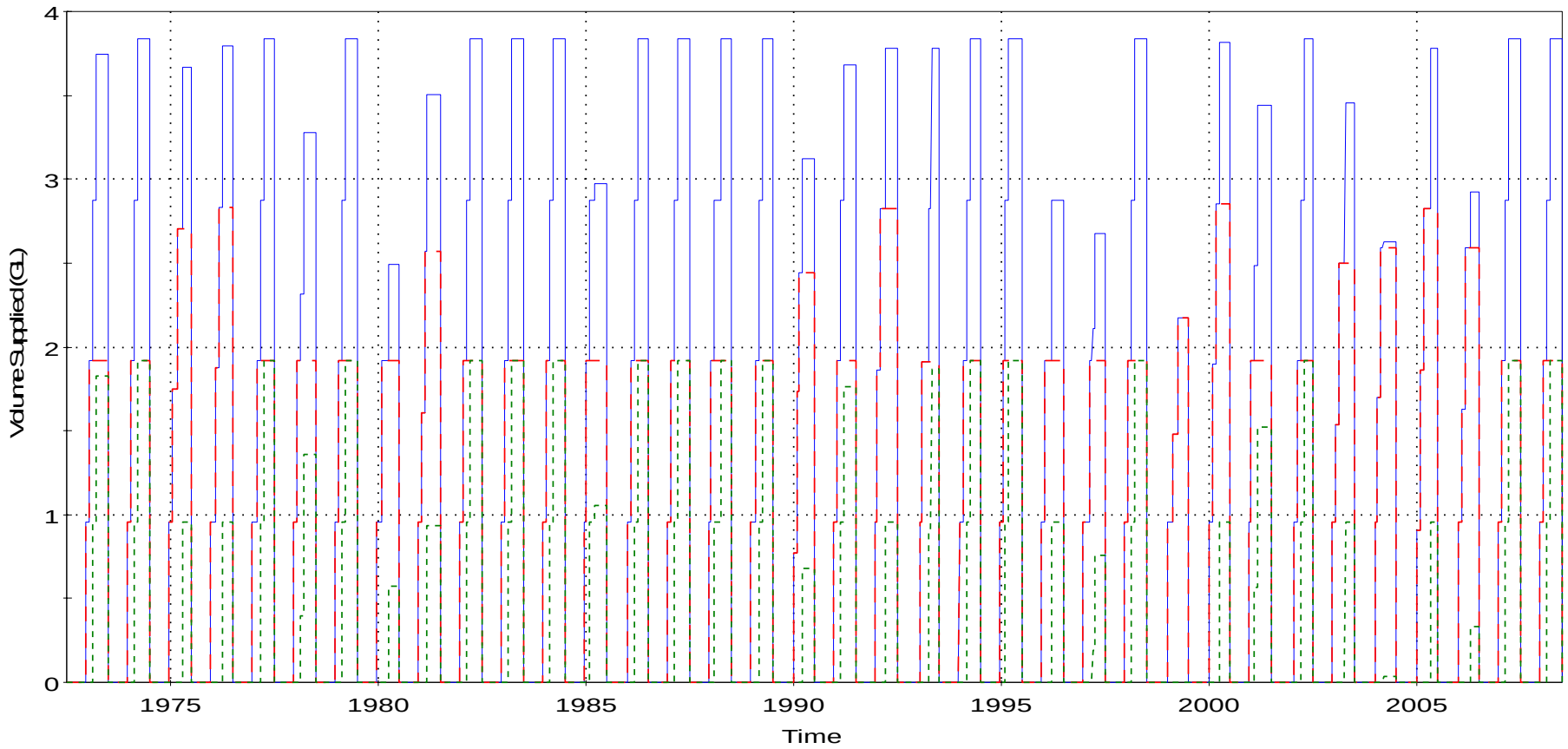
Table 3.1 Annual statistics for predicted flushing flow releases from the Makaroro Reservoir per hydrological year

Hydrological year	Volume of flush flows released under the primary allocation (million m3)	Volume of flush flows released under the secondary allocation (million m3)	Total volume of flush flows released (million m3)
1972 - 1973	1.9	1.8	3.7
1973 - 1974	1.9	1.9	3.8
1974 - 1975	2.7	1.0	3.7
1975 - 1976	2.8	1.0	3.8
1976 - 1977	1.9	1.9	3.8
1977 - 1978	1.9	1.4	3.3
1978 - 1979	1.9	1.9	3.8
1979 - 1980	1.9	0.6	2.5
1980 - 1981	2.6	0.9	3.5
1981 - 1982	1.9	1.9	3.8
1982 - 1983	1.9	1.9	3.8
1983 - 1984	1.9	1.9	3.8
1984 - 1985	1.9	1.1	3.0
1985 - 1986	1.9	1.9	3.8
1986 - 1987	1.9	1.9	3.8
1987 - 1988	1.9	1.9	3.8
1988 - 1989	1.9	1.9	3.8
1989 - 1990	2.4	0.7	3.1

1990 - 1991	1.9	1.8	3.7
1991 - 1992	2.8	1.0	3.8
1992 - 1993	1.9	1.9	3.8
1993 - 1994	1.9	1.9	3.8
1994 - 1995	1.9	1.9	3.8
1995 - 1996	1.9	1.0	2.9
1996 - 1997	1.9	0.8	2.7
1997 - 1998	1.9	1.9	3.8
1998 - 1999	2.2	0.0	2.2
1999 - 2000	2.9	1.0	3.8
2000 - 2001	1.9	1.5	3.4
2001 - 2002	1.9	1.9	3.8
2002 - 2003	2.5	1.0	3.5
2003 - 2004	2.6	0.0	2.6
2004 - 2005	2.8	1.0	3.8
2005 - 2006	2.6	0.3	2.9
2006 - 2007	1.9	1.9	3.8
2007 - 2008	1.9	1.9	3.8
Minimum	1.9	0.0	2.2
Median	1.9	1.8	3.8
Mean	2.1	1.4	3.5
Maximum	2.9	1.9	3.8

Note: The model sometimes predicts releases slightly more than the 2.0 million m^3 caps due to the way it assesses the flushing flow allocation caps. When below the allocation cap the model will release a flushing flow even if releasing will exceed the cap.

Figure 3.5 *Predicted flushing flow volumes released per irrigation season. The red dashed line represents flushing flows released from the dam under the primary allocation. The green dotted line represents flushing flows released under the secondary allocation. The solid blue line represents the sum of the primary and secondary flushing flows releases. (Note: 1 GL = 1 million m³)*



3.2.3.4 Secondary irrigation

The model predicts a significant volume of secondary irrigation water is potentially available, with the full 28 million m^3 allocation predicted to be available during 16 of the 36 seasons modelled. There are three years where no secondary irrigation water is available (1972-73, 1982-83 and 1997-98) and a further four years (1983-84, 1990-91, 1993-94 and 2007-2008) when the predicted volume available is less than 5 million m^3.

3.2.3.5 Reservoir operating volumes and levels

Storage volumes within the reservoir are predicted to fluctuate considerably, although generally storage volumes tend to increase over the winter months as flows are harvested and the reservoir refills, with volumes tending to decrease over the summer irrigation season as the reservoir is drawn down to supply irrigation demand. Predicted reservoir volume fluctuations over the 36 hydrological years that were modelled are shown in Figure 3.6 with summary statistics provided in Tables 3.2 and 3.3. Figure 3.7 shows the corresponding predicted water level fluctuations in the reservoir.

The corresponding reservoir water level duration curves (i.e. cumulative probability curves of the reservoir levels) are shown in Figure 3.8, both for the full year and for each of 4 seasons.

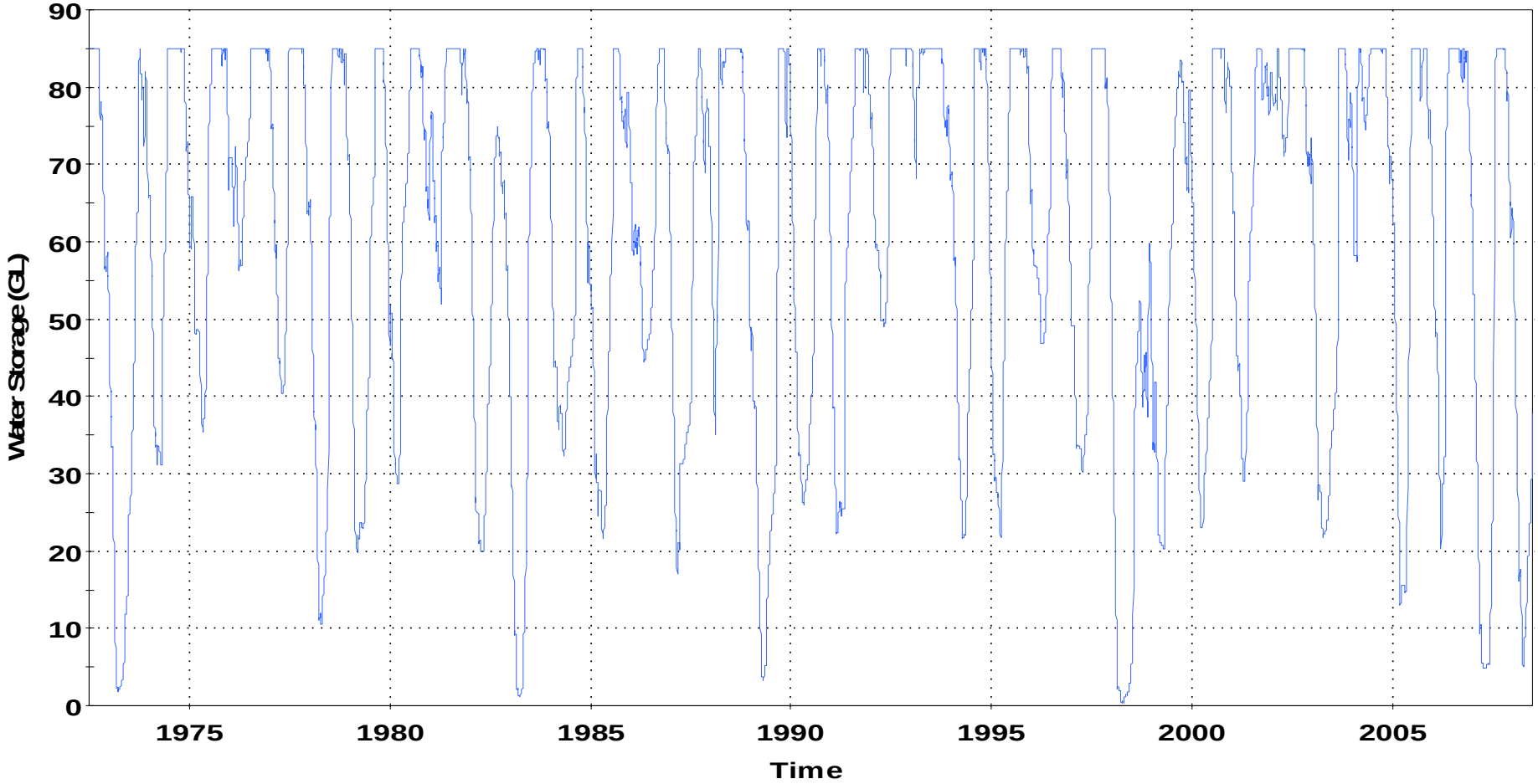


Figure 3.6 Predicted behaviour of live storage volume with time in the Makaroro Reservoir (note: on the vertical axis 1 GL = 1 million m³).

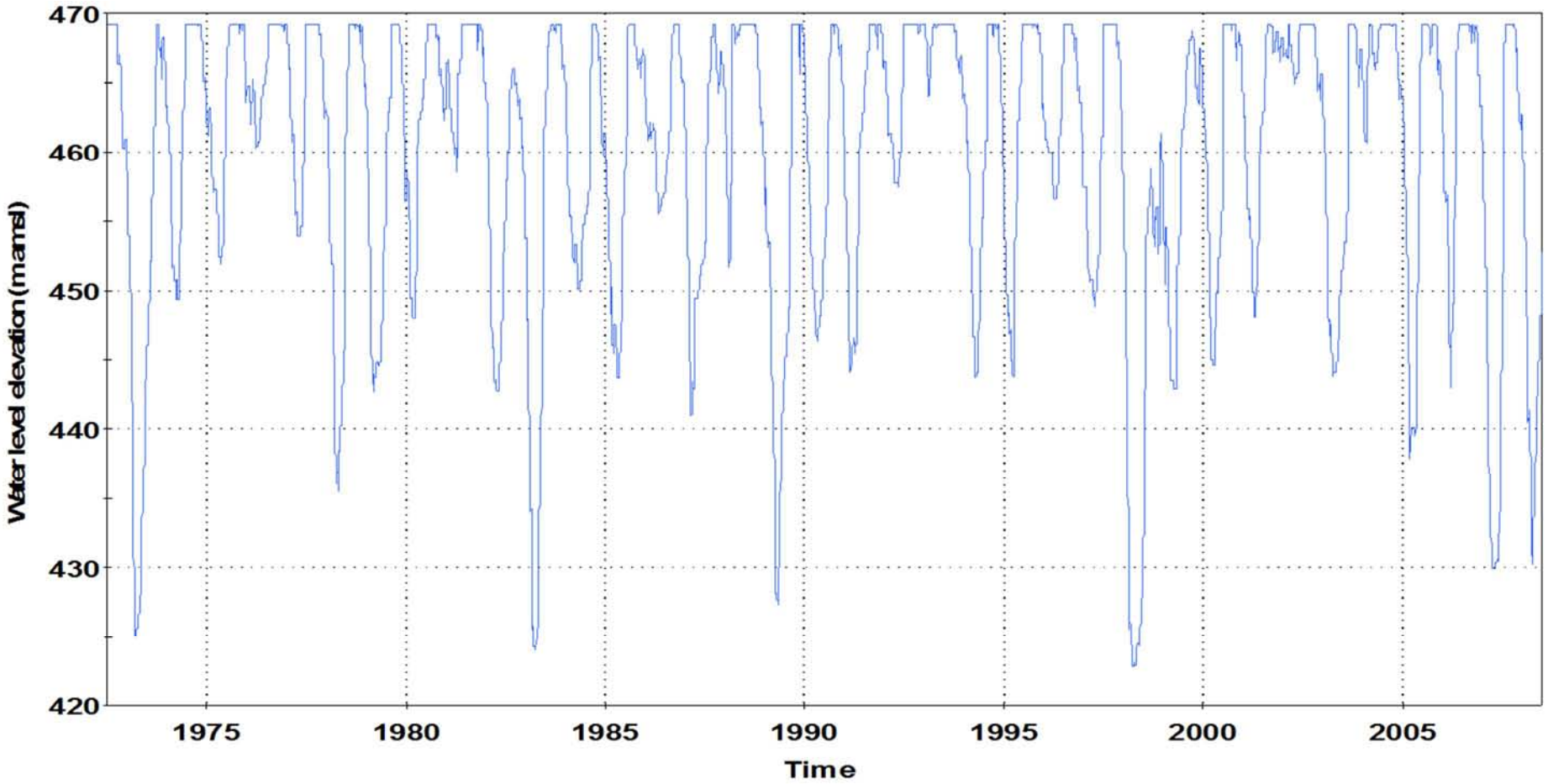


Figure 3.7 Predicted behaviour of water levels with time in the Makaroro Reservoir. (Note: water levels are to HBRC datum, mean sea level = 10 mRL)

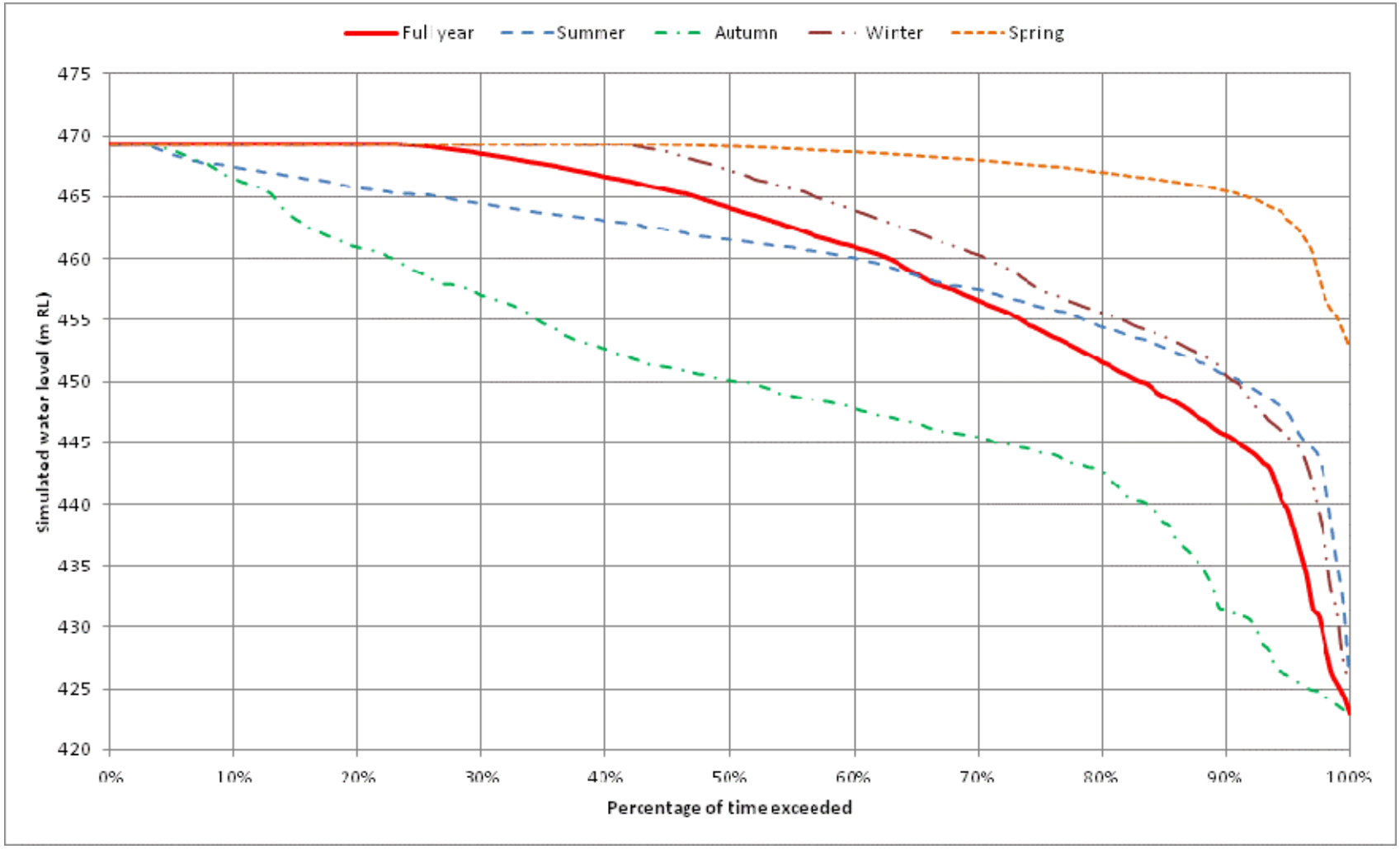


Figure 3.8 Modelled water level duration curves for Makaroro Reservoir for full year and by season. (Water levels are to HBRC datum, mean sea level = 10 mRL)

Table 3.2 Annual statistics for predicted live storage volume in the Makaroro Reservoir per hydrological year

Hydrological year	Minimum Live Storage (million m^3)	Average Live Storage (million m^3)	Maximum Live Storage (million m^3)
1972 - 1973	1.7	45.1	85.0
1973 - 1974	24.9	56.1	85.0
1974 - 1975	35.4	66.6	85.0
1975 - 1976	56.3	74.6	85.0
1976 - 1977	40.4	71.1	85.0
1977 - 1978	10.5	55.5	85.0
1978 - 1979	19.8	56.7	85.0
1979 - 1980	28.7	61.6	85.0
1980 - 1981	51.9	74.7	85.0
1981 - 1982	20.0	58.3	85.0
1982 - 1983	1.2	40.7	75.0
1983 - 1984	32.3	61.6	85.0
1984 - 1985	21.5	51.2	85.0
1985 - 1986	44.4	65.8	85.0
1986 - 1987	17.1	52.8	85.0
1987 - 1988	35.0	71.4	85.0
1988 - 1989	3.3	49.5	85.0
1989 - 1990	22.3	51.6	85.0
1990 - 1991	22.2	55.8	85.0
1991 - 1992	49.0	71.0	85.0
1992 - 1993	68.1	83.5	85.0
1993 - 1994	21.5	60.7	85.0
1994 - 1995	21.8	60.4	85.0
1995 - 1996	46.9	69.2	85.0
1996 - 1997	30.2	59.1	85.0
1997 - 1998	0.3	44.8	85.0
1998 - 1999	5.2	37.6	60.5
1999 - 2000	23.0	57.4	83.5
2000 - 2001	29.0	64.1	85.0
2001 - 2002	63.5	79.4	85.0

2002 - 2003	21.8	57.4	85.0
2003 - 2004	34.7	72.2	85.0
2004 - 2005	13.0	59.3	85.0
2005 - 2006	20.3	67.6	85.0
2006 - 2007	4.9	50.5	85.0
2007 - 2008	5.1	49.8	85.0
Minimum	0.3	37.6	60.5
Mean	26.3	60.1	84.0
Maximum	68.1	83.5	85.0
Volume exceeded 95% of the time	1.6	43.8	81.3
Volume exceeded 90% of the time	4.1	47.3	85.0
Volume exceeded 75% of the time	16.1	52.5	85.0
Median	22.3	59.2	85.0

Table 3.3 Average monthly predicted live storage volume in the Makaroro Reservoir per hydrological year (million m^3)

Hydrological Year	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Mean
1972 - 1973	85.0	85.0	84.9	77.2	62.5	56.0	39.2	17.8	2.4	2.6	7.3	17.7	45.1
1973 - 1974	28.9	45.8	68.8	81.9	78.4	71.5	59.0	40.0	33.3	32.9	55.3	76.9	56.1
1974 - 1975	85.0	85.0	85.0	85.0	81.6	70.0	62.8	55.9	48.6	41.1	37.6	60.2	66.6
1975 - 1976	81.1	85.0	85.0	84.6	84.8	76.2	70.6	67.0	63.3	58.4	66.7	72.4	74.6
1976 - 1977	81.8	85.0	85.0	85.0	85.0	84.7	78.4	63.5	50.3	41.7	44.1	67.4	71.1
1977 - 1978	85.0	85.0	85.0	84.9	79.3	65.5	57.1	37.6	20.7	12.1	16.8	34.4	55.5
1978 - 1979	75.3	85.0	84.8	84.7	82.5	72.1	54.8	29.3	21.1	23.1	25.8	39.3	56.7
1979 - 1980	60.8	74.2	85.0	84.9	78.1	57.5	48.1	34.1	29.2	50.3	64.7	72.2	61.6
1980 - 1981	83.6	85.0	84.9	81.6	72.6	65.3	74.5	64.1	57.3	63.0	78.5	85.0	74.7
1981 - 1982	85.0	85.0	85.0	82.9	83.4	74.8	59.1	34.3	23.6	20.3	24.9	38.3	58.3
1982 - 1983	51.5	69.9	73.5	68.8	63.4	50.7	29.7	10.0	1.8	2.9	15.1	48.4	40.7
1983 - 1984	74.5	83.9	84.3	84.7	83.2	74.9	58.1	44.0	37.4	36.3	34.2	42.0	61.6
1984 - 1985	45.6	63.6	84.9	83.7	71.9	58.2	50.3	34.5	27.4	24.4	25.2	43.8	51.2
1985 - 1986	62.6	85.0	84.2	77.1	75.2	76.5	63.7	60.0	60.2	51.7	45.2	47.5	65.8
1986 - 1987	52.2	65.9	83.2	85.0	77.8	68.8	47.4	26.2	23.4	31.3	33.5	36.1	52.8
1987 - 1988	49.3	73.1	83.5	78.2	74.5	73.8	55.5	41.7	75.0	82.4	83.9	85.0	71.4
1988 - 1989	85.0	85.0	85.0	82.8	69.0	57.5	44.4	36.1	18.2	4.3	6.2	18.0	49.5
1989 - 1990	24.6	34.4	76.3	85.0	80.1	82.7	64.9	47.4	34.0	27.9	28.4	33.7	51.6
1990 - 1991	42.7	68.4	84.8	84.7	80.3	69.1	45.4	27.0	25.1	25.4	51.7	63.6	55.8
1991 - 1992	73.4	82.8	85.0	83.7	84.3	79.6	72.9	60.2	56.7	49.8	51.5	72.0	71.0
1992 - 1993	85.0	85.0	85.0	85.0	84.6	85.0	81.6	73.0	82.0	84.9	85.0	85.0	83.5
1993 - 1994	85.0	85.0	85.0	81.9	75.5	72.9	64.5	52.0	33.6	23.5	26.5	41.7	60.7
1994 - 1995	58.3	85.0	84.3	84.9	82.3	62.2	37.8	29.0	24.4	35.1	60.6	79.3	60.4
1995 - 1996	85.0	85.0	84.9	84.8	83.9	72.3	60.3	56.3	52.7	47.0	52.5	65.4	69.2

1996 - 1997	84.1	85.0	84.7	79.8	71.0	60.1	48.6	37.4	33.5	31.7	36.1	54.8	59.1
1997 - 1998	82.5	85.0	85.0	85.0	80.6	62.9	36.1	9.9	1.5	0.6	1.4	3.2	44.8
1998 - 1999	14.9	43.6	49.4	41.9	42.1	55.0	39.4	35.4	21.7	20.4	34.3	53.8	37.6
1999 - 2000	66.2	77.6	82.1	78.5	69.3	76.7	62.2	46.4	27.1	25.4	33.8	42.0	57.4
2000 - 2001	77.1	85.0	84.8	84.0	80.4	78.6	62.9	47.2	40.8	29.9	38.1	58.5	64.1
2001 - 2002	68.9	83.8	84.8	80.1	81.3	80.3	78.7	81.8	81.2	73.1	74.2	84.8	79.4
2002 - 2003	85.0	85.0	85.0	83.9	73.3	71.3	60.2	38.0	27.2	22.4	23.9	31.0	57.4
2003 - 2004	37.2	52.1	85.0	84.9	77.5	76.8	64.0	67.2	84.2	77.2	77.4	84.2	72.2
2004 - 2005	85.0	85.0	84.9	85.0	78.2	70.1	60.0	37.6	16.7	15.5	25.9	66.0	59.3
2005 - 2006	85.0	85.0	81.6	84.6	80.1	73.8	55.9	45.2	26.5	30.8	75.9	84.8	67.6
2006 - 2007	85.0	85.0	83.6	83.5	82.5	69.1	51.1	29.6	11.1	4.9	5.3	11.9	50.5
2007 - 2008	48.8	84.8	85.0	84.9	78.4	64.6	54.6	30.8	14.4	7.1	16.6	26.8	49.8
Minimum	14.9	34.4	49.4	41.9	42.1	50.7	29.7	9.9	1.5	0.6	1.4	3.2	
Mean	67.9	77.5	82.6	81.6	76.9	69.9	57.1	43.0	35.8	33.6	40.7	53.5	
Maximum	85.0	85.0	85.0	85.0	85.0	85.0	81.6	81.8	84.2	84.9	85.0	85.0	
Exceeded 95%	27.9	45.2	72.3	75.0	63.2	55.7	37.4	15.9	2.2	2.8	6.0	16.3	
Exceeded 90%	40.0	57.8	79.0	77.7	69.2	57.5	39.3	26.6	12.8	4.6	11.2	22.4	
Exceeded 75%	52.0	74.0	84.1	81.8	74.2	64.2	48.5	33.2	21.5	20.4	25.1	37.8	
Median	76.2	85.0	84.9	84.3	78.9	71.4	58.5	39.0	28.3	30.4	35.2	54.3	

3.2.3.6 Dam outflow regime

In this section, the modelled dam outflow regime is presented and compared against the pre-existing flow regime at the dam site. Table 3.4 provides a tabulation of the simulated mean monthly outflows from the dam, which may be compared against the corresponding tabulation of reservoir inflows in Table 3.5. Table 3.6 summarises the predicted changes in the monthly mean flows immediately below the dam from operation of the scheme assuming full uptake. This comparison shows that with the scheme in full operation:

- Monthly mean flows would be broadly more uniform throughout the year, with a lesser bias towards winter than under natural flow conditions
- As expected, average flows would be substantially higher during the main irrigation season (October to April), particularly over January and February (more than 100% higher)

- In contrast, average flows would be significantly lower in April, May and June as the reservoir retains inflow to refill.

Table 3.4 Modelled mean monthly dam outflows – comprising irrigation supply, environmental releases and spill flow

Hydrological Year	Jul (m^3/s)	Aug (m^3/s)	Sep (m^3/s)	Oct (m^3/s)	Nov (m^3/s)	Dec (m^3/s)	Jan (m^3/s)	Feb (m^3/s)	Mar (m^3/s)	Apr (m^3/s)	May (m^3/s)	Jun (m^3/s)	Mean (m^3/s)
1972- 1973	6.96	3.76	5.41	6.42	8.15	5.51	9.01	11.04	2.67	1.23	1.23	1.23	5.18
1973- 1974	1.23	1.23	1.24	4.90	5.67	8.61	8.36	8.07	5.38	7.30	1.23	10.52	5.28
1974- 1975	16.54	10.53	15.03	8.05	7.89	6.55	5.48	9.07	10.27	6.61	1.23	1.23	8.20
1975- 1976	1.89	4.93	7.40	7.06	5.84	10.71	9.81	7.08	8.27	1.65	1.23	1.23	5.60
1976- 1977	3.74	15.13	22.84	11.91	10.16	5.97	7.94	8.30	9.12	7.11	1.23	4.10	8.94
1977- 1978	12.26	17.42	16.93	8.07	7.82	5.04	9.74	7.16	8.98	4.01	1.23	1.23	8.34
1978- 1979	9.23	7.05	3.90	3.88	7.05	5.72	11.49	9.69	12.01	1.84	1.23	1.23	6.19
1979- 1980	1.23	2.79	15.37	9.31	7.75	10.28	6.37	9.16	10.22	1.49	1.23	1.23	6.35
1980- 1981	5.21	4.89	5.21	6.47	6.22	14.89	5.94	7.22	4.98	3.75	2.06	12.52	6.61
1981- 1982	7.37	10.17	8.61	4.76	5.83	5.93	10.53	11.32	3.20	5.23	1.23	1.23	6.25
1982- 1983	1.23	1.23	3.43	4.38	6.77	8.14	10.13	8.23	1.59	1.23	1.23	1.23	4.04
1983- 1984	1.23	1.35	3.41	6.35	6.12	6.30	10.59	3.99	3.07	4.30	1.23	1.23	4.10
1984- 1985	1.23	1.23	12.21	6.35	8.02	5.93	6.52	9.36	10.88	5.92	1.23	1.23	5.80
1985- 1986	2.25	8.88	5.70	4.47	3.68	10.79	7.63	7.40	4.63	6.64	1.23	1.23	5.37
1986- 1987	1.23	1.23	13.97	6.34	7.63	6.77	10.24	8.62	5.11	6.05	1.23	1.23	5.76
1987- 1988	1.23	1.23	3.23	6.75	7.47	8.81	11.06	7.63	8.44	3.73	1.71	3.80	5.42
1988- 1989	17.82	13.42	13.72	7.72	7.83	7.60	6.30	6.81	9.32	3.23	1.23	1.23	8.04
1989- 1990	1.23	1.23	18.11	10.39	8.07	9.39	10.85	7.84	6.19	4.10	1.23	1.23	6.63
1990- 1991	1.23	1.30	4.44	5.19	4.49	10.95	8.94	6.79	1.65	4.47	1.23	1.23	4.31
1991- 1992	1.23	5.75	3.19	5.73	10.44	6.60	6.55	5.76	6.38	3.28	1.23	1.52	4.80
1992- 1993	22.10	10.84	10.70	20.49	7.36	8.22	7.73	11.97	4.97	4.14	5.72	4.88	9.94
1993- 1994	2.88	4.40	5.79	6.90	2.51	5.99	5.89	9.58	6.29	4.04	1.23	1.23	4.70
1994- 1995	1.23	6.84	4.82	6.35	10.33	11.25	8.25	5.79	4.79	7.87	1.23	4.17	6.07
1995- 1996	15.17	7.98	5.31	7.30	6.05	8.86	7.89	4.59	6.23	5.25	1.23	1.23	6.46
1996- 1997	17.85	7.98	6.41	6.04	7.15	10.18	6.96	6.22	5.73	2.76	1.23	1.23	6.67
1997- 1998	7.18	7.20	9.49	12.43	8.03	11.49	11.49	8.88	1.59	1.23	1.23	1.23	6.79
1998- 1999	1.23	1.23	5.19	5.67	4.74	6.68	9.75	9.54	4.28	6.83	1.23	1.23	4.76
1999- 2000	1.23	1.23	2.35	6.19	5.04	8.77	6.84	9.77	6.24	6.72	1.23	1.23	4.72
2000- 2001	9.39	5.21	6.32	10.83	2.48	7.89	8.95	5.82	6.86	3.37	1.23	1.23	5.82
2001- 2002	1.23	6.70	5.17	2.86	3.65	8.77	7.79	5.77	6.30	3.67	1.23	8.63	5.14
2002- 2003	19.28	11.69	4.63	6.26	5.68	6.75	8.99	11.13	6.88	3.01	1.23	1.23	7.24
2003- 2004	1.23	2.35	24.29	8.12	7.84	5.90	11.14	10.82	7.45	4.15	1.23	7.36	7.60

2004- 2005	10.02	11.31	5.74	10.06	8.56	5.90	10.14	11.41	8.41	4.12	1.23	3.55	7.53
2005- 2006	17.44	4.65	4.75	14.76	7.38	16.35	9.82	7.45	6.87	1.36	1.23	10.29	8.57
2006- 2007	27.03	10.65	4.90	3.47	8.09	7.51	9.73	10.07	7.94	2.46	1.23	1.23	7.88
2007- 2008	1.23	7.88	7.18	8.13	8.05	5.37	10.41	9.99	6.24	2.03	1.23	1.23	5.74
Minimum	1.23	1.23	1.24	2.86	2.48	5.04	5.48	3.99	1.59	1.23	1.23	1.23	4.04
Mean	6.98	6.19	8.23	7.51	6.83	8.23	8.76	8.31	6.37	4.06	1.39	2.83	6.30
Maximum	27.03	17.42	24.29	20.49	10.44	16.35	11.49	11.97	12.01	7.87	5.72	12.52	9.94

Table 3.5 Mean monthly reservoir inflows, i.e. the pre-existing flow regime at the Makaroro Dam site

Hydrological Year	Jul (m^3/s)	Aug (m^3/s)	Sep (m^3/s)	Oct (m^3/s)	Nov (m^3/s)	Dec (m^3/s)	Jan (m^3/s)	Feb (m^3/s)	Mar (m^3/s)	Apr (m^3/s)	May (m^3/s)	Jun (m^3/s)	Mean (m^3/s)
1972- 1973	6.96	3.76	4.20	2.70	2.07	2.75	1.63	1.24	1.50	1.77	4.44	6.16	3.28
1973- 1974	5.26	10.28	10.34	3.56	6.29	2.85	2.17	1.70	5.22	12.06	8.98	18.14	7.23
1974- 1975	16.54	10.53	15.03	8.10	3.20	2.77	6.28	2.40	9.62	2.18	4.31	13.80	7.93
1975- 1976	5.14	4.93	7.40	7.15	6.06	5.45	8.40	8.44	2.81	3.83	4.39	3.04	5.58
1976- 1977	7.37	15.13	22.84	11.91	10.24	5.87	3.05	3.13	3.73	5.48	5.92	16.66	9.28
1977- 1978	12.26	17.42	16.93	7.42	3.11	2.69	2.00	2.08	1.35	4.96	4.37	16.61	7.62
1978- 1979	18.71	7.05	4.04	3.94	4.05	2.09	1.38	1.66	12.24	2.40	4.56	6.71	5.79
1979- 1980	8.57	10.38	15.37	8.60	2.32	2.79	3.76	3.96	11.87	12.74	3.09	5.88	7.44
1980- 1981	7.36	4.89	4.84	3.39	2.68	16.21	5.02	3.08	3.78	10.84	6.31	12.52	6.76
1981- 1982	7.37	10.17	8.61	4.93	3.34	2.57	1.50	2.95	1.81	4.77	5.24	6.35	4.98
1982- 1983	8.39	5.15	2.52	2.98	2.35	2.08	1.62	1.94	1.25	4.03	7.53	17.55	4.79
1983- 1984	6.63	2.44	2.74	7.22	3.88	2.57	1.91	1.81	2.60	1.73	3.90	2.97	3.38
1984- 1985	2.67	14.66	13.55	3.50	1.83	3.37	2.22	3.68	10.41	3.55	5.10	9.56	6.19
1985- 1986	14.24	8.88	3.75	2.82	3.56	8.50	3.65	6.85	4.10	1.80	1.96	2.08	5.19
1986- 1987	4.70	9.30	16.13	6.17	2.50	2.34	1.43	1.18	10.16	6.06	2.68	2.37	5.44
1987- 1988	12.69	5.35	4.91	2.69	9.63	3.91	2.33	10.80	21.26	3.27	2.65	3.80	6.94
1988- 1989	17.82	13.42	13.68	4.64	2.14	1.79	3.43	2.35	2.25	1.16	5.09	4.46	6.06
1989- 1990	3.08	11.76	31.60	10.41	8.31	6.05	2.58	2.31	2.86	1.94	2.95	4.24	7.34
1990- 1991	8.22	11.75	3.78	5.42	3.16	1.95	1.34	1.52	2.89	5.48	12.77	4.99	5.31
1991- 1992	4.41	8.44	3.26	3.90	14.86	3.14	2.25	3.96	3.02	2.69	4.56	11.99	5.52
1992- 1993	22.10	10.84	10.70	20.49	7.94	8.22	3.27	14.84	6.93	4.15	5.78	4.88	10.01
1993- 1994	2.88	4.40	5.79	3.10	3.54	2.73	1.86	2.34	1.63	1.75	6.83	6.01	3.57
1994- 1995	14.28	7.15	4.97	6.61	6.31	1.89	1.80	4.17	2.71	16.18	9.94	10.18	7.18
1995- 1996	15.17	7.98	5.43	7.17	5.75	3.01	4.23	3.95	3.08	5.61	5.95	8.17	6.31
1996- 1997	20.38	7.98	5.45	3.51	3.92	3.51	4.65	2.31	5.26	3.27	3.46	9.79	6.16
1997- 1998	16.22	7.20	9.49	12.21	4.25	1.98	1.08	1.08	0.84	1.54	1.46	2.56	5.03
1998- 1999	13.33	5.15	3.52	5.47	6.23	6.60	7.86	2.73	2.67	6.66	8.67	9.00	6.52
1999- 2000	6.26	3.85	3.04	2.22	7.83	3.87	2.52	1.53	2.05	9.54	3.91	5.07	4.30
2000- 2001	23.52	5.21	7.03	8.23	4.24	2.88	3.07	1.77	2.70	2.01	10.33	5.20	6.41

2001- 2002	7.52	9.28	4.07	2.76	2.96	9.42	6.74	8.49	2.78	2.45	5.09	9.50	5.91
2002- 2003	19.28	11.69	5.49	4.08	2.69	6.24	1.65	2.18	4.87	2.67	2.84	4.37	5.72
2003- 2004	3.05	19.82	24.29	7.87	4.18	5.02	5.79	19.26	5.49	2.30	3.74	8.68	9.06
2004- 2005	10.02	11.31	6.25	10.18	2.79	5.00	2.49	1.86	5.21	3.80	12.14	20.00	7.63
2005- 2006	17.44	4.65	3.82	15.93	4.27	12.41	3.06	2.13	4.01	11.99	11.83	10.89	8.60
2006- 2007	27.03	10.65	3.93	4.21	4.23	2.61	1.50	1.35	3.21	2.21	1.41	8.16	5.93
2007- 2008	23.55	8.59	7.18	7.75	2.92	3.20	1.66	0.94	1.97	3.04	6.05	7.37	6.23
Minimum	2.67	2.44	2.52	2.22	1.83	1.79	1.08	0.94	0.84	1.16	1.41	2.08	3.28
Mean	11.68	8.93	8.78	6.48	4.71	4.45	3.09	3.83	4.73	4.78	5.56	8.33	6.29
Maximum	27.03	19.82	31.60	20.49	14.86	16.21	8.40	19.26	21.26	16.18	12.77	20.00	10.01

Table 3.6 Modelled change in monthly mean flows below the dam site when the Scheme is at full uptake

Hydrological Year	Jul (m^3/s)	Aug (m^3/s)	Sep (m^3/s)	Oct (m^3/s)	Nov (m^3/s)	Dec (m^3/s)	Jan (m^3/s)	Feb (m^3/s)	Mar (m^3/s)	Apr (m^3/s)	May (m^3/s)	Jun (m^3/s)
Mean Inflow	11.68	8.93	8.78	6.48	4.71	4.45	3.09	3.83	4.73	4.78	5.56	8.33
Mean Outflow	6.98	6.19	8.23	7.51	6.83	8.23	8.76	8.31	6.37	4.06	1.39	2.83
Difference outflow-inflow	-4.70	-2.74	-0.55	1.03	2.12	3.78	5.67	4.48	1.64	-0.72	-4.17	-5.50

The corresponding flow duration curves (i.e. cumulative probability curves of the flow distribution) are shown in Figure 3.9. Compared with the inflow duration curve, the dam outflow duration curve clearly reflects the effect of storage regulation. There is a step in the outflow duration curve at 11.2 m^3/s which indicates that the dam outflow would be sustained constantly at this flow for significant periods (about 9% of the time). At the low flow end, the dam outflow would be maintained at the environmental residual flow of 1.23 m^3/s for prolonged periods (i.e. some 35% of the time). During very low flow periods, it might be noted that the managed outflow regime can be higher than the pre-existing flow regime; this flow augmentation effect is anticipated to occur for about 3% of the time (10 to 12 days per year on average).

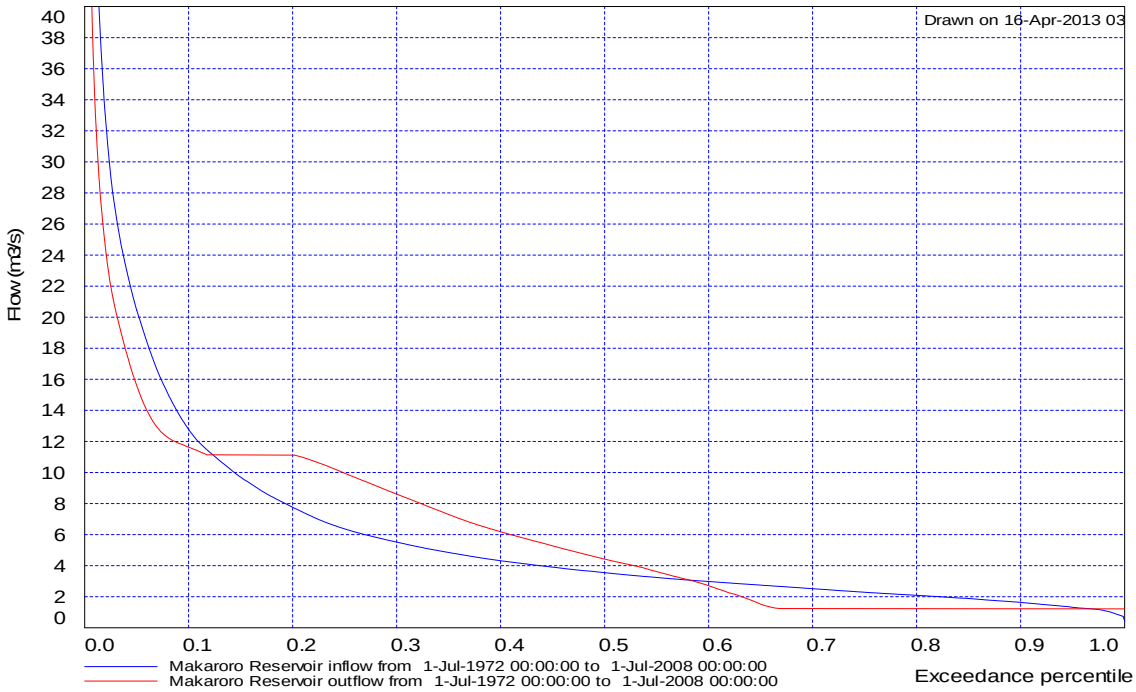


Figure 3.9 Makaroro storage dam outflow (red) versus inflow (blue) duration curves

3.2.4 Downstream river flow regime

Changes in the flow regime immediately below the dam translate to equivalent flow changes over the full length of the river channel downstream of the dam to the sea, including the Waipawa and Tukituki Rivers. Water would be abstracted at the upper and lower Waipawa intakes to supply irrigation therefore imparting further changes to the flow regime downstream.

The assessment of the pre and post scheme river flow regimes for downstream locations are described in separate reports viz. "Ruataniwha Water Storage Scheme: Environmental Flow Optimisation" (Aquanet, May 2013) and "Ruataniwha Water Storage Scheme: Tukituki River Catchment: Assessment of potential effects on groundwater and surface water resources" (HBRC, 2013c).

3.2.5 Hydropower generation operation

Provision is made within the Application Design Scheme for hydropower generation as a secondary function of the storage dam. Effectively, generation is treated as an incidental

by-product of irrigation operation. Nevertheless, a small buffering storage comprising the top 0.20 m of the reservoir operating range has been provided to enable capture and diversion to the turbines a proportion of the flow that would otherwise be spilled when the reservoir was close to full.

Compared with an equivalent storage dam without a hydropower buffer, or where there is no hydropower generation, when the reservoir is close to being full, the reservoir levels will fluctuate frequently within a tight 0.20 m range (i.e. between RL 469.3 m and RL 469.5 m). Figure 3.10 provides a sample comparison of the simulated reservoir level behaviour for "with hydro" and "without hydro" cases.

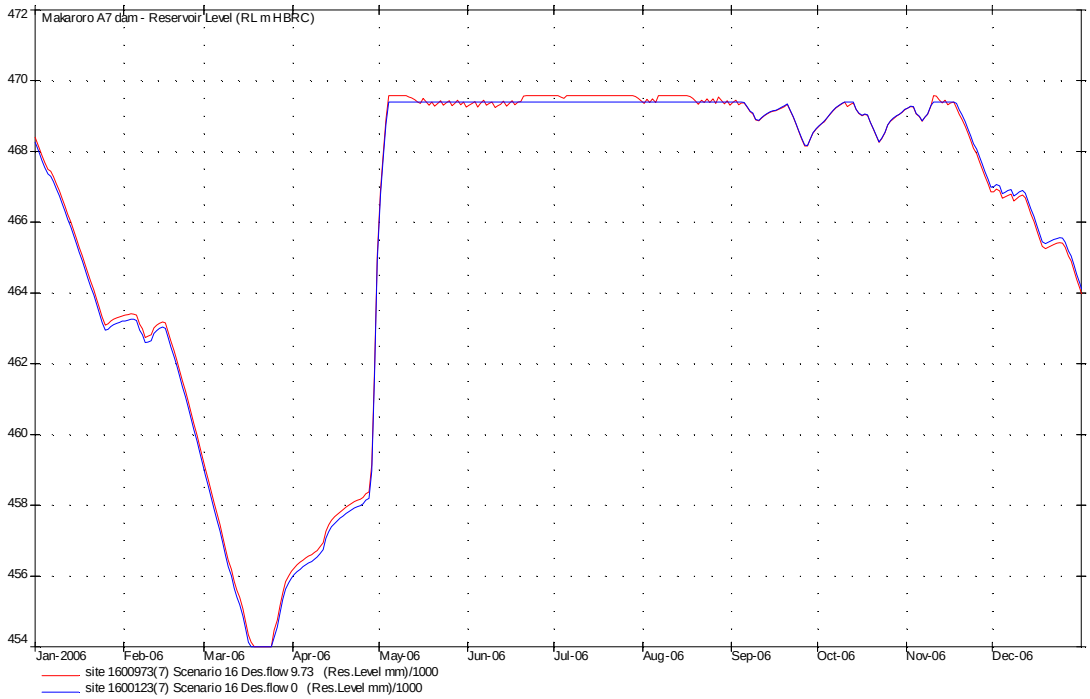


Figure 3.10 Comparison of the modelled reservoir levels for the Makaroro Dam site with (red) and without (blue) hydropower add-on operation for the 2006 year (sample period)

Figure 3.11 provides a sample comparison of the simulated dam outflows for "with hydro" and "without hydro" cases.

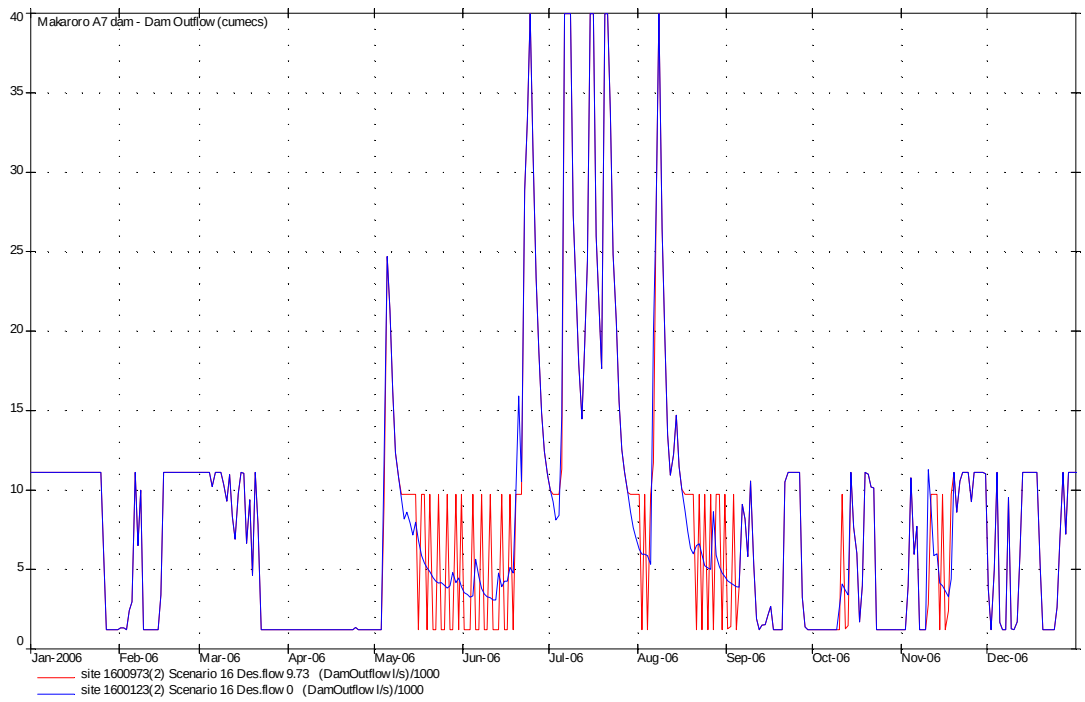


Figure 3.11 Comparison of the modelled dam outflow for the Makaroro Dam site with (red) and without (blue) hydropower add-on operation for the 2006 year (sample period)

With regard to the potential for daily peaking operation, considering the irrigation demand pattern and timing, the potentially feasible period for daily peaking operation would be between mid-May and mid-September, a period of 4 months each year outside of the main irrigation season. However, within this period, given that the period up to mid-July is often spent refilling (and inflows retained in the reservoir except for the residual flow), there is effectively only a period of around 2 months each year where daily peaking may be consistently possible viz. mid July to mid-September.

Daily peaking operation, if any, will not be reflected in the preceding results since the simulation model operates on a daily time step and peaking is an intra-day feature.

3.3 Water supply to downstream users

3.3.1 Overview

The production land use area of the Scheme comprises Zones A to D and Zone M, as shown on Figure 2.1. The Scheme is also able to supply water to other existing consent holders located downstream of Zones A to D on the Upper Tukituki, Waipawa and Tukituki Rivers. Their takes are controlled by minimum flows at a number of reference streamflow gauges. As part of the Tukituki Plan Change, it is intended to increase the minimum flows at these gauges, which would impact adversely on the reliability of these consented takes. Water can be released from the Scheme to provide water to downstream users, thus managing (reducing or eliminating) the additional number of days that downstream water users would have otherwise been unable to take surface water.

Figure 3.12 shows the locations of these consents, which have been divided into 5 zones along the Waipawa, Upper Tukituki and Tukituki Rivers. The 5 zones together with the reference river flow gauges used to regulate abstraction are listed in Table 3.7.

Table 3.7 Downstream zones considered for supply from the RWSS

Zone	Description	Flow monitoring gauge	Gauge flow below which abstractions are banned (l/s)		
			Current	Interim (1/7/18-30/6/23	Future
(1)	Tukituki River below Red Bridge	Site no. 2301, Tukituki at Red Bridge	3,500	4,300	5,200
(2)	Lower Tukituki corridor (Red Bridge to Zone M)				
(3)	Tukituki River adjacent to Zone M				
(4)	Upper Tukituki corridor	Site no. 23207, Tukituki at Tapairu Road	1,900	N/A	2,300
(5)	Waipawa downstream Zone A and upstream of Zone M	Site no. 23235, Waipawa at RDS	2,300	N/A	2,500

3.3.2 Assessment

In relation to this possibility, HBRC commissioned T&T to undertake a study to:

- determine the practicality and approximate costs of providing water from the Scheme to the affected downstream users and identify the preferred conveyance path to deliver water to each downstream zone;
- assess the increase in banned abstraction days when the minimum flow at the reference flow gauges is increased to the interim and future minimum flows;
- estimate the water demand and additional amount of live storage at the dam needed to supply these existing users for two cases: (i) so as not to increase the number of ban days they currently face, and (ii) to eliminate the ban days altogether;
- assess the impact on the supply capability of the Scheme and the reliability of supply to the main command area (Zones A-D and M) if these downstream users were supplied from the Scheme;
- model and take into account the potentially unavoidable losses associated with the considerable conveyance distances involved (i.e. related to the significant time lag and ramp-up and ramp-down of the flow releases at the demand point); and
- provide a perspective on the relative cost of supply per cubic metre to these downstream zones compared with the main command area.

The detailed findings from this study are in the report "Ruataniwha Water Storage Project: Water Supply to Downstream Users" (T&T, November 2012). The main conclusions are:

i. The impact on downstream users of raising the minimum flow is significant, especially for zones which are controlled by the minimum flow at Red Bridge, i.e. the average number of banned days increases by about 5 days and 24 days per year under the interim and future minimum flows respectively, from about 2 days per year currently. For zones which are governed by the minimum flow at RDS and Tapairu Road, the increase in banned days under the future minimum flow is around 10 days per year, from about 21 days per year currently.

ii. Although there are operational challenges presented by the considerable distance to the downstream zones, it is possible to effectively supply the downstream users from

the Scheme by upgrading sections of the originally proposed distribution infrastructure within Zones A and B and utilising the non-losing reaches of the Waipawa and Tukituki Rivers for water conveyance.

iii. Supplying downstream users to reduce or eliminate banned abstraction days will add a moderate burden on the supply capability of the Scheme; there will either be an equivalent contraction in the primary irrigation supply volume (95.8 million m^3) or a reduction in the reliability of supplying that same volume to the main service zones as a result. The reduction is up to 4.9% of the original annual supply volume for the case whereby all banned days are eliminated under a future minimum flow scenario.

iv. By incorporating the anticipated operational losses associated with conveyance to the downstream zones (mainly from flow ramp-up and ramp-down), the additional live storage required increases by only a modest amount, i.e. by about 0.1 million m^3 for the case where all banned days are supplied.

v. The additional capital cost to enable supply to the downstream users is made up of 2 components i.e. the upgrades required to the distribution system in Zones A and B, and the equivalent cost of the additional live storage needed to maintain reliability and volume of supply to the core service zones.

vi. There is a strong rationale for assigning a higher value per m^3 of water supplied to the downstream users compared with that to the main service zones because: (i) the water needed from storage by the downstream users represents only a small proportion of their overall water needs, and (ii) it is likely that stored water will have a very high value at the time it is required by the downstream users i.e. during low flow events and peak irrigation periods.

vii. The assessment is preliminary in a number of respects, and there remain opportunities to optimise the delivery pathway and associated distribution infrastructure required for supplying downstream users.

3.3.3 Provisions for Supply to Downstream Users

On the basis of the above assessment, particular provisions have been made in the proposed Scheme to supply existing downstream water users. The additional elements of infrastructure required to enable supply to these users include, inter alia, the outfall structures adjacent to the Kahahakuri and Mangaonuku Streams as described in Section 5 of this report.

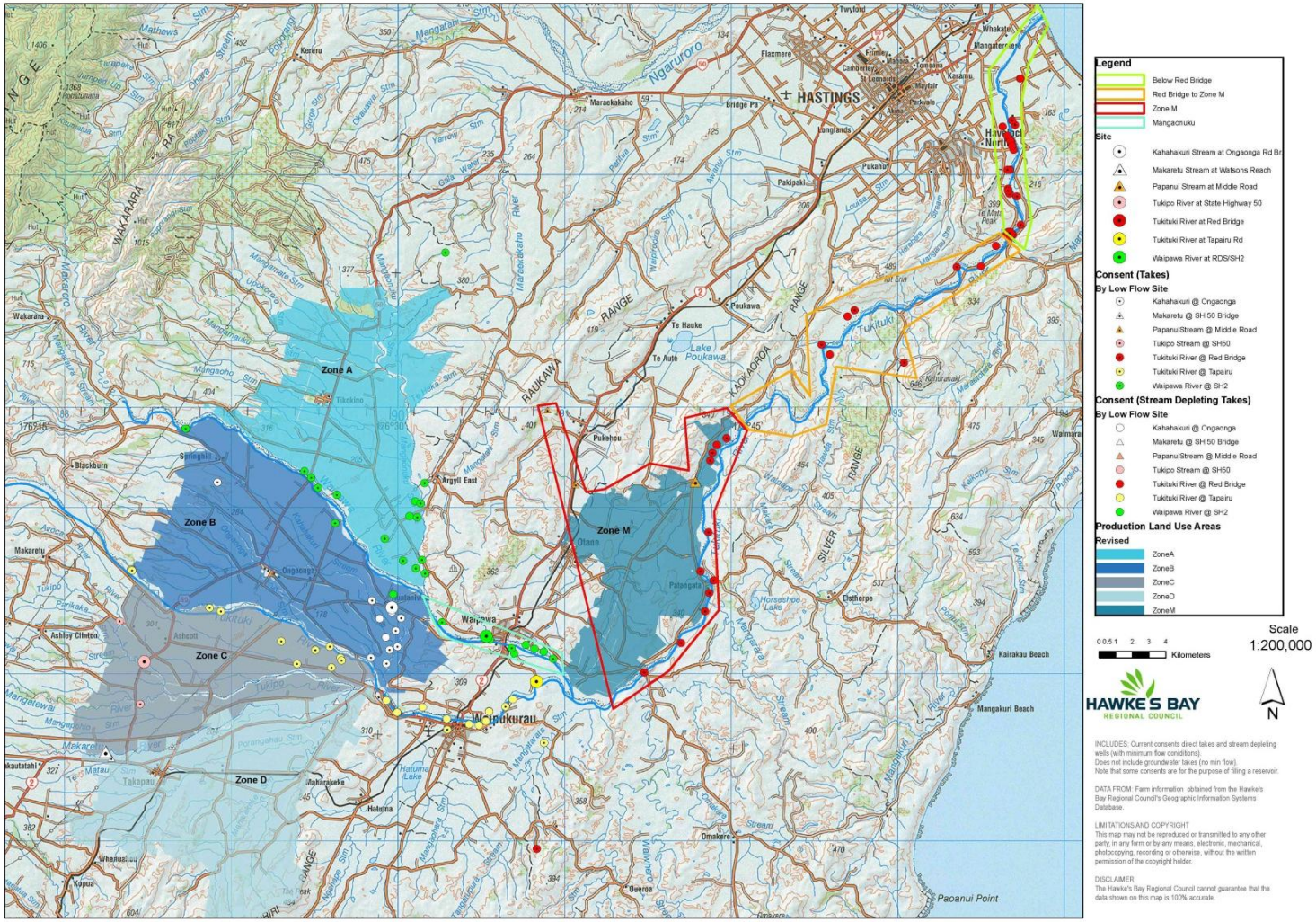


Figure 3.12 Location of existing downstream water users and delineation of zones (Source: HBRC)

4 Dam design

4.1 Introduction

A Concrete Face Rockfill Dam (CFRD) has been selected for the Application Design. Drawings of the proposed scheme are presented in Appendix A. Table 4.1 summarises the key dam characteristics and criteria for the Application Design.

The dam is approximately 83 m high measured from the top of the parapet wall to the existing river bed level at the toe of the dam. The crest is 505 m long and 8 m wide. The top of the parapet wall is 475.5 mRL (minimum) and extends 1.2 m above the top of the embankment.

The dam type, location and general arrangements are heavily influenced by the geotechnical site conditions. Of particular importance are the seismic faulting and secondary displacements that have been addressed in this design. The geological conditions influencing the design and the associated risks are summarised in Section 4.2 of this report. Design solutions have been developed to manage these risks and these are described in the following sections.

Table 4.1 Key dam characteristics

Item	Description
Dam type	CFRD
PIC	High
Spillway type	Concrete lined primary spillway ogee weir (ungated); unlined auxiliary spillway for flood events larger than the 1 in 200 AEP plus climate change
Spillway capacity	PMF (Approximately 752 m^3/s peak outflow)
Full supply level	469.5 mRL
Approximate dam storage volume at FSL	90,700,000 m^3
Approximate buffer storage for hydro generation	700,000 m^3 corresponding to 0.2m water depth
Dam height	Approximately 83m at rivers deepest point
Approximate reservoir area at FSL	3,700,000 m^2
Dam crest level (top of parapet wall)	475.5 mRL (plus 0.5 m camber)
Dam crest width	8 m
Dam crest length	505 m
Dam batter slopes	1V: 1.5H

4.2 Geological and geotechnical conditions

A summary of the key findings from the "Report on Engineering Geological Investigations A7 Site" (T&T 2012) affecting design is outlined below.

- The Mohaka fault is located to the northwest some 700 to 800 m from the right abutment of the dam and crosses the reservoir 1 to 2 km upstream (north) of the dam

- A mappable zone of rock mass weakness (sz1) is identified at the eastern (uphill) edge of the left abutment terrace alignment roughly parallel to the river. The dam alignment has been selected so that the sz1 zone passes through the upstream toe slab at terrace level rather than on the steep face of the gorge
- Preliminary stability analyses based on a seismic refraction survey and inspection of exposures indicate that a 90 m high landslide located on the right bank of the reservoir approximately 0.5 km upstream of the dam could remobilise during reservoir filling or be triggered by the maximum design earthquake. The current assessment of freeboard required for the seismic scenario is based on stabilisation of the landslide as included in the Application Design. The areal extent of the works required is shown on Drawing 27690-DA-104. The method and requirement for landslide stabilisation is an area of optimisation to be considered during detailed design. The method for stabilisation is likely to involve regrading (flattening) the landslide face and buttressing it with material from the regrading operation. Optimisation in respect of this landslide would entail further site investigation, design and analysis to review of the scale of movement and likelihood of wave generation relative to freeboard requirements.
- An area of dilated, potentially unstable rock is identified on the left bank wall of the gorge extending from the downstream toe of the embankment a further 200 to 250 m downstream. Part of this area has failed sometime in the past, most likely as a result of earthquake shaking. There is an expectation, based on visual inspection that the remaining area of dilated rock mass could mobilise during the maximum design earthquake. Therefore the Application Design ensures that the spillway and outlet works return flow to the Makaroro River downstream of this potential rockfall area. This will allow normal operation of the spillway and outlet works during such an event
- The high earthquake loadings on the embankment are in general due to the proximity of the Mohaka and other active faults in the vicinity of the dam. Embankment dams on rock foundations have a high resistance to earthquake loading and allowance for high earthquake loading has been incorporated in the batter slopes as noted in Section 4.4.3
- Core from the drilling investigations show the greywacke sandstone rock mass to be mainly unweathered, moderately strong to strong , with closely spaced, mostly tight defects and therefore likely to require quarry operation type blasting in order to excavate
- Defect data collected during engineering geological mapping has recorded the presence of non-persistent low angle defects in the greywacke rock mass. This may have implications for localised plinth stability, and this will be assessed further during the detailed design stage to enable appropriate detailing to address this issue.

4.3 Dam design criteria

The general design criteria for the works are based on the New Zealand Society on Large Dams (NZSOLD, 2000) Dam Safety Guidelines and also considerations of the Building (Dam Safety) Regulations 2008. These provide guidance on general dam design and construction practice including the assessment of flood and seismic hazards and the provision of appropriate spillway and diversion works and adequate defence against earthquake events. They also provide guidance on the establishment of dam safety surveillance practices to be adopted for initial filling of the storage and subsequent operation and for the provision of emergency action plans.

The design will also be in accordance with the New Zealand Building Act where applicable.

Where the NZSOLD Dam Safety Guidelines do not cover specific design details, the design will be based on international guidelines and practice documents. The International Commission on Large Dams (ICOLD) publications provide a wide range of guidelines that reflect current international practice. Specific to the CFRD detailed in the Application Design ICOLD Bulletin 141 (2010) would be a primary reference for dam design and construction. Other sources may include appropriate guidelines and standards from:

- The US Federal Energy Regulatory Commission (FERC)
- The US Bureau of Reclamation (USBR)
- The US Army Corps of Engineers (USACE)
- The Australian National Committee on Large Dams (ANCOLD).

These organisations are recognised both internationally and in New Zealand as producing authoritative standards and guidelines that represent current best practice.

The dam is assessed as being within the HIGH Potential Impact Category (PIC) for dams as defined by Table III.1 of NZSOLD (2000) Dam Safety Guidelines. This is the highest classification in the Guidelines and the design of the proposed works is based on this rating. A High PIC requires the dam to have sufficient spillway capacity to handle a flood between the 1 in 10,000 AEP Flood and the Probable Maximum Flood (PMF). The Application Design adopts a Maximum Design Flood (MDF) equivalent to the PMF therefore meeting NZSOLD guidelines. The Application Design has been peer reviewed by independent experts. Refer to Tonkin &Taylor, August 2012b for peer review reports and scope.

In summary the following organisations or individuals peer reviewed the following aspects of the dam design:

- Dr David Painter – Hydrology
- Engineering Geology Ltd (Dr Trevor Matushcka) – Dam design
- Damwatch Services Ltd (Murray Gillon) – Seismic design
- NIWA (Dr Murray Hicks) – Sedimentation assessment

A dam break assessment has been undertaken by HBRC (2013a). Results of the assessment support the assumption of a High PIC.

4.4 Arrangement of selected dam

4.4.1 General

The Application Design is presented in Drawings 27690-DA-100 to 502 in Appendix A. The specific arrangements, dimensions and details shown in drawings and described in this section are subject to the introductory comments made in Section 1.1.

The main features of the Application Design of the dam are:

- An 83 m high CFRD dam
- A concrete lined primary spillway located on the right abutment of the dam
- An unlined auxiliary spillway located on the left abutment of the dam
- A concrete intake structure located within the reservoir
- A 4 m diameter diversion tunnel located under the dam, also housing operational outlet works.

4.4.2 Freeboard

The rationale behind the primary spillway, auxiliary spillway and dam crest levels adopted for the Application Design are summarised in this section.

The following spillway arrangements have been assumed for flood routing:

- 25 m wide ogee crest on the primary spillway (narrows to a 15m wide chute), crest level = 469.5 mRL (Full Supply Level)
- 50 m wide auxiliary spill channel, crest level = 472.75 mRL (flood level for OBF).

Assumed starting water level during scenario events = 469.5 mRL.

The routing results for the Operational Basis Flood (OBF) and Maximum Design Flood (MDF) are presented in Figure 4-1 and Figure 4-2 below. The OBF is equivalent to the 1 in 200 AEP event with an adjustment for climate change. The key results are summarised:

<u>OBF</u>

- Peak inflow 413.5 m^3/s
- Peak outflow 305.5 m^3/s (primary spillway only, zero flow over auxiliary spillway)
- Flood rise 3.25 m to 472.75 mRL

<u>MDF</u>

- Peak inflow 794.6 m^3/s
- Peak outflow 585.1 m^3/s down the primary spillway, 167.1 m^3/s down the auxiliary spillway
- Flood rise 4.92 m to 474.42 mRL.

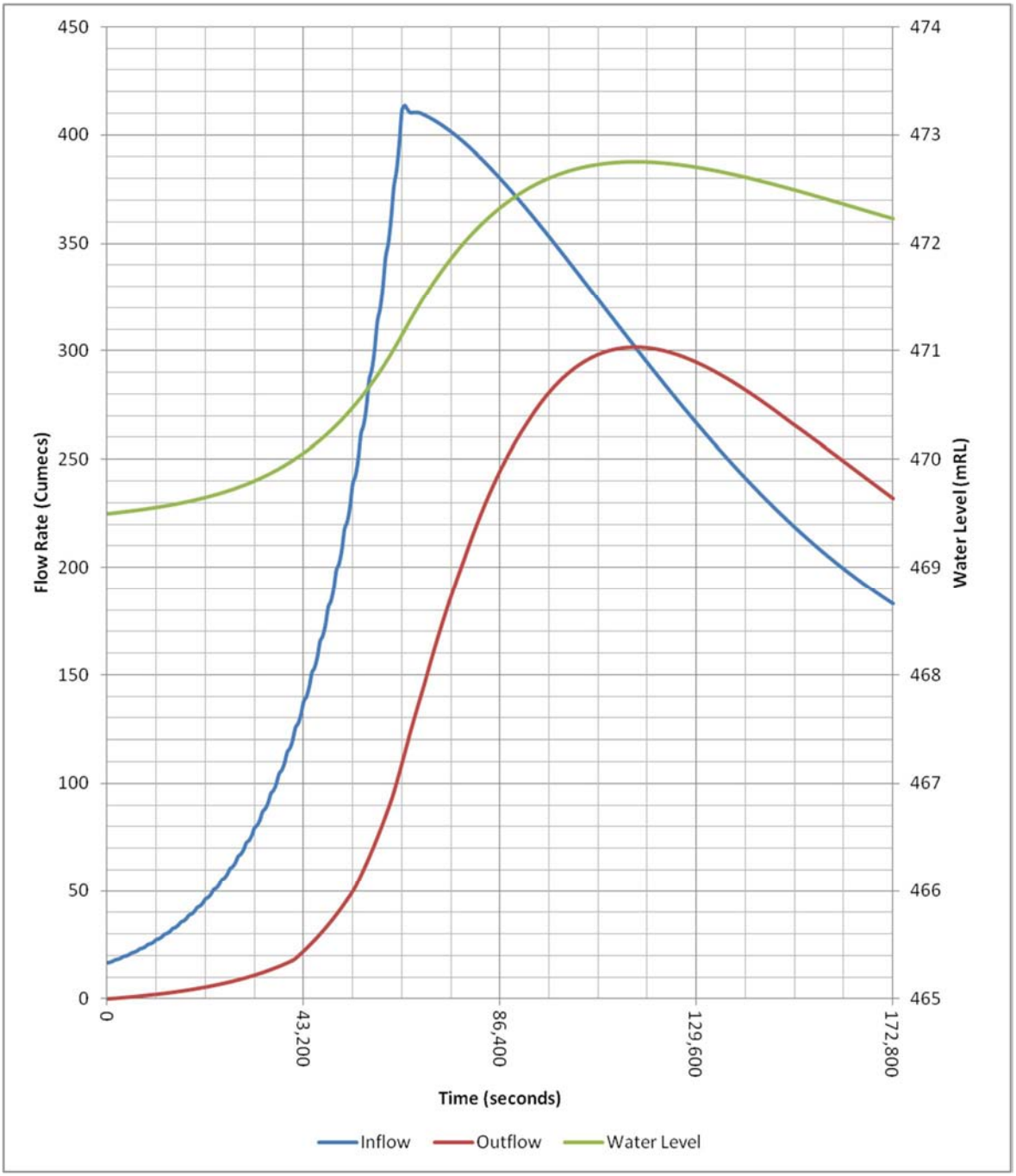


Figure 4-1 Routing for Operational Basis Flood (1 in 200 AEP plus climate change)

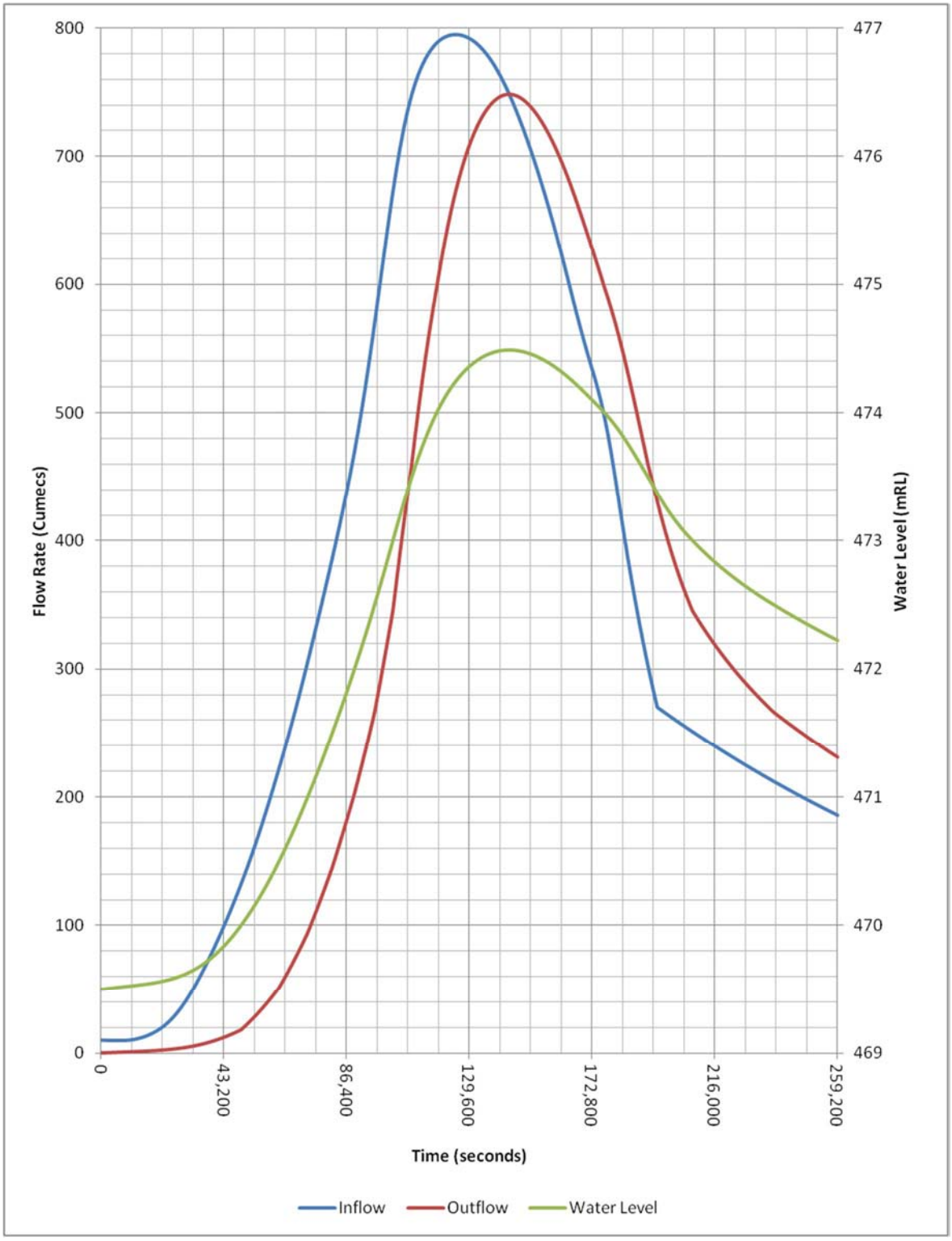


Figure 4-2 Routing for Maximum Design Flood (PMF)

Based on the assumptions above, the Maximum Design Flood is critical in terms of setting the elevation of the top of the dam. On this basis, the top of the parapet wall has been set at 475.5 mRL, which provides 6.0 m freeboard to Full Supply Level, and 1.08 m freeboard to MDF.

The OBF and PMF flood criteria to be adopted will be reviewed during detailed design.

4.4.3 Dam batter slopes

The Application Design embankment layout and details of the cross section are shown at Drawings 27690-DA-100 and 201. Embankment batter slopes of 1.5H:1.0V have been adopted at this stage. This is a relatively flat batter for the downstream rockfill but a conventional slope for the upstream gravel used in Zone 2B (Drawing 27690-DA- 201) under the concrete face slab. There are some differences of opinion on the value of flatter slopes for earthquake loading and some optimisation will be considered during the detailed design stage. The crest width of 8 m is wider than the conventional 6 m width for CFRDs of this height, reflecting the high earthquake loading and following recommendations by Materon and Fernandez (2011).

ANCOLD (1991) notes that in the drained condition CFRD embankments have a factor of safety against sliding of around 7 under reservoir load. Stability analyses are not normally required for CFRD construction unless unfavourably oriented joints or defects are present in the foundation. The possible low angle defects in the underlying greywacke are one example (if they proved more extensive than current indications suggest) while other cases may arise from foundation excavations.

Stability analyses will also be required during future design stages for dynamic loading under earthquake. Dynamic analyses during the detailed design stage could be used during the optimisation process to confirm the need for the flatter downstream slope and wider crest and investigate alternatives such as reinforcement with geogrids.

4.4.4 Spillways

4.4.4.1 Spillway arrangement

Two spillways are proposed, a primary spillway that operates for all floods (when the reservoir is at or above Full Supply Level) and an auxiliary spillway that operates only during very large floods (greater than the Operational Basis Flood). The combined spillways are designed to handle the 752 m^3/s outflow discharge from the Maximum Design Flood (MDF) with the primary spillway taking 585 m^3/s and the auxiliary spillway taking 167 m^3/s.

4.4.4.2 Primary spillway

The Application Design primary spillway arrangement and sections are as shown on Drawings 27690-DA-300 to 302. The primary spillway has a fan shaped contraction controlled by a 25 m long arc-crested concrete ogee weir set at the Full Supply Level of 469.5 mRL. The chute contracts from the 25 m wide crest to a 15 m wide chute that terminates in a hydraulic jump style dissipator with an unlined discharge channel leading to the river. The dissipator and channel excavations provide rockfill for the embankment. The discharge point to the river from the spillway is located downstream of the dilated rock mass referred to in Section 4.2 to ensure normal operation of the spillway and outlet works following any collapse of this rock mass. One of the reasons the primary spillway has been located on the right bank of the river rather than left is to avoid frequently inundating the ground above the dilated rock mass.

The primary spillway discharge is fully contained within a concrete lining from the ogee crest to the dissipator. The dissipator is a 40 m long concrete box located at a level that ensures there is sufficient tailwater depth to stabilise the hydraulic jump for discharges up to the 1 in 1000 AEP flood. More extreme floods would result in high velocity flows passing down the unlined channel but still be contained within the sound rock excavation. Some scour would be expected for these discharges but the need for remedial work is considered unlikely. The length of the concrete box is shorter than typically recommended (USACE EM

110-2-1603, USBR Design of Small Dams 1987) because the sound rock is considered unlikely to be scoured by any residual energy remaining beyond the 40 m length specified.

The chute floor has a minimum thickness of 300 mm and is anchored to the foundation with reinforcing bars grouted 3 m into sound rock. A typical anchor bar arrangement uses 24 mm diameter reinforcing bars on a 2.2 m grid. The chute can be slip formed in relatively long lengths or can be constructed of individual slabs (typically 10 m by 15 m). Side walls consist of a 300 mm thick layer of reinforced shotcrete, anchored to sound excavated rock with dowel bars similar to those used for the chute floor. A typical reinforcement arrangement would consist of 20 mm diameter bars at 250 mm centres located in the water face only of the floor and wall slabs.

Chute drainage consists of an array of 75 mm diameter holes on a 4 m grid drilled 3 m into the foundation and sloping upstream. The holes act as eductor drains and largely eliminate pipe drains.

4.4.4.3 Auxiliary spillway

The Application Design auxiliary spillway (refer Drawing 27690-DA-100) is a simple 50 m wide unlined excavation in rock on the far left abutment. It discharges into a heavily treed gully that leads back to the river. The crest of the auxiliary spillway is set at RL 472.75 m, 3.25 m above the primary spillway crest. This ensures that the auxiliary spillway operates only for very large floods that exceed the Operational Basis Flood (1 in 200 AEP event plus climate change).

Operation of the auxiliary spillway will be infrequent (greater than the 1 in 200 AEP flood). When it does operate there may be erosion in the unlined spillway chute. Downstream of the excavation the flow discharges into a steep gully and through a large stand of trees before joining the river. The gully is an unusual shape from a geotechnical viewpoint and there has been no sub-surficial investigation in this area. Considerable scour would be expected in the gully during an extreme flood event and damage to the stand of trees. A preliminary assessment of scour has been made. The approximate areal extent of long term erosion downstream of the auxiliary spillway is shown in Figure 200 (Appendix C). The actual erosion will be highly dependent on:

- Frequency and duration of flood events requiring the auxiliary spillway to operate, and;
- The depth and quality of bedrock

Further geotechnical investigation is required in the auxiliary spillway area to confirm the rock quality in relation to the erodibility of the spillflow path. Optimisation of the auxiliary spillway requirement and location will be reviewed during detailed design. Specifically a greater capacity primary spillway will be reviewed during detailed design as an alternative to the auxiliary spillway. This option would avoid erosion in the auxiliary spillway location and reduce the overall construction programme.

As noted, the threshold flood event for the auxiliary spillway to commence operation is the 1 in 200 AEP flood event under a future climate scenario (Scenario A1B by 2040). This event is approximately equivalent to the 1 in 450 AEP flood if assessed against flood frequency based on a statistical analysis of historical floods. Floods smaller than this event will not result in any overflow at the auxiliary spillway nor any erosion within the spillpath.

The floodpath from the auxiliary spillway in the 1 in 1000 AEP flood (with climate change) has been modelled and is shown on the right pane in Figure 200 in Appendix C. The peak

auxiliary spillflow in this event is relatively modest at around 21 m^3/s, representing 6% of the total peak outflow from the dam (385 m^3/s). The auxiliary spill lasts for some 20 hours. However, spillflows during the event are typically much less than the peak flow (viz. in the rising and falling limbs of the hydrograph). It is doubtful that the duration of elevated spillflows would be sustained for long enough to result in the extent of "excessive" erosion as indicated in the plan.

By comparison, in the modelled MDF event (equal to the Probable Maximum Flood), shown on the left pane of Figure 200 in Appendix C, the peak auxiliary spillflow is 167 m^3/s and the spillflow lasts for in excess of 30 hours. This is very likely to result in considerable erosion damage.

Peak spillflows for other intermediate but still extreme flood events are shown in Table 4.2.

Table 4.2 Peak auxiliary spillflow versus flood frequency

Flood Frequency (with climate change)	Peak Auxiliary Spillflow (m^3/s)
1 in 200 AEP	0
1 in 250 AEP	1.2
1 in 300 AEP	3.0
1 in 500 AEP	10
1 in 1000 AEP	21
1 in 2000 AEP	34
1 in 5000 AEP	53
1 in 10,000 AEP	66
Probable Maximum Flood	167

With reference to Table 4.2, significant erosion is not anticipated as a result of the operation of the auxiliary spillway in any flood less than a 1 in 300 AEP (with climate change) flood event. On a catchment wide scale, a rainstorm and consequent flood event of this threshold magnitude (1 in 300 AEP flood with climate change) is expected to result in mobilisation of massive amounts of debris (sediment and vegetation) under pre-existing conditions, and the potential additional erosion and vegetation removal caused by auxiliary spillway operation under such extreme hydrological events should be considered in this context.

4.4.5 Outlet works

The Application Design outlet works are as shown on Drawings 27690-DA-100 and 401-404. The outlet works are located within the diversion tunnel and outlet channel construction. The 4 m tunnel diameter is capable of housing the 2100 mm diameter penstock and the 600 mm diameter bypass pipe in a concrete surround but is too small for a free standing penstock allowing access to the intake tower. Outlet works components include:

- A low level submerged intake structure at the upstream portal of the diversion tunnel. This is essentially a reinforced concrete box frame structure with trash racks and a 2.1 m square vertical slide gate. The gate is used to close off the river once the

dam is constructed and to provide an outlet facility at low storage levels. At this stage, it is assumed that the gate will be hydraulically operated. The gate needs to handle the full storage water load (70 m head approx.) but would only operate under balanced conditions with the tunnel pressurised. The intake also houses the entrance to the 600 mm diameter bypass pipe

- A free standing high level wet intake tower founded at 440 mRL at a high point above the diversion tunnel. The tower is connected to the tunnel by a 2.6 m diameter raised bore shaft containing a 2100 mm diameter steel liner in a concrete surround. The tower is expected to be slip formed (or jump jack). Selective withdrawal facilities (intake ports) are proposed at two levels operated by a trash rack and baulk system. Dual ports are provided at each level to restrict port velocities to around 3 m/s. Baulks would close off the ports not being used while trashracks would be installed in operating ports. Trashracks and baulks would be placed and removed using a 6t travelling crane on a steel portal frame. The tower could be dewatered by installing baulks over the 4 ports and closing the slide gate
- Access to the tower is by boat and a system of ladders and platforms
- A 2100 mm diameter steel liner within the tunnel from the grout curtain to the downstream portal. The annulus between liner and tunnel is backfilled with concrete
- A 600 mm diameter bypass pipe located within the diversion tunnel lining to provide flow maintenance releases during outlet construction and a low discharge outlet during normal operation. The bypass also provides a discharge facility that is independent of the main penstock system allowing the latter to be removed from service for maintenance
- At the downstream portal, the 2100 mm diameter steel liner bifurcates to provide:
  - A 1500 mm diameter penstock leading to a valve chamber and dissipator box with the former housing the guard valves and interconnection pipework and the latter containing the fixed cone dispersion (FCD) control valves. The 600 mm diameter bypass is housed in the same valve chamber and dissipator box
  - A 1500 mm diameter penstock leading to the hydro power plant.

The outlet penstock has been sized to ensure the following:

- Ability to dewater the reservoir within 3 months based on criteria developed for the USBR by ACER (1990)
- Reduce head losses to an acceptable level (< 2m) for power station operation.
- The peak irrigation release required of up to 13 m^3/s. This can be provided concurrently with a residual flow of 1.23 m^3/s either via the 1.5 m diameter FCD valve bypassing the power station for reservoir levels above about 407.5 m RL (gross storage capacity about 0.5 million m^3 at that level), or in combination with the power station when the reservoir level is above about 435 m RL (insufficient head for power generation below this level). The discharge requirement during environmental flushing is up to 30 m^3/s depending on the concurrent flow in the tributaries of the Tukituki River below the dam and upstream of Red Bridge. Depending on the concurrent level of inflow from these tributaries, which include the major tributaries upper Waipawa and the upper Tukituki, the irrigation releases may need to be cut back for the duration of the flushing (approximately 9 hours). A flow of 30 m^3/s can be released through the 1.5 m diameter FCD valve for reservoir levels above 432.3 m RL (gross storage capacity about 12 million m^3). Again, this is an area of potential optimisation.

4.5 River diversion during construction

4.5.1 River diversion strategy

The design inflow floods adopted for river diversion during construction are selected to suit the different phases of construction in order to reflect the risk involved and the period of exposure to that risk. There are many different strategies for river diversion that may be optimised with Contractor and HBRIC Ltd input during the detailed design phase.

Initial site works, where expected damages from overtopping failures are low and the period of exposure is short, can apply flood values corresponding to recurrence periods of 2 to 5 years. When construction activities are well advanced and the embankment height is large and could store significant volumes of water during a large flood event, failure from overtopping can cause major damages at the construction site and in the river downstream.

Formal guidelines on selection of construction floods are limited. Furthermore, there have been few large dams constructed in New Zealand in recent years. Therefore, to a certain extent; decisions made for Makaroro Dam will contribute to establishing modern industry practice for the country. Based on an assessment of world practice, NSW Dams Safety Committee (Demonstration of Safety for Dams – DSC2D Section 6.17), whose guidelines are similar to ANCOLD's, advise that they will accept a flood capacity, during those phases of construction with public safety at risk (i.e. during flood events larger than the river's capacity), in the range of the 1 in 500 to 1 in 1,000 AEP flood discharge, provided the risks are as low as reasonably practicable. Therefore in accordance with the NSW Dams Safety Committee publication, for the Application Design a construction flood of 1 in 1000 AEP has been adopted for the Application Design phases where public safety and major damages are at risk.

The Application Design diversion works provide a tunnel and a 10.5m high upstream cofferdam to divert water around the dam site and allow initial river bed work to commence. The initial dam construction comprises a major rockfill cofferdam, referred to as the downstream stage, located within the downstream shell of the embankment. The downstream face of the rockfill is provided with steel reinforcement to permit large floods to safely overtop the downstream stage. This downstream stage provides the main protection for the downstream river during construction and is taken to a height that enables the tunnel alone to provide adequate protection to the works.

Reinforced rockfill is widely used for cofferdam construction to protect the rockfill and any underlying materials from the erosive effects of flowing water. Meshed rockfill is generally considered safe for overtopping depths of 3.0 metres or upstream heads of 4.5 m (ICOLD, 1984). The river flows at Makaroro Dam are not large compared with CFRD dams constructed internationally. As explained below, the 1 in 1000 AEP peak inflow discharge passed over the reinforced downstream stage (to 429 mRL) would produce a maximum upstream head of 2.1 m, well within the capacity of reinforced rockfill.

4.5.2 Proposed diversion works and construction program

The Application Design arrangements for diversion works is outlined below and consists of:

- A 4.0 m diameter concrete lined tunnel through the right abutment that is also capable of housing the permanent outlet works
- A 600 mm diameter bypass pipe located in the tunnel lining to provide environmental flows during outlet works construction and during maintenance of the main penstock during operation
- A low height (10.5 m high) upstream cofferdam with an initial crest level of 408 mRL, giving a maximum storage height of 10.5 m to divert normal river flows and small floods (up to approximately 80 m^3/s) through the tunnel
- A downstream stage with crest at 429 mRL and steel reinforced downstream face to enable large floods to be passed over the coffer dam.

The Application Design dam construction program is outlined below and shown diagrammatically at Drawing 27690-DA-502:

Phase 1 involves construction of the diversion tunnel and associated channels and stripping of the right abutment of the dam. The embankment construction can commence on the right abutment using rockfill from the excavations with excess rockfill and suitable gravels stockpiled for later use. Other works include toe slab construction on the abutments, excavation and treatment of the sz1 zone and construction of the low level intake tower.

Phase 2A commences with construction of the upstream cofferdam to 408 mRL and diversion of the river through the tunnel in early October to take advantage of dry season river flows. River bed foundation excavation commences together with construction of the toe slab in the river bed. The steel reinforced rockfill in the downstream stage is completed to 408 mRL, the height of the upstream cofferdam. At this time, the downstream stage controls the river (e.g. if the upstream coffer dam is overtopped river water would still be retained behind the downstream stage).

Phase 2B requires completion of the downstream stage to 429 mRL and completion of toe slab works in the gorge followed by placement of upstream rockfill to 429 mRL. With completion to 429 mRL, the tunnel can divert the 1 in 1000 AEP flood with an overflow depth of less than 0.9 m of water passing over the reinforced rockfill.

As the steel reinforcement is terminated at 429 mRL, a rapid two stage construction of a minimum width berm is used between 429 mRL and 439 mRL.

Phase 2C constructs a minimum width 10 m high rockfill berm across to the left hand abutment leaving a 40 m “flood gap” at 429 mRL. If an extreme flood occurs during this period it is passed through the flood gap and over the steel reinforced rockfill. When constricted to a 40 m width the tunnel can divert the 1 in 1000 AEP flood with an overflow depth of around 2.1 m passing over the reinforced rockfill.

Phase 2D closes the “flood gap” in a matter of days to complete the berm and then the remainder of the upstream fill is completed to 439 mRL. During that period the risk is confined to loss of the flood gap plug material and construction equipment. The rockfill dam can handle the 1 in 1000 AEP flood with zero overtopping once the embankment reaches 439 mRL.

Phase 2E fills the dam upstream of the closed flood gap to 439 m RL.

Phase 3 consists of rockfill placement to the underside of the parapet wall and completion of the two spillways. Spillways need to be operational by the time fill placement reaches the base of the parapet wall. This is followed by concrete face slab construction.

The final Phase 4 work consists of closure of the diversion tunnel which enables construction of the outlet works and hydro power station and commissioning of the dam. The parapet wall and road works are completed in this phase. Diversion is now by the spillway rather than tunnel, but at a reduced capacity than final spillway arrangements will provide pending completion of the parapet wall. The following measures can be adopted during parapet construction (approximately 3 months):

- Delay closure of the tunnel for a few months
- Defer putting in the ogee crest.

The upstream cofferdam controls the river for less than 3 months while initial river bed construction works are carried out. Although the crest level at 408 mRL is lower than the flood rise expected in the mean annual flood, a flood of this size has a much lower probability of occurring during the dry period identified for river bed construction. In some cases, different sets of return period floods are developed for each season. As the tunnel and upstream weir are currently sized, the tunnel can pass in the order of 80 m^3/s. This flow rate is exceeded 9 times in the 43 year flow record examined above during the dry period (October to January inclusive). This suggests that the upstream coffer dam could be overtopped on average once in every five years (neglecting routing effects, and noting that the synthetic record predicts a greater number of floods than the actual record). As mentioned above, a recurrence period of 2 to 5 years is considered reasonable during initial site works, where there is no risk to population downstream, expected damages from overtopping failures are low and the period of exposure is short.

Once the downstream stage with reinforced rockfill rises above the cofferdam level, it controls the river and the overall safety of the works. The upstream cofferdam continues to provide dry access to works in the river bed such as toe slab construction grouting and upstream rock placement. Failure of this upstream cofferdam may occur in the wetter months but this is a construction issue and does not affect the safety of downstream interests. This is because the downstream stage will be higher than the coffer dam level during the wetter months, and damage would be limited to flooding of the area between the upstream coffer dam and the downstream stage and to any flow damage to the upstream coffer dam itself.

4.5.3 Reservoir filling

Hold points during reservoir filling are not normally required for CFRD constructions as the upstream concrete face and the free draining rockfill ensure there is no build-up of pore pressure within the embankment. International practice is to place no restraint on filling rates and there are numerous examples of fast filling storages:

- 140 m high Alto Anchicaya Dam, Columbia; 1 week
- 202 m high Campos Novus Dam, Brazil; 1 week to 90% height
- 110 m high Cethan Dam, Tasmania, Australia; 10 weeks
- 85 m high Cogswell Dam, USA; 71 hours
- 148 m high El-Infiernillo Dam, Mexico; 17 weeks.
- 125 m high Ita Dam, Brazil; 10.4 weeks.
- Foz Do Areia Dam, Brazil; 21 weeks

- 94 m high Murchison Dam, Tasmania, Australia; 2.6 weeks
- 122 m high Reece Dam, Tasmania, Australia; 7 weeks

Where there have been problems on first filling, such as extensive face cracking at Campos Novus, on re-filling there have been no requirements for immediate lowering of the storage. Leakage of 1500 l/s or more has been safely passed through high rockfill dams with no concern for safety of the dam or public safety.

Dams regularly fill to some extent during construction prior to construction of the concrete face. The dam can be filled entirely provided the downstream toe rockfill is protected with steel mesh, as is proposed in this case. The 38 m high Lostock Dam in NSW Australia filled and the spillway operated prior to construction of the concrete face without damage or concern for dam safety.

No hold points or an additional low level outlet are therefore proposed for dam filling.

4.6 Long term reservoir erosion

Walkover investigation has been undertaken to assess the potential for long term erosion of the reservoir margins. This assessment has not comprehensively covered the entire reservoir perimeter, but has concentrated on key areas accessible and visible to the public.

The steepest slopes within the upper reaches of the reservoir are formed by weak siltstone overlain by gravels. The siltstone outcrops (cliffs) that adjoin the reservoir will fret or slab over time as result of wetting and drying with the changing water levels within the reservoir. This is a natural process that is already evident around the dam site. Figure 4.3 shows existing erosional processes (slabbing of the siltstone on stress relief defects) around the proposed reservoir. It is expected that these processes will continue over the life of the dam (and reservoir). The rate of retreat of the steeper siltstone faces is judged to be in the order of that seen in the river now lying somewhere between 1 to 5 m per 50 years. At the present time the river is actively removing debris from the toe of these slopes, and this erosion this is likely to diminish following the dam construction resulting in an easing of the overall slope gradient.

Terraced gravel layers between approximately 2 to 20 m thick typically overlie the siltstone on a sub-horizontal surface cut onto the siltstone around parts of the proposed reservoir rim. These have been mapped recording the interface position and level of the gravel and siltstone layers where accessible around the reservoir. The gravel beds are currently standing at near vertical slopes mirroring the erosion profile of the underlying siltstone. Where these gravels will be subjected to repeated submergence by the fluctuating reservoir water levels it is likely that that wave action and cyclical wetting and drying due to drawdown and refill of the reservoir will result in a flattening of the overall slope angle that will be locally controlled by thickness of the gravels and their material properties such as grain size and cohesion. The thickest bed of gravel observed that will be submerged by the reservoir is approximately 20m.

The horizontal extent of this erosion is influenced by the following factors:

- Gravel thickness
- Grading of the gravel which will control the gravels ability to self-armour
- Gravel exposure location and direction (in relation to wind and fetch)
- Wind direction and fetch
- Reservoir depth, range and frequency of drawdown

The long term regression due to wave erosion is expected to develop slopes lying between 1V:2H and 1V: 5H to above top water level based on research undertaken at Lake Pukaki (Mathewson 2011).

Figure 4.4 shows the beach profile developed on the western shore of Lake Pukaki. A similar process here would result in a regrading (flattening) of the exposed gravel profile (independent of the siltstone erosion) of approximately between 4 m and up to 100 m (the latter where the full 20 m thickness of the gravels is submerged). Appendix D contains a figure (Figure 201) showing the predicted approximate extent of long term erosion of the gravels.

Some of the siltstone cliffs and gravel layers likely to erode will be fully submerged by the reservoir at FSL, and therefore will only be visible when the reservoir is drawn down.

The reservoir rim will be monitored adjacent to existing or proposed infrastructure (e.g. roads, power poles, the dam and appurtenant structures) that may be affected by the erosion. Should regression of the rim progress towards infrastructure, then mitigation measures can be actioned (e.g. stabilisation works, realignment of roads or power lines).

A preliminary assessment has been made of the potential total sediment volume and rate of erosion resulting from shore edge erosion around the formed lake. This information may assist in the assessment of effects on reservoir water quality (e.g. turbidity) of this additional source of sediment. In this regard, this information should be considered in conjunction with the Sedimentation Assessment (Tonkin &Taylor, May 2013), which is based on the river borne sediment.

The ultimate volume of material that may be eroded from regression of the lake shore margins in the long term is estimated to be roughly around 2 million m^3. However, the rate of erosion over time generating this ultimate volume of sediment is less certain.

While there are many factors that drive erosion and shoreline recession processes, wave climate has been found to be a good indicator of erosion susceptibility (USGS, 2004) and the greatest potential for shoreline erosion exists where there is a large wind fetch and steeper offshore slopes (James, 2012). Typically the establishment of new equilibrium shorelines take a long time to develop i.e. 100 years or more (James, 2012). The resulting foreshore of non-cohesive sediments tend to have a steep foreshore (around 7(H):1(V)) between the Mean Water Line (MWL) and the upper swash extent, a slightly flatter slope (around 9(H):1(V)) from the MWL to the break line typically situated below the typical low water level and around the angle of repose in deeper water.

Shoreline erosion rates of around 1 m/yr have been measured along the western shores of Lake Pukaki in areas of very erodible alluvial material and fetches of between 6 and 13 km. An average rate of 0.52 m/yr over a period of about 30 years has been recorded at the southern end of Lake Hawea in glacial till deposits around 8 m high (James, et al. 2002). Lake Hawea is some 4 km wide and has a north south orientation with a fetch of around 18 km.

For the Ruataniwha Water Storage Scheme, the more erodible sediments around the formed lake comprise extents of unweathered gravels with a D50 of around 10 to 25 mm, and around 15% of the gradings being less than 0.1 mm (Tonkin & Taylor, March 2012), overlain by thick volcanic ash and loess (up to 1.5 m thick). The main areas of gravel exposures in the vicinity of the mean annual flood level are shown in Figure 201 in Appendix D.

Erosion rates for the non-cohesive sediments have been estimated based on published rates of long term erosion at the southern end of Lake Hawea as it was recorded in similar

material to the material observed around the Ruataniwha reservoir. The erosion rates along the western edge of Lake Pukaki were not considered appropriate due to the large extents of alluvial sediments that form the western slopes of that lake.

Adjustment of the 0.52 m/yr rate for Lake Hawea was undertaken to take into account differences in wave climates based on a comparative ratio of fetch length; with the relationship being approximately a function of the square root of the fetch (CIRIA/CUR, 1991). The longest fetch in the formed reservoir for the Scheme is around 3 km and the ratio between the square root of the fetches of 3 km and 18 km is around 0.4. The shortest effective fetch is around 600 m and the ratio in this case is around 0.18. Assuming a direct relationship between fetch and erosion, the expected maximum rate of erosion at Ruataniwha is around 0.2 m/yr. However the actual erosion rate could be less than 0.1 m/yr in more sheltered areas.

Assuming an erosion rate in the range of 0.1 to 0.2 m/yr, the expected erosion volume in those more erodible areas is between 8000 and 18,000 m^3/yr, although due to differences in exposure and orientation, erosion is unlikely to occur uniformly (spatially and in time) in these areas. Based on the grading information set out above, the majority of this sediment would rapidly settle in close proximity to the area eroded. It is estimated that less than 15% of this volume would remain in suspension for periods longer than several minutes.

By comparison, the rate of sediment infill in the reservoir from sediment entrained in the Makaroro River flow is easily an order of magnitude greater at 150,000 to 260,000 m^3/yr, comprising around 80,000 m^3/yr of suspended sediment along with between 80,000 and 180,000 m^3/yr of bed material sediment.

Figure 4.3 Erosion of siltstone around the dam site. The top photo shows a typical gravel layer overlying siltstone

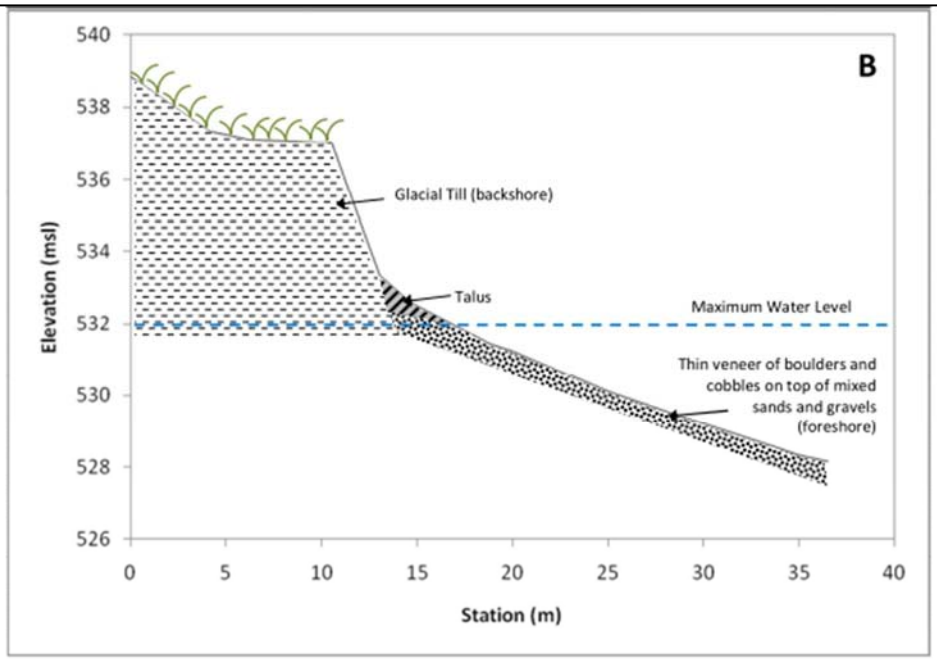


Figure 2.12: A) Site 11 on the western shore, looking north. Southern end of rock revetment near Boundary Stream. Typical 'pavement beach' subject to the high occurrence of longshore currents. Photo taken on 6/11/10. B) Longsection profile of Site 11A, two times vertical exaggeration. Talus accumulation on the lake side of the cliffed backshore acts as protection for any potential incoming wave energy directed at the base of the cliff.

Figure 4.4 Beach profile developed by wave erosion at Lake Pukaki (Mathewson, 2011)

4.7 Construction management

A Construction Environmental Management Plan (CEMP) and a number of Supplementary Construction Environmental Management Plans (SCEMPS) will be prepared and certified prior to works commencing onsite. This section describes the proposed borrows, spoil disposal, construction traffic and sediment management, arrangements for which will be finalised in the CEMP and SCEMPs. A draft CEMP is included with the proposed conditions of consent and provides additional detail to that summarised in this report, along with the scope and range of the SCEMPs. The draft CEMP demonstrates that adequate environmental mitigation measures will be implemented for this project.

4.7.1 Preliminary construction programme

A preliminary construction program is shown in Appendix B. This programme assumes an arbitrary project commencement date. The programme adopts the general sequencing of work outlined in Section 4.5.

The programme is based on getting 240 productive days per annum, the latter being derived from an assumed 5 productive days per week for 48 weeks per annum (this allows for 2 weeks of statutory holidays and two weeks of project shut down for worker annual leave).

Some activities such as the concrete face construction, once started, will continue 7 days per week, 24 hours per day, subject to compliance with resource consent conditions (for example as to construction noise) and the CEMP.

The total construction time from contract award to project completion is currently estimated at 54 months. Depending on the contractual model adopted for the project,

investigation, design and procurement will need to be assessed in relation to the overall project timeline.

The start date for construction could be delayed a subsequent twelve months (or multiple of) to allow for consenting, land purchase, detailed design, procurement etc. This programme simply highlights a sensible time during any given year to start construction on site to take advantage of the seasonality of river flows.

4.7.2 Borrow and fill disposal areas

Gravel and rockfill borrow will be provided primarily from required sub-excavation, primary and auxiliary spillway and diversion channel excavations.

The exception is that Zone 1A material, a low permeability "crack filling" layer overlaying the perimetric joint, is to be provided from a siltstone borrow located 1.5 km upstream as shown on Drawing 27690-DA-104. Also, Zone 2B material, which must conform to a specific grading envelope, is to be sourced from a Q2 gravel borrow near the head of the reservoir. In addition, concrete aggregate is to be provided from a river gravel borrow upstream of the diversion intake as shown on Drawing 27690-DA-104. The river gravels derive primarily from Rakaia terrain greywackes that are in the headwaters of the Makaroro River rather than the Waioeka petrofacies expected in the proposed excavations at site.

With the arrangements shown on the drawings, the cut to fill volume of rockfill is essentially balanced. There is some flexibility to adjust for changes in volume of rockfill required by flattening or steepening excavation cut batters, though steeper batters may require bolting and meshing to provide safe working environments downslope during construction.

The excavations required to establish the diversion channels and tunnel during Phase 1 produce more gravel and rockfill than can be placed on the right bank terrace (with sufficient offset from the gorge). Since Phase 1 (construct diversion) and part of Phase 2a (construct upstream coffer dam) must be complete before in stream dam construction can begin, there will need to be some temporary stockpiling (in the order of 112,000 m^3) of rockfill and gravel.

Excavations are also expected to produce significant excess volumes of gravel, ash and colluvium that is not needed in embankment construction. Based on current material assumptions, disposal volumes of gravel are expected to be in the order of 600,000 m^3 and colluvium and ash in the order of 200,000 m^3. Several potential locations for spoil disposal are shown in Drawings 27690-DA-100 to 105. These locations will be finalised in the CEMP and relevant SCEMP.

Surface water drainage and subsurface drainage in the base of the valley beneath the proposed fill will be required for the spoil disposal areas located in gullies. Robust drainage detailing will also be necessary for the spoil disposal area shown in the river downstream of the dam in order to ensure water drains freely from the downstream shoulder of the main dam.

The design of under drainage and whether shear keys are required will be considered during the detailed design stage.

It is expected that rip rap will be required in specific locations such as at culvert entrances and exits and at the toe of the dam. No rip rap of suitable size is expected to be found close to the dam site. The Application Design therefore assumes that rip rap will be carted

to site from existing quarries. An estimate of quantities required of rip rap is summarised in Section 4.7.3.1.

4.7.3 Construction traffic and haul roads

The construction road arrangements presented represent one possible option among a multitude of alternative arrangements. Typically, a constructor will spend a significant period of time optimising arrangements, considering for instance efficiency of material movements at various stages, how construction roads might need to change with the stage of construction, the plant (and its associated constraints) available to the particular contractor, and safety considerations. It is important not to constrain the contractor to a fixed construction road arrangement even at the design stage in order to allow flexibility to cope with any unexpected site conditions that emerge during the construction phase. Flexibility in construction road arrangements also allows for potential contractor innovations and optimisations that could lead to cost savings, safety improvements, better stockpiling and sediment control practices etc.

Access and haul road arrangements will also clearly depend on borrow locations. Although not expected, borrow sources and consequently potential construction road arrangements could change following further investigations and input from contractors.

The detailed CEMP will confirm borrow and construction road arrangements.

The options for access and haul roads shown on the Application Design drawings are summarised below:

- Access to site from public roads is off Wakarara Road just south of Wakarara (refer Drawing 27690-DA-104)
- New access track from the right bank terrace to river bed near upstream coffer dam (refer Drawing 27690-DA-100)
- New access track from right bank terrace to river bed level near landslide stabilisation works and diversion intake (refer Drawing 27690-DA-100). This track will also provide access to the river gravel borrow. An existing access track to the top of the landslide stabilisation works may also be used
- Upgrade existing farm track to access the siltstone borrow (for Zone 1a material) (refer Drawing 27690-DA-104)
- Haulage of Zone 2B material (approximately 100,000 cu.m) from the head of the reservoir to the dam site will be along low terraces adjacent the river bed or in the river bed itself where the topography is constraining e.g. not involving the public road network
- In addition, access along the river bed in the vicinity of the dam construction works (e.g. in the zone between the river gravel borrow and the spillway outlet channel) is likely to be required.

Construction access affecting public roads is discussed further in Sections 4.7.3.1 to 4.7.3.3.

4.7.3.1 SH50 to dam site along Wakarara Road

Construction vehicles along this section of road could potentially include:

- Road truck and trailers supplying sand for Zone 2A (2,000 cu.m) and sand for concrete (9,000 cu.m), fly ash, and rip rap armour (approximately 500 to 1,000 cu.m)
- Cement delivery truck supplying cement (8,500 – 10,500 t)
- Road trucks supplying:

- reinforcing steel (2,000 – 3,000 t) (note steel density in concrete face 55 kg/cu.m approximately, and no steel in kerb detail)
- rock bolts:
  - 783 no. 25 mm dia 3 m long passive bars
  - 278 no. 28 mm dia 6 m long passive bars
  - 934 no. 20 mm dia 2.4 m long passive bars
  - 56 no. 28 mm dia 6 m long passive bars
  - 6 no. 26 mm dia 8 m long tensioned bolts
  - 15 no. 24 mm dia 10 m long tensioned bolts
  - 1,500 no. 24 mm dia 3 m long passive bars
- structural steel (miscellaneous steelwork, beams, grid flooring, staircase in intake tower and power station)
- scaffolding
- pipework (170 m of 2.1 m ID 8 mm thick, 280 m of 0.6 m ID 6 mm thick cement lined pipe, and 43 m of 2.1 m ID 16 mm thick pipe)
- precast concrete (culverts, and parapet wall (1,440 cu.m) if done with precast units for programming purposes)

- Articulated trucks supplying:
  - precast concrete (bridge beams etc. for 20 m by 5 m bridge over spillway)
  - structural steel
    - 20 no. steel sets (200UC) 4 m diameter for tunnel,
    - trash racks (6 no. x 4t each 2.1m square, 4 no. x 2 t each 2.4 m by 3 m)
    - baulks / stop logs (2 no. x 2 t each 2.4 m by 3 m, 2 no. x 5 t each 2.4 m by 3 m)
    - 15 t portal frame for 6t OHT crane in intake tower
    - roofing for power station (113 sq.m)and two valve boxes (13 sq.m and 75 sq.m)
    - lifting beams for baulks and trash racks in intake tower
- Heavy low loaders supplying (once off deliveries only):
  - valves (1.6 m dia 5 t butterfly valve, 1.2 m 9 t FCD/needle valve, 0.6 m 2 t FCD/needle valve)
  - gates (2.1 m square 8 t slide gate)
  - generator and turbines (5.7 MW and 0.85 MW Francis type) (35 t max indivisible lift)
- Construction vehicles that will mostly remain within site once delivered
- Tractors
- Low loaders delivering:
  - hydraulic excavators (12 t, 20 t, 30 t and 100 t)
  - graders
  - scrapers
  - 6 wheel all terrain dump trucks (Figure 4-5)
  - large earthmoving dump trucks
  - crushers (Figure 4-6)

  - bulldozers (e.g. 320 HP)
  - vibratory compactors (e.g. 6 t, 12 t and 18 t)
  - light wheel loaders (e.g. CAT 966 or similar)
  - hydraulic crawler loaders (> 3 m3 bucket capacity)
  - tower crane (10 t capacity 50 m radius)
  - drill rigs, and;
  - specialised trolleys and kerb extrusion equipment for concrete face construction
- Concrete agitator trucks
- Mobile cranes.

It is expected that there will be an average of approximately 70 workers at the dam site over the construction programme.

Figure 4-5 Articulated dump truck

Figure 4.6 A portable rock crushing plant

4.7.3.2 Construction vehicles for establishing forestry, public and private access

Construction vehicles will be required to establish the new access tracks and roads described in Section 4.8. These replacement access routes will likely be constructed to a similar standard as the existing access, and therefore construction vehicles are unlikely to be constrained by geometry or limited capacity fords and bridges. The exception is that road trucks will likely be required to deliver precast concrete culvert sections for two new fords, and whereby culvert sections could probably be walked in by excavator.

4.7.3.3 Construction access to reservoir rim

Access to the reservoir rim along public roads may be required in order to construct fencing. This is most likely to involve road trucks and tractors to deliver fencing material.

4.7.4 Erosion and sediment control measures

The dam construction including diversion works and spillways comprise earthworks in and out of the Makaroro River and its smaller tributaries. Large volumes of rock and alluvial materials totalling in excess of approximately 2.5 million cubic metres will be cut to spoil disposals and cut and placed to construct the dam.

There are two main physical areas that will require different management practices during construction:

- Within the river or its tributaries, and
- On the river terraces.

Specific activities that will occur within the river or its tributaries are:

- Construction of coffer dams and diversion of the river through a tunnel
- Construction of fords and culverts along new forestry, farm and public roads and tracks
- Construction of the main dam embankment
- Grouting of the foundation under the main dam embankment
- Spillway outlet works
- Stripping river gravels from the reach that will be submerged by the reservoir for use as filters and concrete aggregate
- Temporary haul route to Zone 2B.

Specific activities that will occur on the river terraces are:

- Construction of the main dam embankment
- Grouting of the foundation under the main dam embankment
- Construction of spillways
- Construction of the intake tower
- Construction of temporary haul roads
- Construction of permanent (including replacement) farm, forest and public access roads
- Formation of rock quarries (part of the excavations for river diversion and spillway).

Specific measures to mitigate the effects of the above works in respect of erosion and sediment control will be detailed in the CEMP. This section of the report outlines the possible methods by which these measures will be developed and implemented. The control measures adopted will be in accordance with Hawke's Bay Regional Council Waterways Guidelines - Erosion and Sediment Control.

Out of river works

Specific measures such as mulching, super silt fences, hay bales and sediment retention ponds (Figure 4.7) will be implemented in accordance with the CEMP to prevent silt from entering the Makaroro River below the dam site and the other streams adjacent the works. The exact positions of these measures will change as the construction progresses.

- Stabilised construction entrances will be used at the site access locations to prevent silt from leaving the site.
- Roads will incorporate vee drains and possibly rock check dams to reduce erosion and trap sediment.
- It is envisaged that water collecting in the bottom of either large excavations, or during tunnelling will need to be decanted to sediment retention ponds before discharge back into the river(s).

Post construction reinstatement of areas will need to be undertaken.

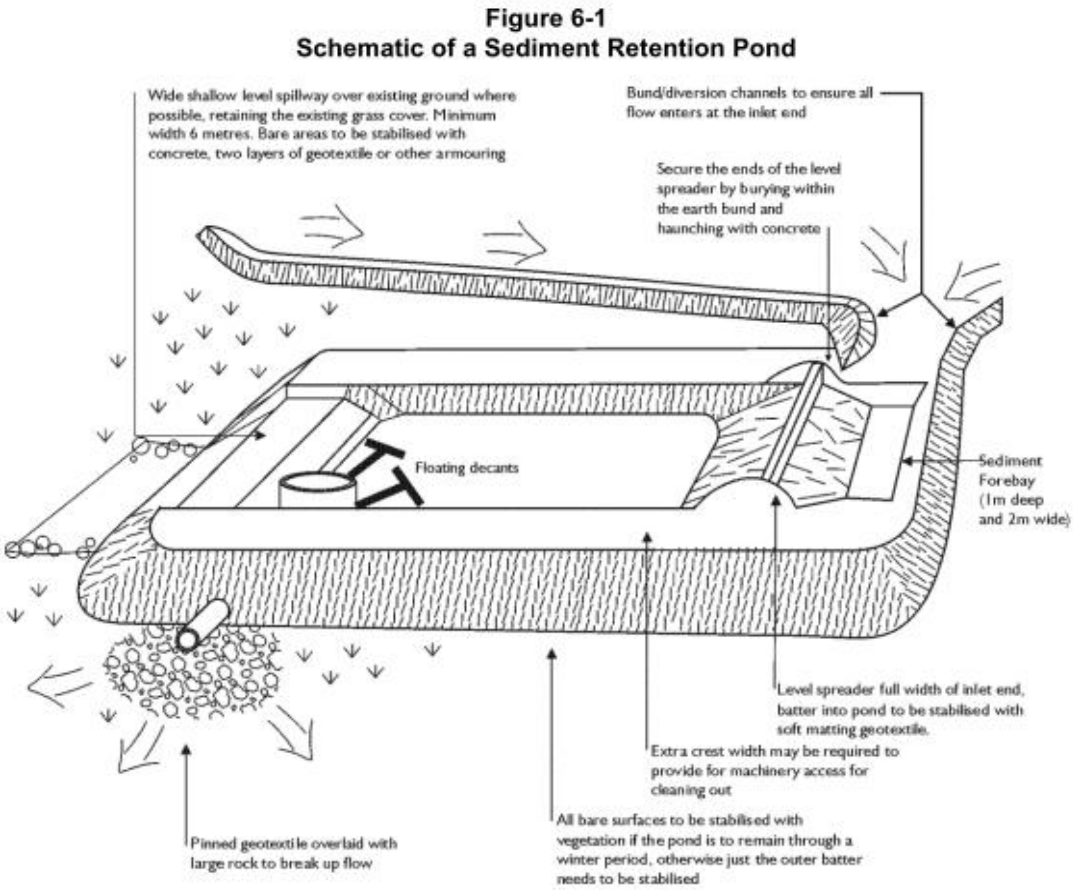


Figure 4.7 Typical Sediment Control Pond (Hawke's Bay Regional Council Waterway Guidelines Erosion and Sediment Control April 2009)

In river works

Dam building by definition involves in-river construction works. It is generally desirable from a constructability point of view to minimise the amount of work in the river whilst the river is flowing. To achieve this, a temporary coffer dam will be constructed upstream of the main dam embankment and the river diverted through a cutting and diversion tunnel through the terrace on the right bank. Once built, this will enable the main dam to be constructed in the river without mobilising sediment.

Extraction of river gravels will be utilised for concrete production. Extraction of river gravels is common in the Hawke's Bay. As such, the contractor will be required to comply with documents such as the HBRC's Environmental Code of Practice for River Control and Drainage Works (June 2003). It is expected that methods such as having designated river crossing locations and utilising out of river shoals are used to minimise disturbance of the river.

4.7.5 Dust control

The foot print of works is several hectares in extent and will involve extensive stripping and carting of alluvial materials. Quarrying of rock to form the embankment and spillway cuts will likely involve blasting. All of these activities have the potential to produce large quantities of dust.

The time when dust is of most issue is when:

- It is windy
- Dust settles on heavily trafficked areas
- Prior to surfaces being stabilised.

The primary mitigation measures for the works are as outlined in the draft CEMP. They include:

- Use of water carts
- Stabilise surfaces (for example with the use of hardfill)
- Sealing (in the case of roads)
- Grassing (for fill disposal areas or borrow areas).

Grassing would only be suitable for areas that are not within the reservoir.

4.7.6 Noise effects

Construction of a dam is an inherently noisy activity. Large plant and numerous handheld tools will be used over a large extent of the site to construct the works. Concentrated activities will change in location throughout the construction programme. Therefore, the noise will not necessarily be limited to any particular location or for any given period of time. The effects of all key elements of construction noise have been assessed by Marshall Day Acoustics at the closest dwellings and daytime construction noise limits can be met in all cases. Night time construction noise limits may be exceeded at one identified dwelling for dam and intake construction, but only during aggregate extraction from one identified borrow area (2B). Road construction may need to be confined to daytime hours to achieve compliance.

There will be blasting for quarrying and tunnelling because the foundation rock is relatively hard. This activity whilst noisy will likely be for relatively short periods at any given time and will be responsibly managed in accordance with the CEMP.

Refer to the Marshall Day Acoustics (May 2013) report for a more detailed assessment on the noise effects of the project.

4.7.7 Construction vehicles and plant

Vehicles will be parked overnight in designated areas away from the river. Fuel storage and refuelling areas will be bunded to prevent any spills from entering waterways.

4.7.8 Concrete batching

Approximately 32,500 cubic metres of concrete plus grout volume of 5,000 cubic metres will be required for the works. It is envisaged that the concrete will be batched onsite using the river gravels as aggregate. This is likely to be located downstream of the dam on the right river terrace as shown on Drawing 27690-DA-104. In the unlikely event that the river aggregates are unsuitable for use as concrete aggregate then crushed rock would be used. Cement, sand, reinforcing steel and concrete additives will be imported to site.

4.8 Replacement of existing forestry, farm and public access

The extent of the reservoir at Full Supply Level will sever existing forestry, farm and public access at several locations. Based on topomaps sourced from Land Information New

Zealand and aerial photography, the following access will be severed (refer Drawings 27690-DA-104 and 105):

- Access along Wakarara Road where it crosses Makaroro River near the historic mill site and Makororo base camp. Currently there is no bridge or other structure, and the crossing for vehicles involves driving across the river bed at low flows. This access would still be available when the reservoir is at low levels.
- Access along Wakarara Road where it crosses Dutch Creek (small ford).
- Private farm access along the terrace at the eastern edge of the Makororo River and along the northern edge of the tributary to the Makororo River north of the proposed dam site.

Replacement access arrangements for feasibility were initially considered to a preliminary level prior to input from access users and landowners. Following discussions with user groups and landowners the plans have been amended to reflect their specific requirements and opportunities. The indicative arrangements as proposed following feedback are presented in Drawings 27690-DA-104 and 105.

The current arrangements are described as follows:

- Recreational access from Wakarara Road across the Makaroro River into the Ruahine Forest Park

  Maintaining access along Wakarara Road at Makaroro River close to the existing crossing would be likely to involve a ford/bridge in the order of 250 m long and 0.7 km new track (along the southern edge of the reservoir to reach a narrow point). Because of the cost of such a long bridge / ford, it is more sensible to provide a new access track so that recreational users can reach the Ruahine Forest Park around the head of the reservoir. At present, this is intended to comprise vehicle access supplemented by walking tracks. The proposed public vehicle access road follows an existing farm access track. A car park will be provided at the end of the new road.

- Forestry access along Wakarara Road across Dutch Creek

  Similar to the recreational access, maintaining access along Wakarara Road at Dutch Creek close to the existing crossing would be likely to involve a very long ford/bridge in the order of 180 m long and 0.7 km new track at Dutch Creek (along the edge of the reservoir to reach a narrow point). Because of the cost of such a long bridge / ford, the option shown on drawings is proposed, which comprises 2 km of new access track from the end of Moore Road (which comes off Wakarara Road) and a 35 m long ford over Dutch Creek.

- Farm access

  The indicative arrangement for replacement farm access follows roughly the same alignment as existing tracks but translated upslope to a higher elevation above reservoir level.

  Constructing replacement tracks is expected to be challenging and costly because of the steepness of the terrain. The farm access track is likely to be located in hard greywacke that will require some blasting.

At present, a relatively steep maximum longitudinal gradient of 1V:5H for farm access and 1V:10H for public and forestry access has been adopted to minimise cut volumes. Based on LiDAR, the maximum gradient of Wakarara Road at Dutch Creek is 1V:10H. Similarly, minimum carriageway widths of 3 m for farm access track and 4 m for forestry and public access has been assumed. The assumed cut batters (1V:0.5H in greywacke and 50° in gravel

and tertiary material) and fill batters (1V:2H) are preliminary and subject to geotechnical review during detailed design. The design allows for a vee drain down one side of access tracks, 150 mm thick GAP65 on farm tracks and 150 mm thick M4 and 250mm thick GAP65 on forestry and public access tracks.

4.9 Emergency Action Plan and Operation, Maintenance and Surveillance Manual

It is standard industry practice for an Emergency Action Plan (EAP) and an Operation, Maintenance and Surveillance Manual (O M & S) to be compiled for a HIGH PIC dam. In this case, the O M & S is intended to form part of the broader Infrastructure Maintenance Management Plan (IMMP) provided for in the proposed conditions.

The EAP and IMMP document the roles and responsibilities of the dam (and overall scheme) owner and operator in design, operation and maintenance of the dam. They also provide information on how to effectively manage emergency situations.

These documents will be developed during the design, construction and commissioning stages of the project. They will be periodically updated to reflect current legislation and ongoing roles in the scheme.

5 Water Distribution Network

5.1 Overview

The Application Design water distribution network is comprised of a primary distribution system involving a hybrid headrace canal and pipeline, along with a secondary distribution system that is fully piped.

Flow released from the proposed Makaroro Dam enters the water distribution network at the upstream water intake. A second downstream intake is proposed on the Waipawa River near Walker Rd (immediately upstream of the confluence of the Waipawa and Tukituki Rivers). The purpose of the water distribution network is to convey water from the intakes, primarily for use within the production land use areas.

Within Zones A-D, the water distribution network comprises:

- The upstream water intake (on the true right bank of the river near Caldwell Rd)
- A headrace canal with inverted siphons under significant waterways and roads, and bridges on other roads, and to enable farm access across the canal
- Large diameter primary distribution system pipelines
- Smaller diameter secondary distribution system pipelines
- A water outfall structure adjacent to the Mangaonuku Stream; and
- A water outfall structure adjacent to the Kahahakuri Stream.

For Zone M, the water distribution network comprises

- The downstream water intake on the true left bank of the Waipawa River near Walker Rd (upstream of the confluence of the Waipawa and Tukituki Rivers)
- Utilising the Papanui Stream (Old Waipawa River) as a primary conveyance system

The primary distribution system headrace canal alignment generally follows the contour to preserve head to the extent permitted by topography and other site constraints (i.e. buildings, property boundaries, roads and services).

The secondary distribution system conveys water from the primary system, via a series of pipelines and, where required, pump stations, to the farm gate throughout the production land use areas. Given that specific demand locations have yet to be determined, the secondary system has been sized to supply water to within 2 km of most areas within Zones A to D. The final arrangements will be determined by the number and location of users of the Scheme.

The secondary distribution system pipelines have been aligned to road corridors and property boundaries as much as practically possible. Insofar as other constraints associated with alignment options allows, secondary distribution system pipelines are typically perpendicular to the contour to generate head that may be utilised by irrigation systems.

The water distribution network does not include the on-farm components (tertiary system) necessary to convey water to farm hydrants from the point that an individual user connects to the scheme. Nor does the Scheme include any existing infrastructure used by current irrigators within the production land use areas.

The works have been developed to provide at least some positive pressure at all points along secondary distribution system pipelines. Consequently, the pressure available at the farm gate will vary (both during the season and between different users). In many cases,

particularly during peak times, on farm works will require pumping stations to boost pressure to enable irrigation equipment to function correctly.

The specific location of individual users and their point of connection to the scheme within the zones are not yet clear. Consequently, revision of aspects of the water distribution network, particularly secondary distribution system components, may be required when the specific locations and demand flow rates of individual users are defined. Further optimisation and refined construction cost estimates for the developed alignment can then be undertaken.

Drawings 27690-DN-000 to 27690-DN-701 illustrate the key components of the Application Design water distribution network (Appendix A). The drawings are outlined as follows:

- Drawing 27690-DN-000: Drawing list
- Drawings 27690-DN-001 to 004: Irrigation supply zones and the overall general arrangement of the distribution works
- Drawings 27690-DN-100 to 110: Waipawa River intakes concept including sediment management and flow control
- Drawings 27690-DN-232 to 240: primary system headrace canal including siphons under significant watercourses and roads
- Drawings 27690-DN-300 to 301: Drop structures for primary system canal
- Drawings 27690-DN-400 to 402: Siphon details
- Drawings 27690-DN-500 to 502: Typical stormwater crossing and bridge details
- Drawing 27690-DN-600: Secondary system off-take details.
- Drawings 27690-DN-700 to 701: Typical stream discharge outlet details

All reduced levels referred to within this section of the report are in terms of Hawke's Bay datum.

5.2 Key components

The Application Design comprises:

- The upstream water intake on the Waipawa River to collect flow released from the proposed dam
- The downstream water intake on the Waipawa River to divert run-of-river flow in order to provide a residual flow down the Old Waipawa River/Papanui Stream bed and to collect flow released from the Mangaonuku outfall for subsequent distribution within Zone M
- Approximately 0.3 km of 11.1 m^3/s capacity primary system headrace canal in Zone B (to Zone A bifurcation)
- Approximately 14.9 km of 7.8 m^3/s capacity primary system headrace canal in Zone B
- Approximately 2.9 km of 4.1 m^3/s capacity primary system headrace canal in Zone C
- Approximately 18 km primary system pipeline within Zones A, C and D
- Approximately 121 km of secondary system distribution pipe line to service the production land use areas as shown on Drawing 27690-DN-001
- A water outfall structure adjacent to the Mangaonuku Stream;
- A water outfall structure adjacent to the Kahahakuri Stream
- Utilising the Papanui Stream (Old Waipawa River) as a primary conveyance system

- Various miscellaneous works including, but not limited to, inverted siphons, control gates, road bridges, farm bridges, emergency spillways, bifurcation works/pipeline inlet structures, access roading and storm water crossings.

Maximising the gravity supplied area of the scheme requires a higher alignment for the primary distribution system works. The primary distribution system headrace canal alignment is constrained by the geography of the area (e.g. the foothills of the Ruahine Range) and the elevation of the intake site (at around 260 m RL which acts as a control for the maximum elevation of the canal). A higher alignment has increased length and/or earthworks for the primary distribution system headrace canal due to the topography of the foothills.

5.3 Water distribution network concept – Zones **A** to **D**

The Application Design criteria and assumptions used to develop the water distribution network in relation to production land use Zones A to D are summarised below in Table 5.1.

In terms of Optimisation, there is opportunity to review and refine the parameters and specifications summarised in Table 5.1 during the early stages of detailed design. By way of example, there may be refinement of the relative length of primary distribution system headrace canal versus pipeline to reduce the length of the former and increase the length of pipeline, as well as route alignment, following further consultation with landowners.

Table 5.1 Ruataniwha Water Storage Scheme – water distribution network design parameters/specifications: Zones **A** to **D**

Parameter	Adopted specification
Upstream Water Intake Structure	
General arrangement	Intake to exploit outside bend adjacent to promontory feature and be sited against extremely weak pumacious sandstone sub vertical face. Stopbank works to protect intake and primary system canal from floods up to the design event (100 year ARI). Requirement to exclude coarse sediment and significant quantities of fine sediment from the primary system. Fish screen and intake bypass or equivalent to exclude fish from the primary system to meet environmental performance requirements advised by Cawthron Institute. Inlet control to primary system canal by underflow gate to accommodate the peak primary system demand and Waipawa River water levels at the intake location within the operational range defined below.
Operational range	Operational range to coincide with the 7 day MALF and 5 year ARI flood respectively. Corresponding operational range between 259.4 m RL and 261.4 m RL at the northwest extent of the infiltration bund.
In river maintenance work	Due to the braided nature of the Waipawa River, in river works will be necessary from time to time throughout the operational life of the project to maintain the river channel (flow and level) at the intake location, and manage the deposition of coarse sediment in the vicinity of the intake.

Intake structure flood protection	Provide 1 m of freeboard to intake structure from nominal 100 year ARI flood level (as derived from interpretation of cross section and flood level information provided by HBRC) to recognise uncertainty in the flood level, and allowance for sediment flux consistent with advised HBRC policy for stopbank works further downstream on the Waipawa River.
Fish exclusion	The NIWA Guidelines set out the following mechanical screen design criteria for the exclusion of adult native and salmonid (i.e. trout and salmon) species (that do not relate to the proposed bund per se even though Cawthron Institute has indicated the proposed bund will provide similar performance): - A maximum permitted bar screen aperture of 2 mm, or 3 mm for mesh aperture. - Screen approach velocity not to exceed 0.12m/s. - Screen sweep velocity to exceed approach velocity.
Intake structure head loss	0.8 m from lowest operational range to settling pond outlet with 20 % screen blockage at low flow (from 259.4 m RL within the realigned river braid at the north west end to 258.6 m RL at the radial gate).
Exclusion of fine sediment	Intake arrangement to remove a high percentage of particles coarser than 0.2 mm.
Exclusion of coarse sediment	Intake design to include potential for exclusion of coarse sediment from the primary system, and to cater for accumulation of some gravel in the vicinity of the intake prior to physical removal.
Debris exclusion	Aside from settling ponds and fish exclusion no specific provision to remove other debris that may have potential to compromise efficiency of irrigation equipment (e.g. floating weed and the like that may otherwise enter or accumulate within the primary system canal).
Primary distribution system headrace canal	
Methodology	Alignment to maximise potential for supply by gravity. Alignments set to minimise the total cost of earthworks, maximise the most economic balance of cut and fill and follow favourable topography.
Primary distribution system headrace canal cross section	Conventional trapezoidal canal with dimensions to suit optimum construction and operational requirements as confirmed by detailed site and material investigations. For example, 3(H):1(V) sideslopes have been assumed for construction of an earth lined canal option, however further investigation at the detailed design stage may show that steeper sideslopes are feasible, or that the alternative high density polyethylene (HDPE) lined section with 2(H):1(V) sideslopes is better suited to the scheme requirements.
Potential Impact Category (PIC)	Assumed to be Low in accordance with the NZSOLD New Zealand Dam Safety Guidelines and the Building (Dam Safety) Regulations for feasibility study assessment purposes. This is based on precedence of similar sized irrigation canals in similar rural environments. E.g. the Rangitata Diversion Race in Canterbury and the Black Point canal in North Otago.

Canal gradient along invert	10,000(H):1(V) (0.0001 m/m) to minimise velocity given lack of specific data on potential earthen lining materials (including material ability to sustain cycles or wetting and drying) and to maximise area serviced by gravity – subject to optimisation once liner geotechnical properties quantified and alignment confirmed.
Surface roughness (Manning's n)	0.030 for earthen liner 0.016 for HDPE liner acknowledging that the membrane will invariably contain significant wrinkles due to thermal movement of the black membrane and if pursued during detailed design, subject to review at that time.
Canal side slopes	3(H):1(V) within channel utilising earthen liner. 2(H):1(V) within channel utilising HDPE liner. 2(H):1(V) cut and fill batters beyond channel (adopted without the benefit of specific geotechnical information that may justify more optimum arrangement).
Freeboard	0.3 m. Canal transients or waves not specifically considered for feasibility assessments.
Canal crest level	Varies to suit canal gradient (i.e. provision to maintain a nominal "parked" water level/ability to pass small stock water flows is included but portions of the canal liner would be subject to cycles of seasonal wetting and drying). Provision for a minimum parked water level needs consideration during detailed design should earth lining be selected or egress from an HDPE lined section should this lining option be selected.
Canal permeability/lining	Earthen liner 0.4 m thick layer of compacted red metals (moderately weathered silty gravel) imported from older high level terraces and/or 1.5 mm thick HDPE liner on a Bidim A44 non-woven geotextile blinding layer. Based on assumed material properties assume that canal liner under drainage and/or filter protection is not required but possible requirement for these works to be reviewed following completion of geotechnical investigations. Cut/fill interface not to cross canal below Full Supply Level except at transition zones (i.e. avoid insofar as is possible sidling fills).
Canal embankment fill zoning and/or internal drainage	No zoning aside from lining above. Bulkfill is expected to predominantly comprise unweathered to slightly weathered gravels. Therefore, for the purposes of Application Design, inclusion of internal zoning and specific drainage is not considered necessary if seepage gradients and fill heights are modest (it is noted that this assumption requires confirmation during detailed design).
Crest width	Earthen liner: 3.8 m wide (plan width) on true left which allows for a 3 m wide access road, and 3.2 m wide (plan width) on true right for a grassed access track. HDPE liner: 4.8 m wide (plan width) on true left which allows for a 3 m wide access road and anchor trench to liner suppliers design, and 4.2 m wide (plan width) on true right for a grassed access track and anchor trench to liner suppliers design. Both canal crests slope at 25(H):1(V) (4 %) towards the canal in cut situations, and are level in fill situations.

Crest road access	3 m wide gravelled access on one side to key structures (intake, siphons, and secondary system intakes).
Canal level control	Intake flow control is provided by the radial gate at inlet to the primary system canal. Additional level control is provided by Rubicon proprietary vertical slide gates at the bifurcation to Zone A and at the end of Zones B and C to accommodate variable draw off, including for stock water low flows.
Zone B canal drop structures	Five baffled reinforced concrete chutes are proposed for the selected alignment option. Drop structures include provision of sharp crested weirs to control upstream water level and maintain parked water level (currently assumed as half the Full Supply Level) to assist with potential earthen liner requirements and stockwater supply outside of the irrigation season.
Zone B primary system canal emergency overflow	3.7 m3/s sidespill broad crested weir to the LHS of the Tukituki River at the south end of Zone B.
Zone B primary system canal emergency overflow	3.3 m3/s sidespill broad crested weir to the RHS of the Waipawa River near the bifurcation to Zone A.
Zone C primary system canal emergency overflow	4.1 m3/s sidespill broad crested weir to the RHS of the Tukituki River at the south end of Zone B.
Canal dewatering provisions	Assumed to be by secondary system pipelines.
Primary Distribution System Headrace Canal access and crossings	
Inverted Siphons	Allowance for 8 inverted siphons comprising single barrel 1.6 to 1.8 m diameter precast reinforced concrete rubber ring jointed pipes under the Waipawa River, Kahahakuri Stream (two locations), Ongaonga Stream (two locations), State Highway 50 and the Tukituki River. The siphons have a minimum cover of 5 m under major rivers, 2 m under smaller streams to allow for river scour effects and minimum cover to accommodate vehicle loading under roads. Concrete pipe bedding is provided under water courses to provide additional scour protection. The assumed design pipe roughness is 0.012.
Road crossings – bridges	Allowance for 3 proprietary bridges at District Council road crossings along Caldwell Road, Wakarara Road and Ngaruru Road. For Application Design purposes it is assumed that three bridges have a deck width of 5.4 m and structural capacity to carry Class 1 loadings and one bridge (Wakarara Road) has a deck width of 8 m and structural capacity to carry HN/HO loading. The bridge abutments protrude into canal as required to limit span between bearings to 14 m for Double Tee sections to carry Class 1 loading. Possible opportunity to accommodate minor disruption of existing sight distances as well as deck widths and required load capacities to be confirmed with the Central Hawke's Bay District Council during detailed design.

Farm crossings - bridges	Allowance for 9 proprietary double tee bridges (or approximately 1 bridge per 2 km of primary system canal). Deck width 3.6 m clear span 14 m between bearings to carry Class 1 loading. Abutments protrude into canal as required.
Stormwater crossings	Sized to pass 100 year ARI flood from minor streams crossing under the canal (additional to five major storm water crossings where the canal passes under significant streams or rivers by inverted siphon).
Primary Distribution System Pipeline	
Pipe material	High density polyethylene up to 1,600 mm diameter.
Pipe horizontal and vertical alignment	Bend diameter not to exceed 50 x pipe diameter for polyethylene pipes. Special fittings may be required for larger bends to be confirmed during detailed design.
Pipe roughness	0.011 for polyethylene pipe, in accordance with the New Zealand Building Code.
Pipe jointing	Butt welds for polyethylene pipe.
Pipe bedding	To AS/NZS 2566. Top of bedding minimum of 0.6 m below existing ground level. In situ material used for bedding subject to geotechnical investigations.
Pipe scour	Provision of scour points at low positions in the alignment where the primary pipeline cannot be drained by secondary pipelines.
Air release	Provision of air release valves at high point in the alignment.
Pipe line access	Sealed manholes at maximum 2 km centres for pipe diameters greater than 1.0 m.
Pipeline abrasion resistance	Not specifically considered and to be considered during detailed design.
Crossing of existing gas pipeline	Water pipeline to go under the 200 mm NB gas pipeline with a minimum cover of 0.5 m vertical separation from the underside of the gas pipeline (as advised by Vector). All works within 6 m of gas pipeline require prior work permit.
Secondary Distribution System Pipelines	
Intake screening	No provision for fine screening of inlets to secondary pipelines.
Pipe material	High density polyethylene.
Crossing of existing gas pipeline	Water pipes to go underneath, minimum 0.5 m vertical separation. All works within 6 m of gas line require prior work permit.
Pipe roughness	0.011 in accordance with the New Zealand Building Code.
Pipe jointing	Butt welded.
Pipe bedding	To AS/NZS 2566. Top of bedding minimum of 0.6 m below existing ground level. In situ material used for bedding.
Pressure at turnouts/connections to users	Positive pressure.

Air release	Provision of air release valves at high point in the alignment.
Pipe scour	Provision of scour points at low positions in the alignment.
Vector gas main crossing	Water pipeline to go under the 200 mm NB gas pipeline with a minimum cover of 0.5 m vertical separation from the underside of the gas pipeline (as advised by Vector). All works within 6 m of gas pipeline require prior work permit.
Pipeline abrasion resistance	Not specifically considered to date, to be considered during detailed design.
Turnouts/connections to users	Assume that all users may connect directly to either primary or secondary distribution canals/pipe work. Utilise proprietary hydrometer or similar. Flow rates for users, and number of users per distribution line to be advised by HBRC. HBRC to advise specific requirement for stock water low flows.
Outfalls	An outfall adjacent to the Mangaonuku Stream with a capacity of 2.69 m^3/s capacity (to accommodate 1.77 m^3/s Zone M demand and 0.92 m^3/s for downstream users) An outfall structure adjacent to the Kahahakuri Stream with 0.35 m^3/s capacity

5.4 Upstream water intake

5.4.1 Intake functionality, considerations and constraints

5.4.1.1 Intake site – location, topography, geology and river morphology

The Application Design upstream water intake site is located on the RHS (southern) bank of the Waipawa River near the 260 m RL contour (Hawke's Bay datum) as shown on Drawings 27690-DN-100 to 104. The site is approximately 1.6 km upstream from the point where there is current access to the Waipawa River bed from the northern end of Caldwell Road.

One intake on the south bank of the Waipawa River is proposed to serve Zones A-D directly and Zone M indirectly (via the Zone A Mangaonuku outfall and the downstream water intake). An inverted siphon would convey Zone A flows east from the Zone B primary distribution system headrace canal.

This intake site was initially selected from the approximate reach of the river that could provide the required primary system water level (at least 250 m RL based on prior work) to maximise the area of the scheme that could be fed by gravity. The location of the intake was chosen based on river morphology, site topography and the presence of a favourable rock outcrop at the outside bend of the current river braid. Based on qualitative interpretation of aerial photograph information and precedence elsewhere, the river braid is considered to be relatively stable at this location.

The Waipawa River is constricted at the proposed intake site between a significant terrace riser feature on the true left bank (approximately 20 m high) and a lower rock outcrop on the true right bank (approximately 10 m high). The true right bank at the proposed intake location outcrop comprises a sub vertical face of extremely weak pumacious sandstone.

Immediately south of the site, the river channel substantially widens. The increase is approximately four times the constriction width. There is a wide and relatively flat low level terrace on the true right bank immediately adjacent to the constriction that is a

suitable area to locate the intake where it is protected to a degree from flood flows by the rock outcrop.

Photograph 5.1 below (view looking upstream to the south west from the Waipawa River) illustrates the rock outcrop. The sub vertical face is assessed as sufficiently stable and of acceptable quality to construct the intake against.

Photo 5.1 Proposed location for upstream intake site (looking upstream from true right bank towards rock outcrop)

The site exploits the "outside bend" effect, where the river is generally stable adjacent to a significant promontory feature. Locating the intake next to a fixed river boundary (such as the rock outcrop hard point that is exploited in this instance) has distinct advantages in terms of minimising river training works requirements and improving ease of flow capture. Observed performance of similarly located intakes on rivers with similar morphology elsewhere in New Zealand suggests that the river flow tends to stay beside the fixed boundary/rock outcrop.

The selected location also enables more expensive and vulnerable aspects of the intake (e.g. gate structures) to be located away from the river, and downstream of a settling area that can be located just south of the infiltration bund and gallery pipework, reducing the risk of significant damage to the intake due to river floods.

5.4.1.2 Intake screening and fish exclusion

A rock fill infiltration bund with internal gallery pipework is proposed for fish exclusion. This arrangement is also consistent with the intent of the NIWA Guidelines.

5.4.1.3 Coarse and fine sediment management

Active management of coarse sediment will be required at the proposed intake location. The Application Design intake arrangement addresses this matter by excluding ingestion of coarse sediment from the intake. The realigned river braid past the intake would be maintained during the life of the project to manage velocities adjacent to the intake to minimise deposition and requirements for excavation of sediment at critical locations.

Subject to final design of the infiltration system it is expected that from time to time suspended sediment may be flushed through the porous infiltration bund. The Application Design assumes that the majority of this material may be intercepted within the intake channel immediately downstream of the gallery pipework. It is anticipated that excavation of accumulated fine sediment would be required from these ponds on a semi-regular basis.

5.4.2 Selected general arrangement

Drawings 27690-DN-100 to 27690-DN-104 illustrate Application Design details of the upstream water intake. As outlined in the preceding sections the proposed intake arrangement is intended to address four key considerations/constraints that are:

i. Conveyance of the design flow of 11.1 m^3/s from the Waipawa River to the primary system canal over the range of design water levels

ii. Management of coarse and fine sediment inflows into the intake. The intake needs to accommodate the potential for coarse sediment deposition at the intake because of the high proportion of flow taken from river

iii. Flood damage to the intake, especially expensive screens and gates

iv. Exclusion of fish from the primary system canal.

Accordingly, an arrangement is proposed that endeavours to address these major risks based on successful recent precedents in similar river environments. The main components of the intake illustrated on Drawing 27690–DN-101 comprise:

- A realigned and modified river channel to form the Intake River Braid at the interface with the rock infiltration bund. Current expectations are that the invert of the modified braid would be in the order of 15 m wide and would slope at about 200H:1V or 0.5 %. It is expected that from time to time that it will be necessary to maintain the Intake River Braid during the life of the project to manage flow velocities and water levels near the intake to minimise insofar as is possible gravel deposition and requirements for excavation of sediment at critical locations
- An infiltration bund with internal gallery pipework approximately 140 m long to abstract the design headrace flow of 11.1 m^3/s

  Very permeable rock armour at the interface of the Intake River Braid and the infiltration bund will protect the intake from flood conditions within the river. Application Design indicates that the armour D_{50} size will be around 600 mm for limestone for the advised specific gravity of 2.35 t/m^3. The length of rock armour to the start of the rock groyne (refer below) is approximately 200 m

  The 1 m diameter infiltration gallery pipe work comprises seven number tee sections each 20 m long parallel to the Intake River Braid as illustrated on Drawings 27690-DN-101 and 102. Current indications are that the gallery pipework would be fabricated from woven steel mesh with an aperture of around 30 mm hot dip galvanised after fabrication.

  The exact arrangement of the infiltration bund and internal gallery pipework is subject to later Optimisation during detailed design
- A stopbank around key areas is provided to suit the HBRC advised flood protection works criteria. The advised criteria comprises the 100 year flood level with a 1 m freeboard/uncertainty allowance to account for sediment flux and other uncertainties in estimating flood levels. Based on the Feasibility Stage interpretation of the Waipawa River flood level and cross section information provided by HBRC, the crest of the stopbank is set at 262.9 m RL

A rock groyne approximately 40 m long provides additional flood protection adjacent to the Intake River Braid at the downstream end of the infiltration bund. The crest of the groyne is set at 261.9 m RL coincident with the nominal 100 year flood level and 1.0m lower than the adjacent stopbank crest and intake access road formation.

Smaller rock armour (D_{50} assumed to be 300 mm) is located along the left hand side of the Inlet Channel and primary system canal embankment for a distance of approximately 240 m to the south of the rock groyne

- The Intake Channel captures flow from the infiltration bund and conveys abstracted water to the reinforced concrete radial gate structure. The Intake Channel will also trap a large proportion of suspended sediment particles above 0.2 mm in the event that these pass through the coarse filter pack during flood events. The Intake Channel has been proportioned based on the maximum suspended sediment particle size of 0.2 mm criteria insofar as capture of fine sediment is concerned. A 1 m deep zone is included for sediment accumulation and the design flow depth is 0.7 m with this zone full of sediment. A bench near existing ground level is included to assist with manual cleaning of the Intake Channel by long reach excavator
- A reinforced concrete gate structure is provided to house an underflow radial gate that will provide fine flow/level control for flow into the primary system canal over the range of design water levels anticipated within the Intake Channel. The reinforced concrete structure features a culvert to enable vehicle access around the intake area. The structure also contains stop log slots to enable isolation of the culvert, radial gate and primary system canal from river level given the very permeable nature of the rock infiltration bund. The minimum Intake Channel water level for gate operation is set at 258.6 m RL, 0.8 m lower than the minimum river level to pass the design flow of 11.1 m^3/s at the north western extent of the Intake River Braid.

Photo 5.2 Indicative example of a radial gate structure provides a general perspective of the radial gate structure proposed for the upstream water intake.

- An intake access road is provided along the crest of the primary system canal to enable vehicle access to the intake area. Access to the river for works in the bed of the Waipawa River in the vicinity of the Intake River Braid is also included
- Power supply to the intake is necessary to enable a hydraulic power pack to operate to control the radial gate.

The detailed design stage assessment should also consider whether it is necessary to include a by-wash from the Intake Channel back to the river to continually flush water through the infiltration bund (including at times of no irrigation), in order to minimise the potential clogging risk at the intake. If this additional feature is required, then the implications for fish exclusion would also need to be considered. This work should also include a detailed reconciliation of the water surface profiles within the Intake River Braid and Intake Channel for a range of river flow and level scenarios. Refinements to the presented Feasibility Stage assessments may also be required to reflect the as yet undefined groundwater regime at the intake site.

5.5 Primary distribution system headrace canal – Zones A to D

5.5.1 Alignment

Drawings 27690-DN-232 to 240 illustrate the Application Design primary distribution system headrace canal alignment for Zones A to D. Drawings 27690-DN-217 shows typical canal cross sections.

The alignment of the headrace canal is constrained by the topography of the area and follows the base of hilly ground to maintain head whilst minimising earthworks. The elevation of the intake site (approximately 260 m RL) determines the maximum elevation of the canal. Based on the Application Design intake discussed in the preceding section, the primary distribution system headrace canal water elevation (hydraulic grade) adopted for assessing alignments is 258.2 m RL at the hydraulic jump location immediately downstream of the radial gate structure.

All of Zone A is proposed to be piped. A siphon under the Waipawa River about 300 m downstream from the upstream water intake connects the Zone A piped system to the primary distribution system headrace canal within Zone B.

The primary distribution system headrace canal supplies all of the Zone B and part of Zone C secondary system via a canal south from the intake past Ongaonga and through several siphons under the Kahahakuri and Ongaonga streams. The canal then transitions to the primary distribution system pipeline near Ashcott Road. The primary distribution system pipeline then services the remainder of Zone C and all of Zone D. The canal end point at this location avoids a bridge over Ashcott road, the gas pipeline and a major storm water crossing under an appreciable but unnamed tributary of the Tukipo River. Supply to the areas east and south of the headrace is provided by the secondary distribution system pipelines which are required to cross the Vector gas pipeline to supply the southern area of Zone C and all of Zone D.

The Application Design has been refined following initial discussions with landowners on the route.

5.5.2 Canal cross section and liner

2H:1V slopes have been adopted for cut and fill profiles from the canal crest to existing ground level outside of the primary distribution system headrace canal. The cut batter slope and fill embankment heights have been restricted to a maximum of 15 m, to reduce the likelihood of potential slope stability issues.

Allowance is made for some landscape fill/cut to waste, rather than a balanced cut to fill. This is because the rate for cut to waste/landscape fill is generally considerably cheaper than balanced cut to engineered fill. This approach is often quicker too.

A minimum freeboard of 0.3 m has been adopted to allow for wave action and other channel transients. The suitability of this freeboard allowance will require confirmation during detailed design when canal transients are specifically assessed.

Based on preliminary understanding of the geology along the primary system canal alignment, substantial sections of the trapezoidal channel may require a liner to minimise canal leakage and demonstrate efficient water use consistent with current industry practice. Preferred lining options comprise either a 1.5 mm HDPE membrane liner or an earthen liner.

Two trapezoidal channel options to meet the design flow and freeboard criteria and featuring the two lining options are illustrated on Drawing 27690-DN-217. These are:

- An HDPE lined canal
- An earth lined canal.

5.5.3 Drop structures

The primary distribution system headrace canal requires a series of drop structures within Zone B between the intake and Ongaonga to safely drop the channel water level to the vicinity of the 220 m RL contour at the base of the foothills. Dropping the canal to 220 m RL avoids the excessive earthwork quantities that are otherwise necessary to maintain the canal at the higher elevation.

Five drop structures are required to suit the alignment topography. Sharp crested weirs are associated with the drop structures and are necessary to assist with water level control as described in the following section. The reinforced concrete drop structures are based on the well proven standard USBR baffled drop arrangement. Energy is dissipated by a series of baffle blocks located within a reinforced concrete chute, sloping at 2.5H:1V in this instance. Photo 5.3 illustrates a typical example of the proposed structure.

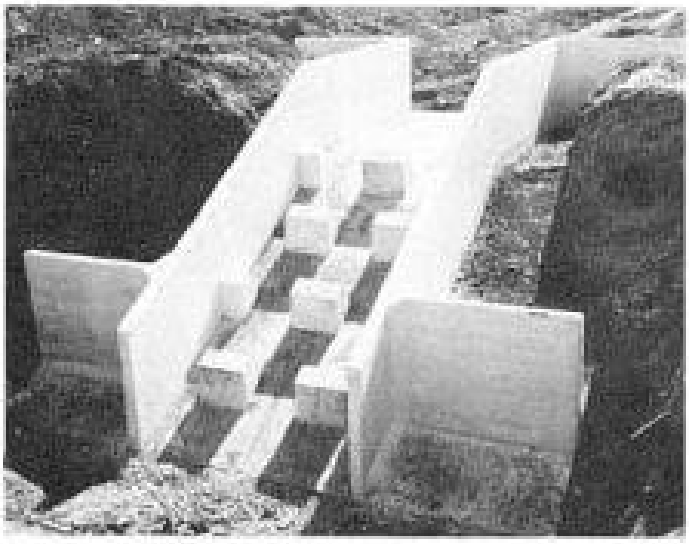


Photo 5.3 Typical baffled drop installation.

Drawings 27690-DN-300 and 301 illustrate arrangements proposed in this instance. The drop structures are situated at the following locations:

- Two 5 m high drop structures south of Caldwell Road and east of Wakarara Road
- Two 6 m high drop structures south of Caldwell Road and east of Wakarara Road

- One 8 m high drop structure west of Wakarara Road and north of SH 50 near Cottesmore Farm.

5.5.4 Water level management

In addition to the radial gate associated with the intake described previously, the primary distribution system headrace canal includes a number of measures to provide the necessary level of water level control. The features comprise:

- Two proprietary three sided Rubicon hydraulically actuated slide gates located respectively at the inlet to the Waipawa River Siphon at the start of Zone A and the Tukituki River Siphon at the south end of Zone B. The gates are required to control flow beyond the gates and to maintain the canal parked water level upstream of the gates to enable conveyance of stock water flow to secondary system pipelines outside of the irrigation season.
- Five sharp crested weirs associated with baffled drop structures, are included to maintain the canal parked water level upstream of the drop structures, and to enable the conveyance of stock water flow to secondary distribution system pipelines outside of the irrigation season. It is noted that geotechnical investigations for the earthen liner may identify requirements to limit the number of wetting and drying cycles that an earthen liner constructed from site materials can accommodate. In this instance revision to the proposed weir arrangement may be required.
- Emergency overflows/spillways from the primary distribution system headrace canal are required to ensure that unsafe water levels cannot occur within the canal, particularly within fill areas. These emergency overflows need to accommodate:
- The safe discharge of stormwater that may enter the canal from cut areas
- Flows already within the primary distribution system headrace canal if users do not take the correct quantity of pre-ordered water.

For this reason it is important that the capacity of the primary distribution system headrace canal is not reduced between emergency overflow locations (for example the capacity of the Zone B canal is maintained at 7.8 m^3/s to the emergency sidespill proposed for the LHS/north bank of the Tukituki River at the siphon inlet).

Emergency overflows/spillways comprise:

- 3.3 m^3/s side spill broad crested weir to the RHS of the Waipawa River near the bifurcation to Zone A
- 3.8 m^3/s side spill broad crested weir to the LHS of the Tukituki River at the south end of Zone B
- 4.1 m^3/s side spill broad crested weir to the RHS of the Tukituki River at the south end of Zone B.

5.5.5 Inverted siphons

Eight inverted siphons are required to preserve the primary distribution system headrace canal hydraulic grade line at locations where the hydraulic grade line is higher than existing ground level and existing ground level must be maintained and/or impact on the flood capacity of significant rivers and channels must be avoided. These positions are where the alignment crosses Pettit Valley Road, SH50 and the following six major water courses:

- Waipawa River between Zones A & B
- Kahahakuri Stream in two locations within Zone B

- Ongaonga Stream in two locations within Zone B
- Tukituki River between Zones B & C.

Drawings 27690-DN-400 to 402 illustrate arrangements adopted for the Application Design. Inverted siphons comprise a single barrel constructed from reinforced concrete rubber ring jointed (RCRRJ) pipes. Pipe diameters are 1,600 mm in Zone A and C and 1,800 mm within Zone B.

Inverted siphons include reinforced concrete headwalls at both the entry and exit. The Waipawa River siphon does not require an exit headwall structure as the siphon connects directly to a buried piped system. Inlet headwalls to inverted siphons are standard precast units, with levels to suit the 300 mm inlet submergence criteria recommended by the USBR[1]. The inlet headwall to the Tukituki River is compatible with the proprietary Rubicon gate proposed for this location. Photo 5.4 shows typical siphon construction.

Proprietary headwall structures are also utilised for the siphon outlets (except at the Waipawa River siphon) with an armoured trapezoidal channel section immediately downstream of the outlet to provide some scour protection. Submergence is not required at siphon outlets.

An important inclusion will be security fencing and signage to limit public access and highlight risks associated with both intentionally or inadvertently entering the primary system canal near siphon inlets.

Photograph 5.4 Example of comparable inverted siphon construction.

5.5.6 Primary distribution system canal minor storm water crossings

Provision for storm water crossings under the primary distribution system headrace canal is required where the canal alignment intersects existing overland flow paths. Several situations arise depending upon whether the canal is formed in cut or fill at the crossing location. Drawings 27690-DN-232 to 240 illustrate the locations where minor stormwater crossings are provided.

[1] U.S. Bureau of Reclamation; Design of Small Canal Structures; 1978.

5.5.7 Bridges

5.5.7.1 Road bridges

The Application Design includes provision for 3 road bridges at Central Hawke's Bay District Council road crossings along Caldwell Road, Ngaruru Road and Wakarara Road. Bridge clear spans will be up to 15 m. Abutments may protrude into the canal as required. Given limited opportunities for scour, shallow foundations are assumed.

The design assumes that minor amendments of road vertical alignments are achievable to suit the primary distribution system headrace canal hydraulic gradeline.

It has been assumed that 2 bridges (Caldwell and Ngaruru Roads) have a deck width of 5.4 m comprising three proprietary 1.8 m wide double tee sections. With allowance for kerbing the clear deck width would be approximately 4.4 m. Structural capacity to carry Class 1 loadings has been assumed.

It has been assumed that the Wakarara Road bridge will have two lanes and a deck width of 8 m comprising seven proprietary 1.145 m wide double hollow core sections. Structural capacity to carry HN/HO loadings has been assumed.

5.5.7.2 Farm bridges

The Application Design includes an allowance for 9 farm crossings over the primary distribution system headrace canal (or approximately 1 bridge per 2 km of primary system canal).

It has been assumed that farm bridge decks have a 3.6 m width and comprise two 1.8 m wide proprietary double tee units that have a 14 m span between bearings to carry Class 1 loadings.

Farm bridges will feature shallow foundations. Based on similar precedents elsewhere, abutments may protrude into the canal as required due to span bridge deck span limitations.

5.5.8 Secondary distribution system pipeline connections

Drawings 27690-DN-002 to 004 illustrate layouts adopted for the purpose of Application Design assessment for the secondary system connections to the primary works canal. The arrangements feature RCRRJ pipes bedded in concrete in the immediate vicinity of the canal liner to minimise leakage. The inlet to the canal will feature a nominal coarse screen to prevent large debris entering the intake. The pipework leads to a manhole structure that may house an isolation valve and/or pipework for secondary system booster pumping stations. Photograph 5.5 illustrates a similar arrangement successfully utilised elsewhere.

Photo 5.5 Example Secondary System inlet from primary system canal

5.6 Primary distribution system pipeline- Zones **A** to **D**

Pipe reticulation is the preferred means of distribution where judged to be cost effective relative to canal construction. Thus, the primary distribution system features a pressurised pipeline for all of Zone A, part of Zone C and all of Zone D. The presence of a significant watercourse, Ashcott and Balfour Roads and the Vector gas pipeline also contribute to the location where the Primary System transitions from canal to pipeline in Zone C.

The primary distribution system pipeline diameters vary from a minimum of 630 mm (at the southern end of Zone D) to 1600 mm (at the transition from the siphon to pipeline in Zone A). Diameters have been selected based on velocity, driving pressure and head loss considerations.

The primary distribution system pipeline for Zones A to D has been aligned roughly through the western centre of the scheme, with secondary pipelines branching to both sides, to minimise the required diameters of the secondary pipelines, while also attempting to minimise the area requiring pumping by moving the primary line too far east. Drawings 27690-DN-001 to 27690-DN-004 illustrate the alignment.

Where possible, the primary distribution system pipeline has been aligned to follow roads or property boundaries in consideration of likely access and easement requirements. The pipeline is aligned to avoid existing structures where possible and, in addition to several crossings under significant water courses and Central Hawke's Bay District Council roads, the alignment also crosses the Vector gas main (minimum 0.5 m vertical separation and all works within 6 m of gas line require prior work permit), State Highway 2 and the Palmerston North to Gisborne railway line near Takapau.

Pipe materials were selected based on lowest estimated costs. For the range of diameters considered, HDPE with butt welded joints has been identified as the most cost effective material.

5.7 Secondary distribution system – Zones **A** to **D**

The Application Design secondary distributions system works that convey water between the primary system and individual farm off-takes comprise of buried HDPE pressure pipelines illustrated in plan on Drawings 27690-DN-001 to 004, identified in Table 5.3.

Table 5.3 Secondary distribution system pipelines

Line	Line
1. SH50 - Mangamate Stream	25. Swamp Road
2. Matheson Road	26. Ongaonga Road East
3. SH50 - Mangamauku Stream	27. Ongaonga Road South
4. Smedly Road	28. Fairfield Road West
5. Butler Road	29. Fairfield Road East
6. Mangaoho Stream	30. Hobin Road
7. Tikokino East	31. Ongaonga Waipukurau Road
8. Tikokino Road East	32. Tukituki River
9. Glenalvon Road North	33. Ashcott Road West
10. Springhill	34. Balfour Road
11. SH50 - Glenalvon Road	35. Ashcott Road East
12. Caldwell Road	36. Burnside Road North
13. Makaroro Road	37. Burnside Road South
14. SH50 - Waipawa River	38. Speedy Road
15. Tikokino Road	39. SH2 East
16. Wakarara Road West	40. Burnside Road West
17. Te Papa	41. SH2 West
18. Wakarara Road East	42. Nelsons Road
19. Ongaonga Stream	43. Maharakeke
20. Plantation Road	44. Oruawhara Road
21. Purinui	45. Awanui Stream
22. Pettit Valley Road	46. Seeforth
23. Taylor Road	47. Hinerangi Road
24. SH50 - Ongaonga West	

Where possible, secondary distribution system pipelines have been aligned to follow roads or property boundaries in consideration of likely access and easement requirements and as endorsed by the reviewer. The pipeline is aligned to avoid existing structures and disruption to farms where possible. In addition to several crossings under significant water courses and Central Hawke's Bay District Council roads, the alignment also crosses the Vector gas main (minimum 0.5 m vertical separation and all works within 6 m of gas line require prior work permit).

HDPE has been selected as the pipe material for the secondary reticulation on the basis of material cost and installation requirements. Refer to the preceding section for discussion on pipe bedding, jointing, manufacture and hydraulic roughness that is also applicable to the secondary distribution system pipelines. Various fittings will also be required including isolation valves, scour valves at low points, and air release valves in some locations.

5.7.1 Secondary distribution system booster pumping

In addition to boosting of pressure on farm, booster pumping is also necessary at several locations within the secondary distribution system to provide positive pressure to all areas within the production land use areas that were determined as not being able to be supplied by gravity. Approximate pumping requirements have been estimated based on the required design pump head and flows as summarised in Table 5.4 following. The location of the pump stations are included on Drawings 27690-DN-002 to 27690-DN-004, which are generally for the secondary lines located west of the primary system works.

Table 5.4 Summary of secondary distribution system booster pumping stations

Pumping Station	Peak flow (l/s)	Maximum pump head (m)	Installed pump capacity (kW)
Richardsons Bridge	220	28	90
Matheson Rd	99	17	22
Sedgwick St	167	33	75
Tikokino	728	27	275
Springhill	96	63	90
Caldwell Rd	503	10	88
Ngaruru Rd	114	33	60
Pettit Valley Rd	225	35	90
SH50 West	200	28	90
Ashcott	760	60	660
Balfour Rd	495	8	75
Burnside Rd	261	60	220
SH2	116	17	37
Fraser Rd	752	11	132
Awanui	354	18	92.5

The required pumps at each pump station have been selected to supply the pump flows and heads as determined by the pipe reticulation model. Between two and six pumps have been adopted at each location, as required, to provide a range of flows depending on demand. The motors on each pump range from 11 kW to 110 kW and are either 4 pole or 6 pole.

Some pump stations have been assumed to be able to be combined into one pump shed. This has been implemented at five locations. Simple pump sheds have been assumed and may require building consents from Central Hawke's Bay District Council. No noise protection has currently been allowed for but may be required, depending on pump station location and applicable noise limits in the resource consents.

5.7.1.1 Proposed Outlet Structures

Outlets capable of energy dissipation are necessary because the system will otherwise cause stream damage. This stream damage would be caused by the high pressures that may

occur when the Scheme has few users, such as during the initial phase after construction. Outlet structure locations are shown on Drawing 27690-DN-001 and are located to suit environmental constraints including fish spawning and related issues as advised by Cawthron.

The Application Design proposes outlet structures including three stilling basins and a plunge pool. The stilling basins have been designed following steps set out by the United States Bureau of Reclamation and the plunge pool was designed following a design note produced by the United States Department of Agriculture.

Outlet structures are proposed for the following scenario:

- 2.69 m^3/s To accommodate 1.77 m^3/s Zone M demand and 0.92 m^3/s for downstream users.

A stilling basin is a concrete structure that uses a baffle to dissipate energy before the water is discharged into a riprap lined channel. A plunge pool, located on the Kahahakuri Stream, uses a drop from height to dissipate energy; water drops into a riprap lined trapezoidal basin. Both means of energy dissipation require provision for automatic flow control.

The sections of channel downstream of these structures may require upgrading to increase the capacity.

Photo 5.6 Typical view of Mangaonuku Tributary

A stilling basin is proposed for installation into the Mangaonuku Tributary (Photo 5.6). The channel will require local upgrade works including consideration of an appropriate flood control with release.

5.8 Downstream water intake

5.8.1 Intake functionality, considerations and constraints

Many of the issues relevant to the proposed upstream water intake, for example river morphology and management of coarse and fine sediment, are also relevant to the downstream water intake. The following discussion relating to the proposed downstream water intake should be read in conjunction with Section 5.4 of this report. Application Design Drawing 27690-DN-110 (Appendix A) illustrates the downstream water intake site and general arrangement. A summary of the arrangements is presented in Table 5.5.

Details of the Zone M primary distribution system are contained in March 2013 EMS report Zone M Primary Distribution Concept Review and Assessment and are summarised in Section 5.9.

Table 5.5 Ruataniwha Water Storage Scheme – Downstream water intake design parameters/specifications

Flow Rate	Adopted specification
Intake capacity	1.82 m^3/s capacity. This allows for 1.77 m^3/s diversion to the Zone M for irrigation and 50 l/s residual flow excluding by-wash flow for fish passage return to the Waipawa River.
General arrangement	Stopbank works to protect intake and primary system canal from floods up to the existing design event (100 year ARI). Requirement to exclude coarse sediment and significant quantities of fine sediment from the primary system. Fish screen and intake bypass or equivalent to exclude fish from the primary system to meet environmental performance requirements advised by Aquanet Consulting Ltd. Inlet control to Zone M distribution system by underflow gate to accommodate the peak Zone M demand and Waipawa River water levels at the intake location within the operational range defined below.
Operational range	Operational range to coincide with the 7-day MALF and 1 year ARI flood respectively. A lower flood standard has been adopted than the upstream water intake to limit the sediment inflows in higher flood situations. Corresponding operational range between 122.4 m RL and 124.3 m RL at the inlet structure as inferred from data provided by HBRC.
In-river maintenance work	Due to the braided nature of the Waipawa River, in-river works will be necessary from time to time throughout the operational life of the project to maintain the river channel (flow and level) at the intake location, and manage the deposition of coarse sediment in the vicinity of the intake and Intake Diversion Channel.
Intake structure flood protection	Maintain existing Bishop's Bank crest level that HBRC advice provides 0.6 m free board to the 100 year flood level. Works west of Bishop's Stop bank are at risk of flood damage from significant flood events beyond the operational range set out above.

Fish exclusion	With respect to the exclusion of fish from the Zone M distribution system the intake arrangement shall be designed so as to exclude adult trout. Above criteria to be adopted for any screen variant that may be provided as an alternative to the rock infiltration gallery described and illustrated on Drawing 27690-DN-110.
Intake Structure head loss	Estimated level difference of 2.4 m from the Waipawa River 7-day MALF river level of 122.4 mRL at the proposed intake location to a potential distribution system water level coincident with existing ground level close to 120.0 m RL.
Exclusion of fine sediment	Adopted criteria previously confirmed by HBRC for the Upper Waipawa intake arrangement to remove a high percentage of particles coarser than 0.2 mm.
Exclusion of coarse sediment	Intake design to include potential for exclusion of coarse sediment and to cater for accumulation of some gravel in the vicinity of the intake prior to physical removal. A guard gate will be included to shut off the system for some larger events.
Debris exclusion	Gallery Intake option: Aside from settling ponds and fish exclusion no specific provision to remove other debris that may have potential to compromise efficiency of irrigation equipment (e.g. floating weed and the like that may otherwise enter or accumulate within the primary system canal). Screen option: to be equipped with automated screen cleaner to remove weed and debris from the Zone M distribution flow (this is appropriate if diverted water contains weed that may pose blockage issues at irrigator nozzles).

5.8.1.1 Intake site – location, topography, geology, and river morphology

The downstream water intake is a relatively small structure and is located on the left hand (northern) bank of Waipawa River downstream of Waipawa and a little under 1 km upstream of the confluence with the Tukituki River in the vicinity of the Bishop's Bank area. The site is 25-30 km south of the proposed Upper Waipawa River intake. Access to the intake area is via Walkers Road.

Two location options have been identified as potentially suitable sites for the intake; a final decision will be made during detailed design. The southern site is located on HBRC administered land. The choice will be made accounting for land availability, river morphology, confirmation of the Zone M distribution design) and further site evaluation at detailed design stage.

The former Waipawa River alignment followed what is now referred to as the Old Waipawa River Bed before former joining the Papanui Stream, and then entering the current Tukituki River 25 km downstream of the proposed intake. The Bishop's Stopbank was constructed some one hundred years ago to enable the diversion of the Waipawa River into the Tukituki River along the present river alignment to facilitate land development adjacent to the former river bed.

EMS (2013) explains the concept of utilising the Papanui Stream (Old Waipawa River Bed) as a key component of the Zone M distribution system. Therefore, the downstream water

intake is located to facilitate a diversion to the Old Waipawa River Bed as is shown on Drawing 27690-DN-110.

At the proposed intake, the river and the stopbank are approximately 300 m apart and the area between is relatively flat (Photos 5.7 - 5.10). We have inferred from LiDAR contours that the area has aggraded since construction of Bishop's Bank by around 2 m. We understand that this area between the main river and Bishop's Bank floods during significant rainfall events; flooding is likely as frequently as the 2 to 5 year AEP event (HBRC advice). It is necessary to ensure that the works in this area are able to withstand such flooding.

A single river braid is almost always present along the straight Waipawa River reach in the vicinity of the intake. Nonetheless, the potential exists for the braid to move within the river bed. In these situations in-river work will be required to divert a relatively modest flow a short distance back to the intake location. The intake is not sited to exploit a promontory or other hard point on the edge of the channel. HBRC has indicated that to a large extent this sort of river training work and coarse sediment removal work may be undertaken at minimal cost by local contractors that require aggregate.

There is a lack of specific information about the groundwater regime. As a result, uncertainties exist around losses, ground water inflow and potential construction complications. These matters should be addressed at the detailed design stage.

Photo 5.7 *Proposed location for downstream water intake (looking from the river towards Bishop's Bank and the proposed settling pond location in the foreground)*

Photo 5.8 *Proposed location for downstream water intake (looking from Bishop's Bank towards the river). By-wash channel to follow existing channel alignment shown in the foreground.*

Photo 5.9 *Waipawa River in the vicinity of the proposed downstream water intake as at 28 November 2012*

Photo 5.10 *Looking along the river bed margin towards NNW direction across proposed Diversion Channel and By-wash Channel culverts.*

5.8.1.2 Intake screening and fish exclusion

An infiltration gallery or mechanical screen (latter given the modest intake flow) is proposed for fish exclusion.

5.8.1.3 Coarse and fine sediment management

Active management of coarse sediment will be required at the proposed intake location. HBRC has indicated that given the close proximity of the intake location to Waipawa, to a large extent it will be possible to rely on others to achieve this (e.g. by way of gravel extraction for use as aggregate). It is expected that the river braid past the intake will be maintained during the life of the project to manage velocities past the intake to minimise (as far as possible) deposition and requirements for excavation of sediment at critical locations.

5.8.2 Selected general arrangement

Drawing 27690-DN-110 illustrates the Application Design general arrangement of the downstream water intake. As outlined in the preceding sections the proposed intake arrangement is intended to address four key considerations/constraints that are:

i. Conveyance of the design flow of 1.82 m^3/s (including a 50 l/s residual flow) from the Waipawa River to the distribution system over the range of design water levels

ii. Management of coarse and fine sediment inflows into the intake. The intake needs to accommodate the potential for coarse sediment deposition at the intake because of the significant proportion of flow taken from river

iii. Flood damage to the intake, especially expensive screens and gates

iv. Exclusion of adult trout from the primary system canal.

An arrangement is proposed that endeavours to address these major risks based on successful recent precedents in similar river environments. The downstream water intake has not been externally peer reviewed; however, it is considered that the concepts are consistent with the philosophy that the external reviewer has endorsed for the upstream water intake. The main components of the Application Design general arrangement illustrated on Drawing 27690-DN-110 compromise:

i. A Diversion Channel aligned more or less orthogonal to the river to divert water from the river. The channel capacity/diversion flow may be around 2 m^3/s including by-wash to facilitate fish passage back to the river and the 1.82 m^3/s flow abstracted into the Zone M distribution system. The channel will be approximately up to 90 m long and include nominal allowance for the accumulation of coarse sediment. From time to time it will be necessary to remove/excavate sediment from this channel. Rock protection will be necessary at the Diversion Channel bifurcation from the main river channel. A culvert will be provided adjacent to the existing fence on the main river channel margin and a vertical lift steel guard gate provided on the culvert head wall. The purpose of the gate is to isolate the settling pond from the river during high flows, including minimising to some extent the quantity of coarse sediment that is ingested into the channel. It is envisaged that gate actuation will be provided by a small ram and the ram and associated control systems mounted at an adequate height to avoid damage during the more frequent floods that inundate the area west of Bishop's Bank

ii. The Diversion Channel will discharge to a Settling Pond. The purpose of the Settling Pond is to remove suspended sediment prior to the gallery intake. The settling pond will be provided with a broad crested weir at the By-wash Channel outlet to maintain the pond water level

iii. Gallery Pipework will be buried at the eastern end of the pond. Subject to detailed design the screen could comprise two number one metre pipes each about thirty metres long. The filter pack thickness below the pond invert would be about 1.2 m deep and the gallery pipes at least 3 m apart. The filter pack would extend round 0.3 m below the base of the screen in accordance with accepted design criteria. The Application Design assumes a screen similar to that proposed for the upper intake, the screen would be 1 m in diameter formed from 8 mm diameter mesh with a 30 mm square aperture. Screen sizing is on the basis that the settling pond provides a constant water level and the gallery operates as a "bed mounted infiltration gallery". Detailed design may identify a requirement to include a compressed air and/or water back flush system to maintain the filter pack clear of fine sediment. The Gallery Pipe work will be set pack from the river for protection. It is anticipated that excavation

of accumulated fine sediment would be required from this pond on a semi-regular basis

iv. The Gallery Pipework will connect to pipework passing under Bishop's Bank. Bishop's Bank will be reinstated to the current crest level and a flow control gate provided on the culvert downstream head wall. A vertical lift gate is expected to provide flow control. Detailed design will consider the stability of the Bishop's Bank foundation as a consequence of locating an unlined settling pond and infiltration gallery in the vicinity of the embankment toe

v. A By-wash Channel will convey excess flow and divert fish back to the river. The channel capacity will be in the order of two hundred litres per second. The channel will follow an ephemeral channel observed near the base of Bishop's Bank. A culvert is included over the By-wash channel near to the existing fence on the river bed margin. The confluence with the main river channel will be armoured as required.

An alternative proprietary arrangement comprises an intake gate and fish by-pass gate. The intake gate would be a typical sluice gate while the fish by-pass gate would have a screen that would exclude adult trout. The fish exclusion gate features a mild steel screen 3.5 x 8.5 m screen mounted at approximately 40° and conveyor and brush automatic cleaning system possibly run on a timer.

Due to the nature of the intake structure, further work is required during detailed design to ensure operational and performance risks associated with this arrangement are manageable within the particular circumstances posed by the Waipawa River at the selected site. Issues to resolve include management of fine and coarse sediment, and potential for clogging of the intake, particularly during flood events. Detailed design may identify a need to introduce a flushing system utilising compressed air and/or water to maintain the effectiveness of a gallery intake in this particular river environment.

5.9 Primary distribution system summary for Zone M

The proposed primary distribution system for Zone M is illustrated in Figure 5.1. The plan route is shown on drawings in Appendix E.

The Waipawa River previously flowed in a north-northeast direction through the area defined as Zone M. Following a historical diversion in the 1880's, the Waipawa River was re-directed to join the Tukituki River at its current confluence. The Papanui Stream now flows down the old Waipawa River bed until its confluence with the Tukituki River. Initially fed from groundwater, the Papanui Stream receives additional flow from run-off from surrounding agricultural land and drainage water from the Te Aute basin, via a man made diversion.

Through nutrient contributions from surrounding agricultural land, low flow, lack of appropriate riparian planting and shading and generally unrestricted stock access, the Papanui Stream can generally be described as degraded.

In summary, the Papanui Headrace Concept (EMS, 2013) is as follows:

- The existing Old Waipawa River Bed / Papanui Stream ("Papanui Stream") is utilised to provide primary distribution of irrigation water to Zone M. The stream would, in effect, function as an open course headrace canal.
- Within specific sections of the Papanui Stream, particularly in the upper reaches where the old river bed is much wider, a channel will be established utilising a combination of canal and stopbank construction within the old bed. In other sections, the existing stream profile may be used. Engineered sections of the canal

would incorporate specific design elements to provide habitat and enhance ecological values.

- Irrigation water supplied by the RWS Scheme will be diverted from the Waipawa River and released into the Papanui Stream in accordance with irrigation demand in Zone M. At peak demand, irrigation flow has been estimated at 1.77 m^3/s. Individual farm irrigation takes would be installed at various locations along the length of the stream.
- Residual flow of 50 l/s will be supplied continuously to the Papanui Stream via a diversion from the Waipawa River to support in-stream ecology.
- Riparian planting will be installed along the entire length of the Papanui Stream. Planting density and species type will be chosen to provide stream shading and filtering of run-off from surrounding agricultural land. There is also potential for wetland development and enhancement in some sections of the stream.
- Riparian margins will be fenced with permanent post and wire fencing to exclude stock from the entire length of the Papanui Stream.
- Provide recreational opportunities, such as developing a walking/cycling path, linking the settlements of Otane and Waipawa at the edge of the new riparian margins along a portion of the Papanui Stream, subject to engagement with and approval by landowners.

The concept is mapped in the plan series attached within Appendix E. A 40 m buffer (20 m either side of the Papanui channel centreline) has been shown to provide for some movement in terms of design around the defined channel.

Hydrodynamic modelling undertaken by HBRC and reported in EMS 2013 has confirmed that the concept can provide an effective means of distributing irrigation water to Zone M.

The concept also presents some significant ecological benefits for the Papanui Stream and wider Tukituki catchment. For example, phosphorus loading can be reduced through riparian filtering and stock exclusion, and the effects of elevated nitrogen and the resultant periphyton and macrophyte growth are expected to be reduced through stream shading from riparian planting. The rehabilitation of the Papanui Stream is understood to be culturally significant and an addendum to the Cultural Impact Assessment Report has been prepared by Taiwhenua o Tamatea to address this.

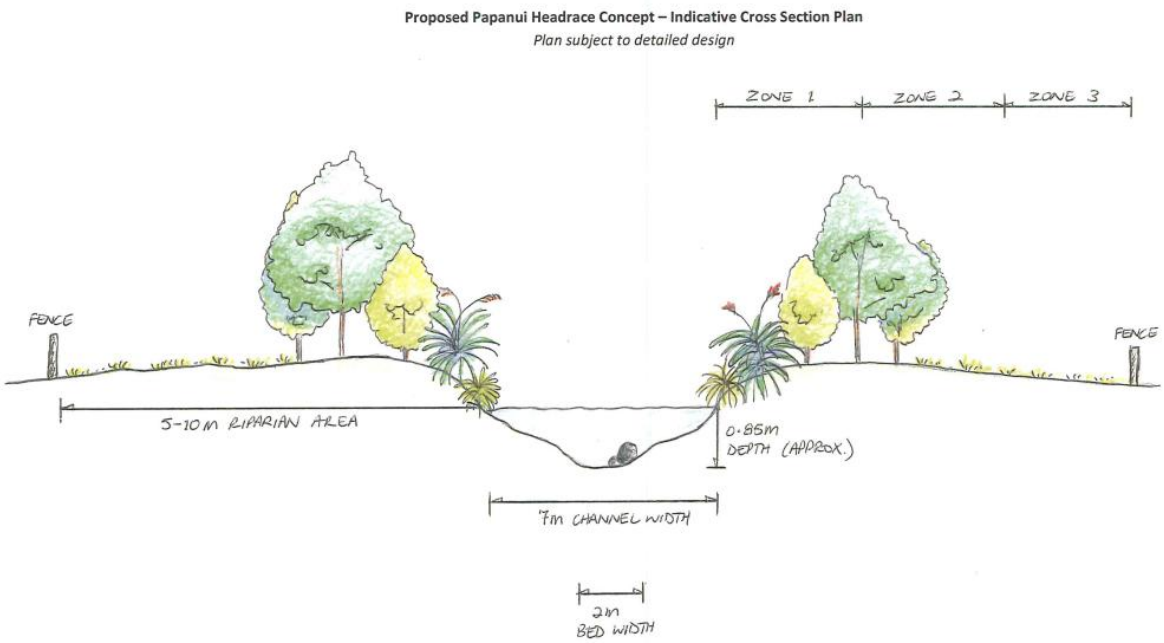


Figure 5.1 Typical cross section of primary distribution system Zone M (EMS 2013)

6 Hydropower generation

6.1 Introduction

Storage dams, while principally for water harvesting, storage and subsequent release for irrigation, also present an opportunity for hydroelectric power generation. The dam and reservoir although intended principally to serve irrigation purposes, effectively represent the headworks of a hydroelectric power scheme. Hydropower may be generated at the base of the dam by attaching a set of turbines and associated electrical gear to the outlet conduit. Other required works comprise a power house to house the equipment, switchgear and switchyard, transmission line to link in with a local network or the national grid, and various minor works.

6.2 Power station at the toe of the dam

The proposed outlet works for the dam are described in Section 4. Aspects of the outlet works that are relevant to the hydropower arrangements are as follows:

- 2.1 m diameter steel pipe (penstock), within the 4.0 m diversion tunnel, leading from the inlet works at the reservoir to the power station, with a tapered reduction to 1.5 m diameter before entering the power house and connecting into the turbine inlet
- Bifurcation of the penstock upstream of entry to the powerhouse to 1.5 m diameter bypass pipe and fixed cone dispersion valve which discharges into the concrete tailbay at the power house
- 12 m diameter circular concrete power house with removable roof for construction and maintenance access – see Figures 6.1 and 6.2 for existing examples of what is being proposed, and Figure 6.3 for a view of a typical internal layout of the power house
- Within the power house, a two-turbine arrangement is proposed, comprising a residual flow generator with a matching design flow capacity of 1.23 m^3/s (0.85 MW) and a main generator with flow capacity of 8.5 m^3/s (5.7 MW), giving a total installed capacity of 6.5 MW; horizontal axis Francis turbines proposed for both
- Draft tube for each turbine leading to concrete tailbay below the power house.

Figure 6.1 Hydropower station and switchyard at Opuha Dam: 7.5 MW, 15 m³/s design flow, power house diameter 9.5 m, considered to best resemble the proposed power station.

Figure 6.2 Power house at TrustPower's Wahapo hydro-electric power scheme: 3.1 MW, 12 m³/s design flow, power house diameter 10 m.

Figure 6.3 Inside TrustPower's Wahapo HEPS power house: single 3.1 MW turbine and generator, 12 m³/s design flow.

6.3 Transmission

Unison Networks Limited (Unison) has a management contract with Centralines, the local electricity distribution company in Hawke's Bay. Unison has a proprietary computer model of the transmission network of the area. Unison has determined that the output from the proposed hydro-electric power station (6.5 MW) is too great to feed into Centralines' nearby 11kV line. Thus, it will be necessary for the power station to connect into Transpower's grid exit point (GXP) located at Ongaonga substation, about 21.5 km (as the crow flies) southeast of the A7 dam site (Figure 6.4).

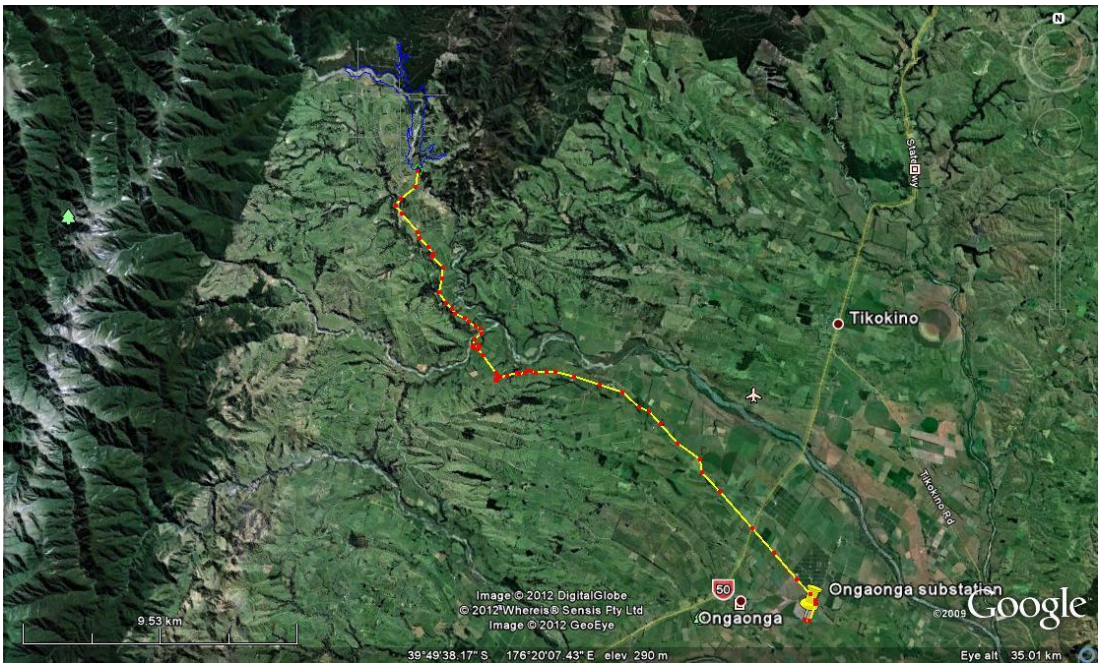


Figure 6.4 Alignment of the proposed 33kV transmission line to the Ongaonga GXP from the A7 dam site.

7 Integrated Mitigation and Offset Approach

Five Projects (Projects A to E) are included in the Integrated Mitigation and Offset Approach (HBRIC Ltd, 2013).

Projects A to C set out biodiversity restoration and enhancement strategies proposed to address residual effects on both terrestrial and aquatic biodiversity. These projects also address effects on recreation, cultural and heritage values associated with the Wakarara Road-end area. Project D provides an additional offset for adverse effects of the project on phosphorus inputs to the streams and the availability and quality of in-stream habitat for trout spawning, native fish and invertebrates. Project E relates to the Papanui stream rehabilitation associated with the Zone M water distribution system.

The five projects are described in Table 7.1.

Table 7.1Integrated mitigation and offset approach projects

Project	Description
Project A	Ruataniwha reservoir restoration buffer and catchment enhancement zone
Project B	Ruataniwha Riparian Enhancement Zone (River Halo Project)
Project C	Ruataniwha Threatened Species Habitat Enhancement
Project D	Ruataniwha Plains Spring Fed Stream Enhancement and Priority Sub-catchment Phosphorous Mitigation
Project E	Old Waipawa River Bed and Papanui Stream Restoration

8 Remediation of sediment loss to coast

T&T (May 2013) and HBRC (2013b) indicate that there will be a sediment loss to the coast on the Tukituki River as an effect of the proposed dam. The loss is estimated at approximately 1,700 m^3/year. This estimate is considered a conservative estimate i.e. the actual loss is likely to be less.

The coastal sediment transport is predominantly to the north, although there may be times when sediment from the Tukituki River is deposited to the south of the river mouth. To mitigate this effect it is proposed that 1700 m^3/year of river sediment will be placed directly along the barrier beach between Richmond Road and School Road extension and an additional 1,700 m^3/year to the south along the spit within the Coastal Marine Area (i.e. a total of 3,400 m^3/year). The provision of 3,400 m^3/year split north and south is considered to be a conservative approach to mitigate for the estimated reduction of sediment transport due to the proposed dam, being double the modelled shortfall.

The sediment would be extracted from upstream of Black Bridge or other locations in the Tukituki/Waipawa Rivers as determined by the HBRC annual gravel allocation process. The extraction of sediment would be in accordance with HBRC's river management practices (described in the Sedimentation Assessment, Tonkin &Taylor (May 2013). The sediment extracted from upstream of Black Bridge would be transported by truck from Mill Road and then taken along both Haumoana Road on the southern bank of the Tukituki River and Lawn Road on the northern bank. This methodology is chosen to avoid the conservation area as defined in the HBRC Regional Coastal Environment Plan. Refer to Figure 8.1. This would result in approximately 280 return trips by truck and trailer vehicles (assuming 12 m^3 capacity). The beach nourishment will carried out around October/November, after the winter storms.

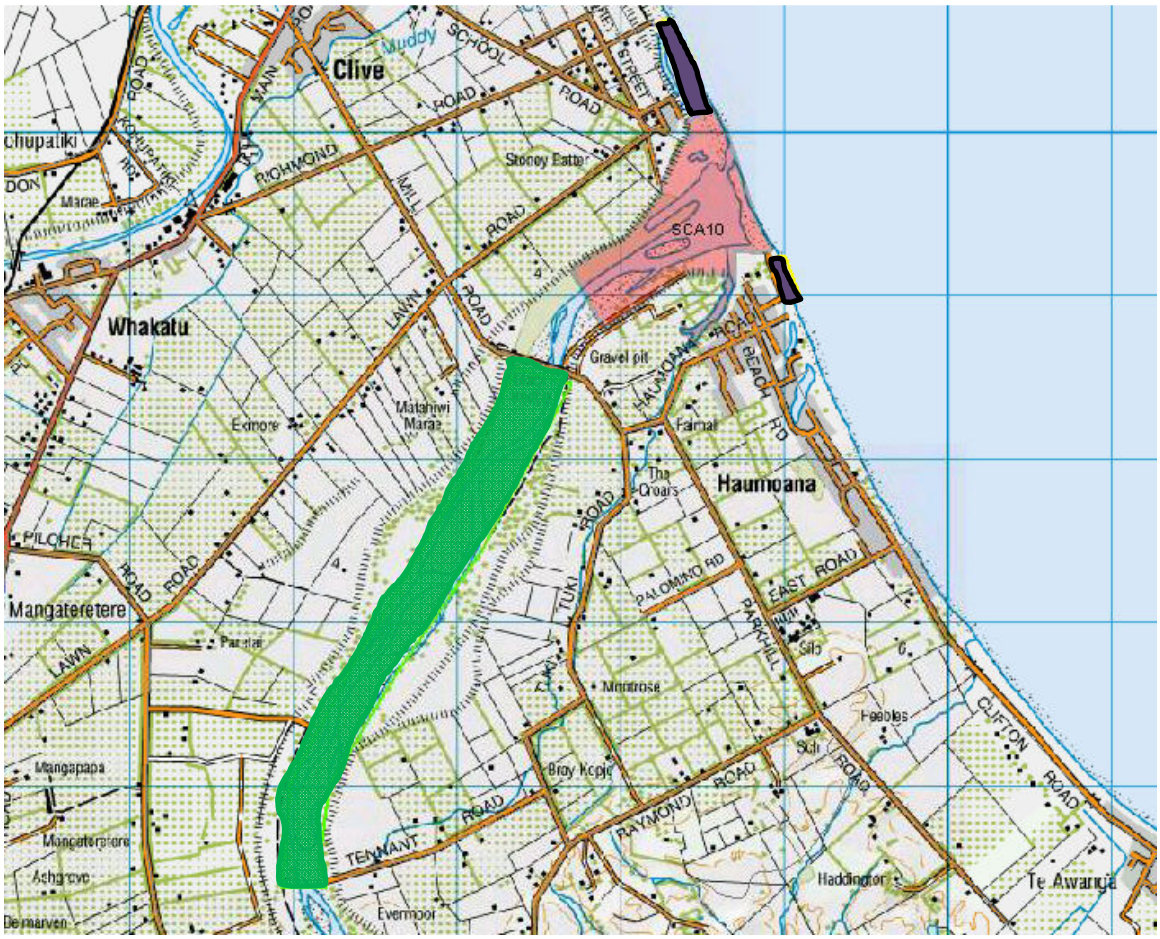


Figure 8.1 *Location of gravel extraction and beach nourishment. The proposed extraction area is depicted by the green box upstream of Black Bridge. A significant conservation area as defined in the HBRC Regional Coastal Environment Plan is shown shaded red. The black bordered purple zones are where the material will be deposited.*

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10 Applicability

This report has been prepared for the benefit of Hawke's Bay Regional Investment Company Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor by:
Mark Taylor	
David Leong	pp David Bouma
Tim Morris	Project Director
Mark Taylor	
Phil Carter	
Dewi Knappstein	
EMS Ltd	
Aquanet Consulting Ltd	
Golders Ltd	
HBRC	