RILEY CONSULTANTS

HWP WAITOHI IRRIGATION AND HYDRO SCHEME

ENGINEERING REPORT

Engineers and Geologists

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Report prepared for: Hurunui Water Project

Report prepared by: Paul Rivett, Water Resources Engineer

Paul Morgan, Director, CPEng

Andrei van Dusschoten, Geologist

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Report reviewed by: Don Tate, Director, CPEng

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	Riley Consultants Ltd	1 copy

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Riley Consultants Limited, Ground Floor, 395 Madras Street
PO Box 4355, Christchurch 8140, New Zealand
Telephone 64 3 379 4402, Facsimile 64 3 379 4403
rileychch@riley.co.nz www.riley.co.nz

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HWP WAITOHI IRRIGATION AND HYDRO SCHEME ENGINEERING REPORT

1.0 Introduction

Riley Consultants Ltd (RILEY) has been engaged by Hurunui Water Project Ltd (HWP) to undertake a study into the engineering aspects of the proposed irrigation scheme. The HWP proposal is to irrigate 58,500 ha of land in the Hurunui catchment (Amuri Basin, Hawarden, mid- and lower-Hurunui Riparian, Greta Valley, Omihi and Scargill Valley) and Waipara catchment (Upper Waipara, Glasnevin, Amberley and Kowai). A number of storage options have been proposed, and consents are presently with Environment Canterbury Regional Council (ECan) for potential storage provided by a control of Lake Sumner outflows, construction of a dam on the Hurunui South Branch, and a river intake near the Mandamus River confluence.

The present report considers an alternative for the storage and delivery of water to the aforementioned areas. This involves a pumped-storage option with water pumped from the Hurunui River downstream of Surveyors Stream and conveyed via buried pipe through a tunnel and into the Waitohi River catchment for storage. A series of dams are proposed to be constructed on the Waitohi River to develop the irrigation storage and hydro-generation to their full potential. The aim of the engineering report is to provide supporting information for a Resource Consent application for the project, of which, the Waitohi River dams and associated scheme intakes and conveyance structures are key components.

The purpose of this report is to describe the results at this stage of the study, concentrating on progress of key feasibility issues. The present work is considered at a pre-feasibility level of detail. The focus of present work is as follows:

- Geological mapping and interpretation of key geological information.
- Comparison of various dam types for all dam sites and related height versus storage relationships.
- Development of conceptual dam design.
- Consideration of hydrological issues, including flooding during construction and permanent spillway types and dimensions.
- Optimisation of hydro-generation potential in the Waitohi River system.
- Assessment of applicable design standards and criteria for the various dam structures.
- Identification of key areas for further investigation and/or assessment.

The report includes a summary of the hydrological analysis undertaken with a more detailed description in Appendix A.

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1.1 Supporting Documents

This report should be read in conjunction with:

- Pattle Delamore Partners Ltd, Assessment of Environmental Effects: HWP Waitohi Irrigation and Hydro Scheme. Stage 1.
- Pattle Delamore Partners Ltd, Assessment of Environmental Effects: HWP Waitohi Irrigation and Hydro Scheme. Stage 2.
- Pattle Delamore Partners Ltd, HWP Waitohi Irrigation and Hydro Scheme: Assessment of Sediment Transport Effects.

1.2 Survey Data

The levels referred to in this report are based on an aerial survey undertaken by Precision Aerial Surveys in August 2007. The co-ordinate system used for this survey is New Zealand Map Grid and the datum is the New Zealand Geodetic Datum 1949.

2.0 Scheme Description

The Hurunui Water Project proposes to irrigate 58,500 ha of land in the Hurunui catchment (Amuri Basin, Hawarden, mid- and lower-Hurunui Riparian, Greta Valley, Omihi and Scargill Valley) and Waipara catchment (Upper Waipara, Glasnevin, Amberley and Kowai). A combination of run-of-river and stored water is required to meet irrigation demand in these areas. Run-of-river water taken directly from the Hurunui River will be the predominant supply of water to the distribution network at times when it is available. When this water is not available, water stored in the Waitohi River catchment will be released to meet the irrigation demand. To implement this proposal, a number of river intakes and water storage structures are required. The location of the project area and scheme layout is presented in Appendix C on RILEY drawings 11841-01 and 11841-02 respectively. This section describes the proposed layout and construction staging of the scheme.

2.1 Proposed Scheme Layout and Development Staging

A two stage development of the proposal is considered the most effective way to implement the scheme. The first stage provides irrigation water to 6,900 ha net irrigation area at an early stage of the development before construction of the main infrastructure is completed and in the second stage full development of the scheme is completed.

2.1.1 Stage 1: Initial Scheme Development

The proposed new intakes and storage dams for Stage 1 are presented in Tables 1 and 2 respectively and their locations presented on RILEY drawing 11841-10. This stage will include the construction of Seven Hills and Inches Road dams to store water from the Waitohi catchment for development of an initial 6,900 ha of irrigable land. Also constructed at this time will be the Hurunui River intake at Mandamus and the Waitohi River intake downstream of the Inches Road dam. Both dams will be operated to meet Stage 1 irrigation demand and may include full drawdown to specified levels depending on irrigation demand and natural inflows.

Table 1: Proposed Stage 1 Intakes

Intake	Intake No. (1)	Water RL (m)	Max. Flow (m^3/s)	Comment
Hurunui River (at Mandamus)	1	280.0 to 305.0	3.9	Required to supply first-stage water.
Waitohi River	2	280.0 to 305.0	3.9	Required to supply first-stage water.

Note: (1) Refer to RILEY DWG 11841-10 for locations.

Table 2: Proposed Stage 1 Water Storage Dams

Dam	Crest RL (m)	Water RL (m)	Dam Height (m)	Crest Length (m)	Live Storage Volume (MCM)	Peak Irrigation Outflow (m^3/s)
Seven Hills	405	400	46	150	7.3	3.9
Inches Road	375	370	31	245	3.9	3.9

2.1.2 Stage 2: Full Scheme Development

The proposed new intakes and storage dams for Stage 2 are presented in Tables 3 and 4 respectively and their locations presented on RILEY drawing 11841-11. This stage will see the construction of all aspects of the scheme, including the pump station on the Hurunui River downstream of Surveyors Stream and the main storage reservoir at Hurricane Gully. In addition, hydro-generation equipment will be installed to enable electricity generation from the releases from all reservoirs.

Table 3: Proposed Stage 2 Intakes

Intake	Intake No. (1)	Water RL (m)	Max. Flow (m^3/s)	Comment
Hurunui River (at Mandamus)	1	280.0 or 305.0	26.1	To supply run-of-river water to the distribution system.
Waitohi River	2	n/a	n/a	The Stage 1 intake at this location will be decommissioned.
Hurunui River (at pump station)	3	383.5	17	To supply pumped water into the Waitohi catchment for irrigation storage and hydro-generation.
Hurunui River (existing intake Amuri Scheme)	4	unknown	8.5	To supply run-of-river water to the existing Balmoral Scheme and Balmoral Forest.
Waitohi River (at Lower Gorge 1 Dam)	5	325.0 & 340.0	42.4	To supply the main distribution canal and other areas from Waitohi storage.

Note: (1) Refer to RILEY DWG 11841-11 for locations.

Table 4: Proposed Stage 2 Water Storage Dams

Dam	Crest RL (m)	Water RL (m)	Dam Height (m)	Crest Length (m)	Live Storage Volume (MCM)	Peak Inflow(1) (m^3/s)	Peak Generation Outflow (m^3/s)(3)(4)	Peak Irrigation Outflow (m^3/s)(3)
Hurricane Gully	505	500	105	429	209.5	17(2)	35	42
Seven Hills	405	400	46	150	7.3	42	8	42
Inches Road	375	370	31	245	3.9	42	8	42
Lower Gorge 1	345	340	21	71	0.4	42	8	42

Notes: (1) From irrigation releases only. Flood flows are considered in Section 7.0. (2) Not including Waitohi natural inflows. (3) The difference between peak generation and irrigation flows is to maximise generation efficiency. During peak irrigation flows, generation will be at peak and the difference will be released via a bypass. (4) This is currently as estimated value and it may change, but it will not exceed peak irrigation outflow.

2.2 Intakes

For the full implementation of the scheme four separate intakes are proposed, with three intakes on the Hurunui River and one intake on the Waitohi River. The following comments are a brief description of the location of each intake with full descriptions provided in Section 9.0.

Hurunui River at Mandamus (Intake 1): The key run-of-river intake that directly supplies the main distribution canal. There are two potential options for the location of this intake. The first option is to be located on the true-right bank of the Hurunui River immediately downstream of the Mandamus confluence and upstream of the existing Amuri scheme intake. This intake is included in the Resource Consent application for the Hurunui South Branch and Lake Sumner options. The second option is for the intake to be located on the true-right bank approximately 5 km downstream of the Amuri scheme intake.

Waitohi River (Intake 2): During Stage 1 a small weir will be located on the Waitohi River upstream of the distribution canal allowing water to be diverted from storage and supplied to the Stage 1 irrigable areas. This structure will be decommissioned upon completion of Stage 2 and replaced by Intake 5.

Hurunui River at pump station (Intake 3): This intake diverts water via pump station, rising main and tunnel into the Waitohi River catchment. This intake is located on the true-right bank of the Hurunui River about 1.5 km downstream of the confluence with Surveyors Stream. The river turns 90 degrees at this point and is considered to be the most favourable location for an intake on this stretch of the river.

Hurunui River at Amuri scheme intake (Intake 4): This is an existing intake that will have its capacity increased for additional water that will be delivered to the Amuri scheme and areas to the north of the Hurunui River. This will be included in a separate consent application and not considered in any detail in this report.

Waitohi River at Lower Gorge 1 Dam (Intake 5): Intakes on the Lower Gorge 1 Dam will be constructed during Stage 2. A high-level intake will divert water to gravity-feed irrigable land located above the distribution canal and a low-level intake will divert water directly to the main distribution canal.

2.3 Dams

Four separate dams and reservoirs are proposed on the Waitohi River. The dams are located as shown on drawing 11841-02 utilising the favourable natural topography of each site. A number of dam siting options have been explored in a number of locations along the Waitohi River. The sites discussed herein have been assessed as the most efficient to meet the requirements of the staged approached discussed above, and have been located to minimise the impact on productive farmland. The following comments are a brief description of the location of each dam with full descriptions provided in Section 10.

Hurricane Gully Dam: The largest storage dam on the scheme providing the majority of the scheme reservoir volume. This dam is located immediately downstream of the confluence of the Waitohi River and Hurricane Gully. Based on the dam siting options undertaken to date, this location provides the most cost-effective site for the main water storage structure within the Waitohi catchment in terms of the volume of water stored relative to the volume of dam fill required. This dam is proposed to be constructed during Stage 2 and has been optimised for irrigation requirements, but has the ability to be used as a peaking station for hydro-generation development.

Seven Hills Dam: This dam will be the first dam constructed during Stage 1. During Stage 1, it will be operated solely for irrigation requirements to maximise the irrigable command area. It will be filled with in-catchment water from the Waitohi River and release will be controlled in conjunction with Inches Road Dam downstream. During Stage 2, this dam will be operated as a re-regulation pond to attenuate hydro-generation flows from Hurricane Gully Dam, i.e. water will be released from the Hurricane Gully Dam to maximise hydro-generation capacity, and the Seven Hill reservoir can store this peak flow and discharge a lower flow to meet irrigation demand. In addition, it can still play a role of providing irrigation storage during extreme dry years.

Inches Road Dam: This dam will be constructed during Stage 1 and after Seven Hills Dam. It will be operated in the same manner as the Seven Hills Dam during Stage 1, but during Stage 2 will be operated for irrigation demand, i.e. will not play a role in attenuating hydro-generation outflows from the upstream dams. As with the Seven Hills reservoir, it will provide irrigation storage during extreme dry years.

Lower Gorge 1 Dam: This dam will be the final water retaining structure constructed during Stage 2 and is the main structure for diverting stored water into the distribution system. The reservoir created by this dam will not provide stored water for irrigation.

3.0 Scope of Investigations to Date

3.1.1 Geological

A preliminary phase of engineering geological mapping was undertaken between 15 and 17 June 2011 (Hurricane Gully dam and reservoir), and on 11 August 2011 (lower Waitohi River dam sites). This mapping focused on developing the ground model for the four dam sites and the lower and middle reaches of the Hurricane Gully reservoir. Mapping has yet to be undertaken in the upper reaches of the Hurricane Gully reservoir area, or for the reservoirs associated with the three lower dam sites. This mapping focussed on highlighting:

- Geological constraints for the proposed development.
- Levels of geological uncertainty in the ground model.
- Further investigation stages necessary to reduce key areas of uncertainty.

Specific desktop studies have been undertaken for these areas and tasks completed to date include assessment of:

- 1:50,000-scale stereo-pairs of aerial photographs of the site from 1986.
- 1:250,000-scale geological map of the Kaikoura Area (Institute of Geological and Nuclear Sciences, Map 13, 2006).
- Fault data from the GNS Science fault database (interactive internet resource).
- Selected publications documenting tectonic setting, bedrock, rock avalanches, and glacial history.
- Previous reports related to the proposed development.

The conclusions drawn in this report are based on broad-scale qualitative desktop and field studies appropriate for the pre-feasibility level work. Field mapping to date for the three smaller dam sites has involved sketching geomorphological features from one or two vantage points, and quick inspections of one or two convenient outcrops at/near dam footprints. The 1:50,000 scale photos that were procured before field mapping allowed for assessment of large-scale features such as deep-seated bedrock landslides, but not of small to moderate-sized features such as river terraces and shallow slides. Additional desktop and fieldwork will be required to prove the feasibility of the various scheme components as discussed in Section 12.

3.1.2 Engineering

The engineering work that has been undertaken to date includes the following:

Design Standards and Criteria

- Consideration of applicable design criteria and standards to be used. This has included consideration of the Potential Impact Classification (PIC) in terms of the New Zealand Dam Safety Guidelines (NZSOLD 2000) and the new dam safety regulations as outlined in the Building (Dam Safety) Regulations (2008) and the Building (Dam Safety) Amendment Regulations 2010.
- Consideration of fish screening requirements at all intakes as outlined in the "Fish screening: good practice guidelines for Canterbury" publication prepared by NIWA in 2007.

Alternative Dam Location Options and Types

A 3-D model of the Waitohi River was used to assess different dam locations along the river. Based on geometry the Hurricane Gully site was found to be the best site along the river for a large storage in regard to minimising the volume of the dam structure. Many different sites downstream of Hurricane Gully were assessed in terms of providing additional storage, acting as a reregulation pond, utilising the fall down the valley for hydro generation and providing options for staging construction. A number of alternative options for Waitohi River storage have been assessed. The scheme arrangement and scheme staging as outlined in Section 2 is considered to be the most efficient for meeting the various technical, irrigation, hydro-generation, and economic demands of the project. Table 5 lists the main alternatives that were considered for development of the scheme.

Table 5: Alternative Options for Waitohi River Storage

Alternative	Dams[(1)]	Storage Volume (MCM)	Comment
1	HG, SH	90, 1	The minimum storage option considered for the scheme. SH dam only for hydro-generation re-regulation with no additional irrigation storage. No staging for this Alternative.
2	HG, SH	90, 7	As per Alternative 1, but with additional irrigation storage in SH reservoir. No staging for this Alternative.
3	HG, SH	140, 1	Medium storage considered for the scheme. SH dam only for hydro-generation re-regulation with no additional irrigation storage. No staging for this Alternative.
4	HG, SH	140, 7	As per Alternative 3, but with additional irrigation storage in SH reservoir. No staging for this Alternative.
5	HG, SH	210, 1	The maximum storage considered for the scheme. SH dam only for hydro-generation re-regulation with no additional irrigation storage. No staging for this Alternative.
6	HG, SH	210, 7	As per Alternative 5, but with additional irrigation storage in SH reservoir. No staging for this Alternative.
7	HG, SH, IR, LG1	90, 7, 4, 0.4	Development of the full scheme allowing staging and with minimum storage volume at HG.
8	HG, SH, IR, LG1	140, 7, 4, 0.4	Development of the full scheme allowing staging with medium storage volume at HG.
9	HG, SH, IR, LG1	210, 7, 4, 0.4	Development of the full scheme allowing staging with maximum storage volume at HG.
10	Lower Gorge[(2)]	140	Development of a single large reservoir in the Lower Gorge area of the Waitohi River.

Note: (1) HG – Hurricane Gully; SH – Seven Hills; IR – Inches Road; LG1 – Lower Gorge 1. (2) This dam option is in the general vicinity of the Lower Gorge 1 dam site and approximately 90 m high.

Alternatives 1 to 9 are all based on Hurricane Gully being the main storage component of the scheme, with the lower dams' inclusion to enable staging. Alternative 10 is a large reservoir with a live storage volume of 140 MCM in the general vicinity of the Lower Gorge 1 dam. This site was discounted as it resulted in significantly reduced storage-to-dam fill efficiencies and an overall higher cost for water storage compared with Hurricane Gully. In addition, areas of productive farmland would require inundation.

Alternative Dam Types

- Comparison of dam types and, in particular, the pros/cons of each option and related feasibility issues.

Intake Locations and Types

- Alternative intake locations have been considered for all proposed intakes and the key issues and constraints for each location.
- Comparison of intake types and their suitability for each location and their fish exclusion ability.
- Sediment management issues were considered with regard to maintaining sediment within the river.

Flood Hydrology and Spillways

- Derivation of various design flood flows and hydrographs up to the Probable Maximum Flood (PMF) level for the Waitohi River, with PMF calculation at a preliminary level of detail only.
- Preliminary identification of type and sizing of diversion components for several options, and the permanent service and emergency spillways.

4.0 Regional Geological and Geomorphic Setting

This section summarises the regional geology and geomorphology of the scheme area. drawing 11841-100 shows the project area on the published regional geological map and extract of relevant sections of the legend to accompany the map.

4.1 Stratigraphic Setting

The published geological map for the area (Geology of the Kaikoura Area, Institute of Geological and Nuclear Sciences, 1:250,000 Geological Map 13, 2006) indicates that bedrock in project area comprises Esk Head Belt and Pahau terrane rocks. The Hurricane Gully dam area is underlain by Esk Head Belt rocks, while the lesser three dam sites are underlain by Pahau terrane rocks. The Esk Head Belt forms a 20 km-wide north-trending fault-bound 'suture' between the Rakaia (older - western) and Pahau (younger – eastern) terranes, which together form the Torlesse composite terrane. Following juxtaposition of these three rock-bodies, the region has been further disrupted by recent strike-slip faulting on structures within the Marlborough Fault System.

The Rakaia and Pahau terranes comprise predominantly sedimentary rocks which are typically quartzofeldspathic, indurated, variably bedded, sandstone and siltstone, and less common limestone, chert, and volcanics. The Esk Head Belt includes these rock types, albeit more-highly-fractured (locally including 'melange').

Overlying bedrock on the Waitohi valley floor and lower slopes are thin discontinuous layers of alluvial sediment, predominantly comprising sandy gravel. The Waitohi River and its tributaries have incised into bedrock, forming a series of gorges on the site, mostly less than 10 m deep but locally up to 40 m (Inches Road). The few alluvial deposits that do occur have very limited areal extent and are generally less than 5 m thick. Alluvium only becomes widespread where the river exits into the Culverden basin. Colluvial cover, however, forms a blanket over much of the project area, though this is usually less than 3 m thick.

Notably, no Tertiary-aged or glacial sediments have been mapped in the project area. The closest Tertiary-aged sediments are approximately 1 km northeast of the Lower Gorge 1 dam site, while the closest glacial sediments occur on the east side of Lake Taylor 20 km northeast of the project site.

4.2 Tectonic Setting

The project area is located in a tectonically very active zone on the east side of the Alpine Fault which forms the convergent boundary between the Australian (west) and Pacific (east) tectonic plates. In the upper South Island, a combination of compression, shear and local tension associated with this plate boundary has resulted in formation of mountainous terrain and sedimentary basins (e.g. Hanmer basin). Strain in this area has primarily been taken up along a number of southwest to northeast trending faults which form the Marlborough Fault System. The southern-most faults of this system are located less than 40 km north of the project area and include the following faults.

- *Hope Fault*, located 30 km to the north of the project area, is a strike-slip structure which last ruptured along a 30 km long segment in the Hope River in the 1888 M7.0-7.3 North Canterbury Earthquake. This segment of the fault has an estimated average event-re-occurrence interval of 120 years for a similar magnitude event.

- *Conway Fault* (eastern) extension of the Hope Fault, located 40 km northeast of the project area, ruptured around 170 years ago, and is estimated to produce an M7.6 event every 120-300 years.
- *Kakapo Fault*, located 25 km north of the project area is a major splinter of the Hope Fault, and is expected to result in a M7.3 event approximately every 500 years, with the last event likely to have occurred within the last 10,000 years.

The Waitohi River catchment area is flanked by two active northeast-southwest trending faults:

- *The Esk Fault*, located 2-10 km northwest of the project area; projected recurrence intervals of 5,000-10,000 years reverse fault (down-thrown on the southeast side); last event unknown.
- *Waitohi Downs Fault*, located 1-5 km southeast of the project area; characteristics same as Esk Fault.

Both of these faults are of similar length to the Conway and Kakapo Faults and also to the Greendale Fault which ruptured in the 4 September 2010 M7.0 Darfield Earthquake. It is reasonable to assume that the Esk Head and Waitohi Downs faults are capable of producing similar magnitude events to those experienced by these faults.

The alignment of the middle section of the Waitohi valley parallels the Esk and Waitohi Downs faults, which hints at the possibility of a fault along the valley. No evidence for a fault (asymmetric slope morphology, offset terraces, active springs) was noted during our site investigation of 15-17 June 2011, nor are any active faults shown in the most recent literature. Investigations will be undertaken during the feasibility stage to assess the presence of faulting at this location.

4.3 Geomorphology

The general geomorphology of the project area reflects influences of the underlying geology, the current compressional tectonic regime (mountain building and erosion), and locally, from glaciation.

Key geomorphologic features of the project area are:

- Predominantly moderately steep to gentle hill-slopes (most/all are farmed) interrupted by small bluffs and “knobbly-knocker” bedrock outcrops. Ridgelines/spurs are often broken by a series of peaks and saddles indicating changes in bedrock geology (thick sandstone/siltstone beds) or possibly past instability.
- In the lower reaches the Waitohi River and its tributaries, these watercourses have cut gorges into bedrock; upper reaches are marshy with meandering watercourses.
- A series of small (i.e. 2 m to 5 m high) narrow (typically less than 15 m wide) terraces lines both sides of the Waitohi River. These are evidently underlain by bedrock capped by a thin (less than 3 m thick) discontinuous blanket of alluvium lapping with colluvium at the toe of adjoining slopes. Terrace deposits and surfaces are often obscured by colluvium and alluvial fans
- Several knobbly landforms (unusual landforms or “ULF”’s) occur on the Waitohi valley floor and in the major tributary. These are possibly ice-sculptured features (roche moutonnée’s) cored by resistant (sandstone) bedrock.

- The right-angle bend in the Waitohi River immediately upstream of the proposed Hurricane Gully dam location requires further investigation to examine why the river has made this sharp change in route. Possible explanations include presence of a fault, or juxtaposition of bodies of bedrock with differing bedding/defect orientations.

4.4 Geological Materials

The main materials present in the Waitohi valley area are colluvium, alluvium and Esk Head Belt and Pahau terrane bedrock units. Alluvial gravels are notably sparse, while Tertiary-aged strata and glacial deposits are apparently absent.

4.4.1 Colluvium

Colluvium is gravity-transported sediment of variable composition (silt to boulders), and is typically poorly consolidated. It includes scree and landslide deposits. On the project site colluvium blankets hill-slopes and the toe of terraces.

4.4.2 Alluvium

Alluvium is water-transported (river) sediment, typically comprising sand and gravel, ranging from poorly-consolidated (fan alluvium) to well-consolidated (terrace alluvium), and is typically highly permeable. It includes terrace and fan deposits. Openwork lenses (i.e. lacking silt and clay) are likely to be present within terrace alluvium, though their extent and persistence and influence on permeability of the terraces is unknown.

4.4.3 Esk Head Belt Bedrock

On the published geological map, bedrock in the Hurricane Gully and Seven Hills area is mapped as Esk Head Belt rock. Although Esk Head Belt includes many rock types, the areas proposed for these dams consist largely of interbedded siltstone and sandstone that has been stretched and deformed before solidifying (resulting in “broken formation” fabric).

The siltstone has a fabric which tends to cleave (break in to elongate shards), typically breaking sub-parallel to apparent bedding. However, sandstone typically has a blocky rock mass as a result of intersecting irregularly-orientated closely to moderately-widely spaced joints. While the material strength of siltstone is typically only slightly lower than sandstone, the pervasive siltstone fabric results in a relatively poor rock-mass quality, compared to sandstone which typically displays a moderate to high (large-blocky) rock-mass quality.

Bedding is typically steeply-dipping. The published geology map indicates a regional northeast-southwest dip-direction for bedding, but on-the-ground mapping revealed highly variable dip-directions across short distances indicating folding and/or faulting

4.4.4 Pahau Terrane Bedrock

Bedrock in the Inches Road and Lower Gorge 1 areas consist of Pahau Terrane rocks. This rock is Esk Head belt bedrock that has not been subject to as much deformation and stretching. Within the project area Pahau terrane rocks mostly comprise interbedded siltstone and sandstone, which is typically steeply dipping.

5.0 Geology of Dam Sites and Reservoirs

In this section, the Hurricane Gully site is described at a greater level of detail than the lower three dam sites. From the assessment that has been completed to date, we consider it appropriate to apply the geological model developed for the Hurricane Gully site to the lower three dam sites. The wider issues discussed for the Hurricane Gully area in Section 5.1 can generally be applied to these three sites. For these sites, discussion is limited to considerations that differ from those discussed for the Hurricane Gully site. The drawings relevant to the engineering geology of the dam sites are listed below and presented in Appendix C:

Hurricane Gully	11841-110 Overview Site Plan 11841-111 Engineering Geology Plan 11981-112 Dam Cross Section
Seven Hills	11841-120 Overview Site Plan 11841-121 Engineering Geology Plan 11981-122 Dam Cross Section
Inches Road	11841-130 Overview Site Plan 11841-131 Engineering Geology Plan 11981-132 Dam Cross Section
Lower Gorge 1	11841-140 Overview Site Plan 11841-141 Engineering Geology Plan 11981-142 Dam Cross Section

5.1 Hurricane Gully

5.1.1 Dam

The geological model for the proposed Hurricane Gully dam area consists of a thin discontinuous blanket of alluvium and colluvium overlying fractured bedrock (illustrated on cross section, drawing 11841-112). This is evident from the numerous widely-distributed bedrock exposures along river banks as well as on both abutment slopes.

Key geological considerations for the Hurricane Gully dam site include:

- Soil (alluvium and colluvium) deposits are thin and discontinuous. The depth to acceptable bedrock founding material is expected to be generally less than the order of three metres both in the river bed and on the abutments.
- There is significant variability in bedrock lithology and mechanical properties. This will require detailed consideration during subsequent investigations and design stages to ensure adequate dam performance and safety.
- Bedrock is typically steeply to sub-vertically dipping; at river-level bedding strikes roughly sub-parallel with the river, thus increasing the potential for seepage through the foundation. Away from the river, bedding strike is highly variable. Our assessment to date has revealed high variability in defect orientation, persistence, spacing etc.

- A fresh scar and angular debris near dam centreline above the true right bank of the river indicates a recent shallow bedrock failure has occurred, apparently at the intersection of two joints (wedge failure). Some large blocks in the bed load indicate similar such events occurred in recent past. There was also evidence of active erosion of over-steepened ground on the opposite bank.
- Any stilling basin founded in siltstone will require erosion protection due to susceptibility to slaking.

5.1.2 Reservoir

The geological model for the proposed Hurricane Gully reservoir area consists of a thin discontinuous blanket of alluvium and colluvium overlying bedrock.

Key geological considerations for the Hurricane Gully reservoir area include:

5.1.2.1 Seepage Potential

- A potential seepage path was identified immediately upstream (south) of the right abutment area where a patch of lush green grass is visible on both sides of the saddle here. This may indicate presence of higher-permeability bedrock (siltstone) or a fault. An unfavourable combination of slope geometry and bedrock bedding orientation here could facilitate seepage from the reservoir. Similar features may occur further upstream where highly variable bedding and defect orientations were observed. This seepage potential will be assessed in detail during the feasibility stage.
- There is potential for seepage across the low saddle between the major southern “arm” and the main body of the proposed lake.

5.1.2.2 Bedrock Stability

- Several large-scale bedrock instability features were identified from aerial photos of the Hurricane Gully area. These ancient failures may have occurred due to relaxation of the valley slopes following glacial retreat.
- Mapping undertaken to date has revealed several over-steepened slopes and small bluffs on the true left of Hurricane Gully, particularly at and around the approximate inundation level (RL 500 m). These are underlain by bedrock and are likely to have formed by ice-scouring during the last glaciation. There was no evidence of instability relating to these features. Bedrock failures or movements are typically triggered by seismic events or high intensity rainfall events. The lack of apparent instability indicates bedrock in the reservoir area has performed reasonably well given that it will have been subjected to a number of these events.
- Given their apparent stability of bedrock following glacial recession, the potential for failure of these slopes is considered to be low. However, as only limited assessment has been undertaken to date, and the potential consequences of a large-scale failure event is substantial, consideration should be given to further investigation of this hazard.

- As seen in the dam area, there is a potential for localised and small-volume rock-fall and failures (planar/wedge/toppling) of bedrock outcrops into the reservoir. Such events are likely within a 50 year design life of the proposed reservoir, but are not expected to result in significant effects outside the reservoir.

5.1.2.3 Soil Stability

- Our assessment revealed some evidence of past instability of soil deposits underlying slopes in the reservoir area, such as faint head scarps and hummocky ground. Similar failure events are anticipated on slopes within the live range of the proposed reservoir, however these are likely to be of limited thickness and extent and involve relatively small volumes.
- Steep, and poorly consolidated scree and alluvial fan slopes are likely to be susceptible to instability if fully or partially saturated by the reservoir. Scree is poorly compacted and typically at the critical angle of repose. Again, such failure events are expected to involve relatively small volumes.
- Edges of terraces capped with alluvium are considered to have a low susceptibility to instability because they are relatively free-draining and of limited thickness and extent.
- Further investigation will be required to quantify the potential for the above events to occur. If the potential is high, for such events, and the consequences serious, susceptible slopes may require flattening or buttressing.

5.2 Seven Hills

The Seven Hills dam site was viewed from the true-right abutment above the design dam crest level, and upstream of the dam site on a terrace on the true-right. The reservoir assessment was limited to a desktop study.

5.2.1 Dam

Geological considerations for the Seven Hills dam site include:

- Numerous bedrock outcrops in the dam footprint area indicate that alluvial and colluvial soil deposits are generally thin and discontinuous (drawing 11841-121)
- At the dam location the river has incised into bedrock forming a 10 m deep gorge with moderately steep-sided embankments (steepest at river level). A few hundred metres downstream, the gorge opens up and gives way to prominent terrace on the true right. The slope angle of the true right embankment becomes less steep between the dam site and these terraces indicating presence of thicker soil deposits.
- Above the gorge on the true right abutment, a small 15 m wide terrace occurs, above which, a uniform slope with 25-30° gradient extends above the proposed crest level.
- The lower true left abutment is divided by a prominent rock-spine ridge composed of sub-vertical sandstone beds divides. The crest level slope gradient of 30° steepens to 45° toward the river (i.e. convex). The slope upstream of the centreline has a hummocky appearance, though this was obscured by vegetation.
- Evidence of a shallow (colluvium) slide was observed on the true right abutment area upstream of the proposed centreline where a small head scarp occurs with hummocky ground uphill and downhill of the scarp. The failure event is likely to be related to past flooding of an ephemeral stream (dry) located up-slope of the feature.

- There was no obvious evidence of faults at/near the dam area.

5.2.2 Reservoir

- Aerial photographs indicate that prominent terraces occur upstream of the dam site on the true-right.
- A small waterfall occurs 1 km upstream of dam site (500 m upstream of the sharp bend in the river). A waterfall feature often indicates the presence of a fault, and further examination of this will be undertaken during the feasibility stage.

5.3 Inches Road

5.3.1 Dam

The Inches Road dam site was viewed from a slope and ridge on the true-right side of the river 250 m upstream from the dam centreline, and from the edge of a 30 m+ deep gorge located on the true-left side of the river 30 m downstream of the centreline. The reservoir assessment was limited to a desktop study.

Geological considerations for the Inches Road dam site include:

- A significant gorge occurs at the Inches Road dam centreline area, with the left abutment forming a 30 m+ high sub-vertical cliff; the right abutment is less steep at the dam centreline area (gradient roughly 45°), but 30 m upstream, it steepens to also form a sub-vertical cliff 30 m+ high.
- The gorge is flanked on either side by terraces which are roughly at the same elevation, and are presumably underlain by bedrock capped with alluvium. The true-left terrace is 100 m wide and 500 m long, while the true right terrace is much smaller at 20 m wide and 100 m long. Approximately 70 m upstream of the dam centreline, the true-right terrace ends abruptly where a sub-vertical cliff of sandstone drops 30 m to a small terrace situated just above river level. This terrace is likely to be underlain by recent alluvium (Q1a on published map)
- The published geological map indicates that the dam area is underlain by Pahau Terrane bedrock. As discussed earlier, rocks of the Pahau Terrane are typically less fractured than Esk Head Belt rocks. The published map indicates the boundary between these two rock bodies is approximately 1 km south of the dam, however, the actual location of this boundary is known to be poorly constrained (i.e. not mapped with confidence). Bedrock bedding and defect orientations have not been examined.
- The published map also indicates presence of young alluvial sediments (Q1a and Q2a) underlying terraces along the river both upstream and downstream of dam area. While numerous terraces do occur in this area, they appear to have been cut into bedrock, and alluvium is largely absent, or forms a thin discontinuous layer (such as seen upstream of the dam location at a road cut upstream of the bridge across the river). Numerous bedrock outcrops indicate a minimal soil thickness and extent in the dam footprint area.
- The lower reaches of the tributary streams immediately upstream of the dam will be flooded - Jacks Stream on the true left and un-named on the true right. The potential for seepage here requires assessment, though the risk would appear to be low.

- There is no obvious evidence of faulting at or near the dam area. Active faults present in the greater area are the same as discussed above for the Hurricane Gully dam. Of note, the Waitohi Downs Fault trace is mapped 2 km southwest of the dam area (two strands).

5.3.2 Reservoir

A preliminary assessment of the available aerial photography indicated no obvious geological constraints for the Inches Road reservoir.

5.4 Lower Gorge 1

The Lower Gorge 1 dam site was viewed from Lake Sumner Road on the true-left side of the gorge, approximately at the dam centreline. The reservoir assessment was limited to a desktop study.

5.4.1 Dam

Geological considerations for the Lower Gorge 1 dam site include:

- At the dam site the river has incised into bedrock to form an open gorge with moderately steep-sided abutment. The true right abutment is steeply sloping, and appears to steepen at the base. Bedrock is exposed in several places. The true left abutment is less steep and covered in thick vegetation (possibly obscuring hummocky ground). The true left slope extends at a similar gradient above Lake Sumner Road to well above the proposed crest level (i.e. no terrace on this side).
- The river course around the dam area is distinctively linear compared to the three upstream dam sites. This location appears to be a transitional zone for depositional/erosional processes in the valley, with degradation (erosion, gorge forming; meandering river) dominating upstream, and aggradation (deposition; broad terraces; linear and/or braided rivers) dominating downstream.
- Above the true right abutment a 100 m-wide terrace planted with crops occurs. As observed for terraces elsewhere on the site, this terrace appears to be underlain by bedrock thin cap of alluvium. This terrace is not expected to be flooded as the proposed dam crest-level will confine it within the gorge.
- The published geological map indicates the dam area is underlain by Pahau Terrane bedrock, with local deposits of young alluvial sediments (Q1a and Q2a) occurring along the valley floor upstream and downstream of the dam. As observed at the other three dam sites, no significant alluvial deposits (>3m thick) were observed in the dam area. However, 250 m downstream of the dam site, the gorge opens to river-plains which are likely to be underlain by relatively thick and continuous alluvial deposits.
- There is no obvious evidence for faulting at/near the Lower Gorge 1 dam area. Of note however, the Waitohi Downs Fault (described in section 4.2) is located 500 m southeast of the dam site (mapped as two separate strands).

5.4.2 Reservoir

A preliminary assessment of the available aerial photography indicated no obvious geological constraints for the Lower Gorge 1 reservoir.

6.0 Potential Impact Classification Rating and Design Standards

6.1 Design Standards and Criteria

The New Zealand Dam Safety Guidelines produced by the New Zealand Society on Large Dams (NZSOLD 2000) outline typical design criteria based on the Potential Impact Classification (PIC) of the dam. The classifications are related to the potential consequences of a dam breach, primarily related to potential for loss of life, and financial, social and environmental impacts.

In July 2008, new Building (Dam Safety) Regulations were promulgated as part of the Building Act 2004. These regulations define three dam classifications (High, Medium, Low) based on the consequences of dam failure. The main factors in the classification include:

- Population at risk (PAR).
- Potential damage to residential houses, critical infrastructure and time to restore to operation.
- Effects on natural environment and community recovery time.

This methodology is slightly different to that currently used in the NZSOLD Guidelines, although the Guidelines are being updated to be consistent with the new regulations. The Building (Dam Safety) Amendment Regulations 2010 set the timeframe for their implementation as July 2012. Although these regulations are not yet implemented, the method outlined therein is considered the appropriate method to use for this project.

The proposed cascade of dams on the Waitohi River is in a remote area. While there are a small scattering of farmhouses around the Inches Road dam and reservoir, there are no low-lying townships or major settlements on the Waitohi River or the Hurunui River downstream of the confluence. Thus, the PAR is likely to be low pending the results of a detailed dam break analysis.

However, for the Hurricane Gully dam, due to the large size of the dam and, in particular, the large size of the retained reservoir (i.e. 100 m height and in the order of 227 MCM stored volume), environmental damage in the advent of dam breach is assessed as at least major or possibly catastrophic. Thus, on this basis, the PIC is high irrespective of the PAR. A high rating would also be compatible with the importance of the asset to the owner. The preliminary dam break analysis (Section 6.2) appears to confirm the High PIC for the Hurricane Gully dam.

6.1.1 Maximum Design Flood (MDF)

The NZSOLD Guidelines recommend an MDF between a 1:10,000 Annual Exceedence Probability (AEP) event and Probable Maximum Flood (PMF) for a high PIC dam. It is possible incremental consequences may be at the low end of the high range, justifying a MDF at the lower end of this range. At this stage, it is conservative to assume the PMF as the maximum design flood, particularly if the incremental cost of providing this capacity is not great.

6.1.2 Flood Diversion during Construction

No explicit guidance for flood diversion during construction is given in the Guidelines. The decision should be made on consequences (both in terms of downstream impacts and disruption to construction). The flood diversion standard is also dependent on the dam type; an earth dam or concrete face rockfill dam (CFRD) must have a significantly greater degree of protection compared to a roller compacted concrete (RCC) dam. This is because there is almost certain failure if an earth dam is overtopped, whereas an RCC dam should withstand even considerable overtopping without failure.

Tentative diversion flood standards for a high PIC dam could be:

Earth Dam/CFRD

- 1:10 to 1:20 AEP, where consequences are minor, e.g. where the dam height is less than 10 m to 20 m.
- 1:100 AEP or even greater where large volumes of water were retained at the later stages of construction.

The dam break analysis would be used to define failure consequences and allow further refinement of flood diversion capacity.

RCC Dam

- 1:5 or 1:10 AEP or even lower based on the resistance to overtopping during construction.

It is possible regulatory authorities will not allow lesser diversion standards for earth/CFRD dams than were used for the Opuha Dam reconstruction (Opuha is of lesser height and stored volume compared to the Waitohi Dam). The Opuha Dam used an original 1:10 AEP diversion standard, whereas the reconstructed dam after the breach used 1:100 to 1:300 AEP depending on the stage of construction.

6.2 Preliminary Dam Break Study

A dam break analysis has been undertaken for the Hurricane Gully Dam (full details are included in Appendix B). Dam break analysis are normally undertaken to assess the downstream hazard potential from the failure of a dam. The magnitude of the effects is used to set the safety standards in the design, construction and operation of the dam. The assessment starts from what is the worst case scenario in the event of a failure of the dam and then the basis of design is to ensure that this failure will not occur under any realistic scenario. The downstream impacts are used to determine the PIC. The PIC determines the standards and criteria for design, commissioning and operation of the dam which are set out in the NZSOLD Dam Safety Guidelines.

6.2.1 Hurricane Gully Dam Break Analysis

The most likely potential failure mode for the Hurricane Gully Dam is considered from either piping or overtopping. A piping and overtopping failure were used to assess the flow hydrograph downstream of the dam.

It is important that the distinction between hazard potential (the effects of the dam break) and risk of dam break actually occurring are clear. The risk of failure of a RCC or CFRD dam engineered to appropriate standards is extremely low.

6.2.2 Model

A preliminary dam break assessment for the Hurricane Gully dam has been undertaken to assess the areas downstream at risk of inundation. A simple hydraulic model of the Waitohi River and Hurunui River down to the coast has been constructed to assess the effects of a dam breach. Modelling software Hec-Ras has been used. A dam-break module within the software allows formation of a dam break outflow hydrograph based on input parameters.

The upper gorge section of the Waitohi River upstream of the plains has been modelled using cross sections derived from aerial survey information. For downstream section of channel for the Waitohi River and Hurunui River topomap data has been used (20m contours) along with observation of landscape features from Google Earth.

The extent of inundation has been mapped for the worst case scenario and shown on Drawing 11841-30. The drawing shows the inundation and houses within that area.

6.3 Maximum Design Earthquake (MDE) and Operating Basis Earthquake (OBE)

Industry practise as stated in the NZSOLD Guidelines is a design for two levels of earthquake. For a high PIC dam the NZSOLD guidelines recommend a maximum credible earthquake based on the strongest earthquake from a specific source or 1:10,000 AEP event if statistically derived.

The dam must withstand the MDE without an uncontrolled release of the reservoir. Repairable damage is acceptable. The OBE is selected on a probabilistic basis, with a recommended AEP of 1:150. For this event the performance criterion is that only minor damage is acceptable.

7.0 Flood Hydrology

An analysis of the flood hydrology for the Waitohi River catchment has been undertaken using available flow gauge data for the Waitohi River. Appendix A includes more detail of the hydrological assessment undertaken.

Pattle Delamore Partners (PDP) has undertaken a detailed assessment of the hydrology with regard to the irrigation demand and operation of storage. Therefore, only the flood hydrology as it relates to design flows for construction and spillway design is included in this report.

7.1 Flood Hydrology

7.1.1 Available Data

A permanent water level recorder was established at the Lake Sumner bridge that crosses the Waitohi River with recording commencing in December 2007. The site is referred to as Waitohi River at Lake Sumner Road (Site Number 65106), which is located immediately downstream of the Lower Gorge 1 dam site.

In addition, the long-term flow record held by Environment Canterbury (ECan) for the Waipara River at White Gorge site (Site Number 65109) has data available over the period February 1988 to March 2011. There is a strong correlation (R^2=92%) between these two sites.

7.1.2 Flood Frequency Analysis

Flood frequency analysis was undertaken within Hilltop Hydro on the longer term Waipara River flow series at White Gorge. This analysis indicated a mean annual flood ($Q_{2.33}$) of approximately 97 m³/s for the Waipara River site. On an areal basis ($A^{0.8}$), this indicates a provisional mean annual flood for the Waitohi gauge site of approximately 40 m³/s.

The Regional Flood Methodology (McKerchar and Pearson 1989) derived contour maps that provided site specific values to convert $Q_{2.33}$ to Q_{100} (i.e. annual to 100-year Annual Recurrence Interval [ARI]). This approach is an accepted, independent method of checking flood frequency analyses of gauged catchments. They have also established a procedure for pooling the available site data with the flood estimation contour maps. The $Q_{2.33}$ determined by the Q_{pool} method outlined in Section 5 of the McKerchar and Pearson publication gives $Q_{2.33}$ = 43 m³/s. The $Q_{100}/Q_{2.33}$ ratio is approximately 3.0 for the Waitohi gauge, which indicates a flow for the 100 year ARI event (Q_{100}) of 3.0 x 43 m³/s or 129 m³/s. This gives a good agreement to the areal method.

7.1.3 Design Flows

The Regional Flood Methodology provides some certainty for flood estimates up to the 100 year ARI event. The design flood flows derived for the Waitohi River dams for return periods greater than 100 year are therefore indicative and preliminary in nature. These design inflows are presented in Table 6. The Probable Maximum Flood (PMF) has been provisionally estimated at equal to three times the 100-year flood, which is regarded as an appropriate ratio for preliminary estimates of the PMF.

Table 6: Waitohi River - Design Flood Estimates for each dam site

Flood Event (ARI, years)	Waitohi River Flood Flows (m^3/s)			
	Hurricane Gully	Seven Hills	Inches Road	Lower Gorge 1
2.33	35	39	43	45
5	51	57	63	66
10	65	72	79	83
20	77	86	95	99
50	93	104	115	120
100	106	118	130	135
200	118	132	145	151
500	134	150	165	172
1000	146	163	180	187
10000	187	208	229	239
PMF	317	354	390	406

Note: Flows for return periods greater than 100-year are indicative.

During the detailed design stage of the project, the design standards will be confirmed for the dam design and flood spillway sizing. At this stage, a full review of the flood calculations should be undertaken for the Waitohi River catchment. The study would adopt the established Probable Maximum Precipitation/Probable Maximum Flood (PMP/PMF) Methodology (NZ) and would include:

- The development of a rainfall runoff model calibrated to catchment flows.
- Derivation of the 1,000 year, 10,000 year and PMP input storms as required to provide an appropriate design standard for the relevant Potential Impact category (PIC) rating of the dam.
- Derivation of 1,000-year, 10,000-year and PMF flood inflows as required.

7.2 Flood Management

7.2.1 Inflow Hydrographs

Inflows hydrographs for each dam site were derived by extending the Waitohi River flow series using the strong correlation with the Waipara River series. In addition, recorded floods in the Waitohi catchment from July 2009 and July 2010 were used to confirm hydrograph shape, and all flood hydrographs typically indicated an approximate three-day period from the start to the end of each flood and the hydrograph shapes are very similar as expected

7.2.2 Diversion Sizing

Two alternative construction and diversion approaches for the management of flood flows during construction have been assessed for each reservoir:

1. Roller Compacted Concrete (RCC) dam with lesser protection required to prevent overtopping, i.e. 2 to 5 year ARI.

2. Concrete-face rockfill dam (CFRD) with greater protection to prevent overtopping, i.e. 100 year ARI.

The hydrological software package HEC-HMS (Version 3.5) was used to simulate flood routing through the reservoirs.

The flood routing undertaken for the Hurricane Gully dam confirms that the coffer dam for the RCC option would need to be 7 m high to avoid overtopping in the ARI 5-year event. For the CFRD option, the coffer dam would need to be 20 m high to avoid overtopping in the ARI 100-year event.

The flood routing undertaken for the Seven Hills dam confirms that the coffer dam for the RCC option would need to be 8 m high to avoid overtopping in the ARI 5-year event. For the CFRD option, the coffer dam would need to be 18 m high to avoid overtopping in the ARI 100-year event.

The other Inches Road and Lower Gorge 1 dams will be constructed downstream of the Seven Hills and Inches Road dams respectively and flood inflows may be able to be managed by the operation of the upstream reservoirs and only minor flood protection in the form of coffer dams is may be sufficient.

7.2.3 Service and Emergency Spillway Sizing

The service spillway for each dam has been designed to pass the 100-year ARI flood, and the combined service and emergency spillways have been designed to pass the PMF with some freeboard. A summary of the flood routing results undertaken to date are presented in Table 7. Some refinement of the freeboard requirements and spillway levels and sizing are expected during detailed design.

Table 7: Summary of Reservoir and Spillway Performance during PMF

Reservoir	Spillway Type	Dam Crest RL (m)	Spillway Crest RL (m)	Maximum Reservoir Level RL (m)	PMF Inflow (m^3/s)	Peak Outflow (m^3/s)
Hurricane Gully(1)	Service	505	500(3)	502	317	110
	Emergency		501(3)			32
Seven Hills(2)	Service	405	400(3)	403.3	354	238
	Emergency		402(4)			100
Inches Road(2)	Service	375	370(3)	373.4	374	252
	Emergency		372(4)			115
Lower Gorge 1(1)	Service	345	340(3)	342.2	179	133
	Emergency		341(3)			46

Notes: (1) PMF following Stage 2 scheme operation. (2) PMF during Stage 1 scheme operation. (3) 20 m wide crest. (3) 40 m wide crest.

8.0 Discussion of Dams and Appurtenant Structures

8.1 Potential Dam Types

The key features influencing the dam type and zoning of each dam include:

- The type and location of permanent spillways.
- Methods of diversion during construction and security against overtopping and possible uncontrolled reservoir release during construction.
- Availability of materials, particularly abundance of sandstone to use for rockfill, but limited sources of material for a dam core.
- Founding conditions.
- Resistance to strong seismic shaking.
- To a degree, the storage requirements and height of the dam.

Three concepts for dam types have been considered in the study to date. These are:

- Concrete-faced rockfill dam (CFRD).
- Roller compacted concrete (RCC) gravity dam.
- Earth core with rockfill shoulders (ECD).

In general, the limited sources of material available for the construction of a low permeability earth core favours the RCC and CFRD option above the ECD option. Thus, RCC and CFRD are the preferred dam options at this stage. The pros and cons of each dam type are discussed below and dam volumes for the RCC and CFRD options are presented in Table 8.

Table 8: Dam Volumes for RCC and CFRD

Dam	Crest Level (RL)	Dam Volume (m^3)		Live Storage Volume (MCM)
		RCC	CFRD	
Hurricane Gully	505	695,000	2,700,000	210
Seven Hills	400	60,000	175, 000	7.3
Inches Road	370	25,000	85,000	3.9
Lower Gorge 1	340	8,000	23,000	0.4

8.1.1 Concrete Faced Rockfill (CFRD)

A concrete-faced rockfill dam concept has been developed for all dams, with a number of advantages over an earth dam option. The prevalent sandstone and siltstone bedrock would form the bulk of the dam section (i.e. 'rockfill') with the concrete facing on the upstream face providing the water barrier. The seismic resistance is very good and many large CFRD have now been built worldwide. The main features would include:

- Excavation to rock for a concrete plinth at the toe of the upstream embankment and subsequent grouting beneath the plinth.

- Higher-grade selected rockfill to directly support the concrete slab and to prevent slab deformation and cracking. A potential enhancement is the installation of an extruded curb to support the concrete face and to provide face protection during construction.
- Concrete face on the upstream embankment slope providing impermeable water barrier.
- Possibly foundation filters and drains to control seepage through rock foundations.
- Batter slopes of 1V:1.5H.

Generally a rock foundation is required for a CFRD and this is available for the entire footprint of all proposed embankments. Overseas, CFRD have been constructed on non-rock (or alluvial) foundations, but this in general would not be favoured where alternatives were available.

8.1.2 Roller Compacted Concrete (RCC)

A RCC dam is another favoured option for all dams due to the overtopping resistance during construction and potentially for the lower cost. Investigations will be required to assess the suitability of locally available sandstone as concrete aggregate so it can meet the strength and durability requirements for construction. A number of RCC dams have been recently constructed overseas, for example, in Australia, and have become a major contender for moderate to large dams. Experience in New Zealand is limited to the Horseshoe Bend project in Otago and the spillway section of Opuha Dam in South Canterbury. Worldwide experience in the seismic resistance of gravity dams in general has been good, including the severe Winchuan earthquake in China in 2008. Several large RCC dams were subject to severe seismic shaking in that event without reported major damage.

The quality of the rock foundations in general is a key issue with an RCC dam, as in general higher quality founding rock is required compared to the alternative dam types. In addition, the high seismic loading will place significant strength demands on the foundations. This aspect in turn is influenced by the height of the dam, and for a 105 m dam will be considerable.

The main elements of the spillway would include:

- A stepped profile for energy dissipation, as used for many RCC dams.
- Sidewalls to contain the flow, with a stilling basin lined with concrete located within the river channel.

The alignment of the river channel downstream is also important in order that excessive excavation for a stilling basin is avoided and transition to the river section is not too sharp. All dam locations have favourable downstream receiving river geometries as they are essentially on straight sections of river.

8.1.3 Earth Dam (ECD)

The earth dam option on present information is less favoured. Rockfill can form the bulk of the dam shoulders, with a central earth core. However, no significant volumes of either silty-gravel or fine-grained materials for a core have been identified to date and this is the most significant constraint for this type of dam. Possible features of an earth dam would include:

- Batter slopes at about 1V:2H and slightly flatter on the upstream slope (say 1V:2.5H).

- Fully intercepting chimney drain connecting to a base drainage blanket to safely intercept seepage through the dam and upper foundations.
- Excavation to rock on both abutments and river section for foundations.
- A layer of rip-rap over the reservoir operating range.
- Grouting of rock foundations.
- A narrow core if core materials are scarce (which is likely).

The main disadvantages of this option include:

- Lack of dam core material.
- Greater vulnerability to flood risk during construction.
- For construction greater weather dependency due to core placement requirements.
- Higher volume of material required due to flatter slopes.

8.1.4 Construction Materials

A key consideration for the determination of final dam type is the availability of construction materials.

8.1.4.1 Earth Dam (ECD)

No appreciable exposures of finer-grained material for a lower permeability dam core have been encountered during this investigation, which has significantly limited an ECD-type of dam at these sites. The closest source is likely to be Tertiary sediments some 20 km downstream of the site.

8.1.4.2 Roller Compacted Concrete (RCC)

It is anticipated that terrace alluvium and colluvium near the proposed dam site will not provide sufficient quantities of aggregate for construction of an RCC dam.

Local aggregate options for an RCC dam are limited to site-available sandstone, which will require testing to determine its strength and durability and potential suitability as concrete aggregate. Precedent for the use of sandstone as concrete aggregate is seen at Wyaralong Dam in Australia[1], however this was in an area of low seismic loading. Sources of sandstone could be from bedrock stripped from the dam and spillway foundations, and quarrying and crushing of nearby sources of bedrock. A potential source of good quality aggregate is a knob of bedrock located 900 m upstream of the dam on the true right bank of the Waitohi River.

A suite of testing will be required to determine feasibility of an RCC dam constructed from local materials, such as aggregate crushing tests (to evaluate wastage from sandstone breakdown), aggregate testing (strength, petrographic and XRD analysis, durability, abrasion resistance), and RCC testing.

[1] Stratford, C., E. Schwartz, R. Montalvo, E. Schrader, and R. Herweynen, 2011. Wyaralong RCC dam summary and the impact of 'low' quality aggregate on design, In *21st Century Dam Design – Advances and Adaptations*, 31st Annual USSD Conference San Diego, California, April 11-15, 2001, pp.245-258.

8.1.4.3 Concrete Faced Rockfill (CFRD)

For rockfill sources, the availability of quantities of acceptable material requires evaluation. Excavations and potential quarries should be core-drilled, and the cores tested for compressive strength in both the dry and saturated conditions.

8.1.5 Discussion

Concrete sources for conventional concrete for the concrete face, diversion culverts, and spillways also require evaluation and compared with importing aggregate. Subsequent investigation phases will be required to confirm the best source of each construction material, and assess measures to mitigate potential constraints. Types and preferred sources of construction materials are summarised in Table 9.

Table 9: Main Construction Details

Construction materials	Source	Comments
Concrete Aggregate (RCC)	Terrace Alluvium	• Only limited quantities available. • Processing and washing required to provide aggregate within a specified grading-envelope. • Potentially high quantities of cohesive material may require processing.
	Colluvium	• Processing and washing required as per terrace alluvium. • Potentially high quantities of cohesive material may require processing.
	Esk Head belt sandstone Bedrock	• Processing and washing required as per terrace alluvium. • Siltstone may constrain viability of quarrying for aggregate.
Concrete Aggregate (Conventional)	Terrace Alluvium	• Only limited quantities available. • Processing and washing required to provide aggregate within a specified grading-envelope. • Potentially high quantities of cohesive material may require processing.
	Esk Head belt sandstone Bedrock	• Processing and washing required as per terrace alluvium. • Siltstone may constrain viability of quarrying for aggregate.
Rock fill / rip rap (i.e. greater than 0.5 m sized blocks	Sandstone	• Siltstone quantities in placed rockfill should be minimised. • Quarried from suitable exposure near the site. • Any quarrying operation should not decrease slope stability.
Filters	Esk Head belt sandstone Bedrock	• Lower volumes of specifically designed and graded filters, with low-fines content, may be required to control seepage in the embankment depending on the final grading of the rockfill. These can be produced by high standard processing of selected higher-quality geology.

A preferred dam concept on present information for all dam sites has been developed. This concept involves a CFRD dam across the full width of each proposed site. As highlighted by the geological work undertaken to date, due to the shallowness of the rock on both abutments and in the gully floor, excessive excavation would not be required for a CFRD dam across the entire cross-section. The treatment of the rock foundations to ensure excessive leakage does not occur requires careful evaluation. Treatment, such as grouting, of the gully floor and abutments would be an accepted method of mitigating potential seepage concerns and is a standard item for CFRD construction.

8.2 Dam Layout

A conceptual design of all four dams has been prepared for project planning purposes. It should be noted that specific details may be altered during the detail design and review process to ensure a robust and economic design. The layout of the proposed dam and appurtenant structures for the purposes of presenting drawings are assumed to be CFRD options and are presented in the drawings detailed in Table 10 with key details presented in Table 11. All drawings are in Appendix C and stage – storage volumes for all dams are included in Appendix A.

Table 10: Dam Layout Drawing Numbers

Dam	RILEY Drawing Numbers
Hurricane Gully	11841-301 to 302
Seven Hills	11841-311
Inches Road	11841-321
Lower Gorge 1	11841-331

Table 11: Key Details of Dams

Dam	Crest RL (m)	Water RL (m)	Dam Height (m)	Crest Length (m)	Drawdown (m)	Live Storage Volume (MCM)	Peak Inflow (m^3/s)	Peak Generation Outflow (m^3/s)	Peak Irrigation Outflow (m^3/s)
Hurricane Gully	505	500	105	429	50	209.5	17(1)	35	42
Seven Hills	405	400	46	150	20	7.8	42	8	42
Inches Road	375	370	31	245	20	3.9	42	8	42
Lower Gorge 1	345	340	21	71	Nil	0.4	42	8	42

Note: (1) This is the maximum pumped flow rate and does not include natural inflows of the Waitohi River.

A number of key details are common for all four dams, and the layout features include the following:

- A straight dam axis across the narrowest point of the gorge/valley. All dam axes have been chosen to balance the needs of the staging of the scheme development (i.e. maximising combined scheme storage) and to minimise fill volumes in their respective locations.
- The typical dam section has 1.5H:1V upstream and downstream slopes, which is typical of CFRD for lesser quality rockfill[2]. This batter-slope is flatter than typically used for high-quality free-draining rockfill.

[2] Fell, R., P. MacGregor, D. Stapledon, and G. Bell. 2005. *Geotechnical Engineering of Dams*. London: A. A. Balkema Publishers. Pg. 615.

- A concrete parapet wave-wall is included on the crest to cover freeboard requirements during flood events, and is an effective means of reducing the overall quantities of rockfill. The economics of the parapet wall will be assessed for the lower dams as the wall is generally less cost-effective in smaller dams.
- The rockfill will consist of sandstone from a suitable borrow area(s) within the reservoir area on the side slopes where stripping is nominal with processing required for some dam zones. Typically in CFRD the rockfill will be free draining to reduce the build-up of pore water pressures within the embankment; if during the process of detailed design the rockfill does not have adequate permeability then internal drains will be required to control seepage flows.
- The upstream slope of the embankment that supports the concrete face will be compacted and sealed to hold the compacted face intact and to protect the face from erosion. An extruded concrete curb may be used as the finishing surface as per modern practice.
- A typical design for the concrete face is a slab 300 mm thick at the top increasing to 500 mm near the plinth.
- A plinth will be provided at the toe of the upstream face of the dam to provide a watertight connection between the face slab and the dam foundation. The dimensions of the plinth are generally set by assessing the allowable hydraulic gradient for the particular foundation rock quality. It is typical to install a continuous grout curtain beneath the plinth to minimise the risk of seepage losses beneath the embankment. The extent of grouting that may be required will require further investigation.
- An un-gated service spillway designed to spill for all flows and will include an ogee crest, spillway chute, and flip bucket all constructed of concrete. This spillway will contain all flows below the AEP 1 in 100-year flood. The energy from the spillway flows will be dissipated in a rock-lined plunge-pool before re-entering the Waitohi River at the base of each dam. The plunge pool is required to ensure that the high velocity water spilling from the dam does not erode any weak siltstone or mudstone rock. The economics of this type of arrangement for all dams will be assessed during detailed design; more cost effective arrangements for the lower three dams may be available.
- An emergency spillway to control spilling from floods above the AEP 1 in 100-year. The emergency spillway will generally be excavated in rock to mitigate against erosion and will convey the water away from the toe of the downstream embankment.
- Diversion of the Waitohi River to allow foundation preparation and embankment construction. The diversion will be sized to pass floods during construction. Upon completion of the dam the diversion will become part of the main structure as the irrigation/hydro-generation outlet following the upstream end being blocked by concrete.

8.3 River Diversion during Construction

Diversion during construction is an important feasibility issue and there are no specific guidelines for their sizing as it is essentially a construction risk-based decision. The choice of diversion type and design capacity is also strongly influenced by the dam type. As discussed in Section 6.1.2, diversion structures for an RCC dam are able to be designed to a lower standard as it is able to withstand overtopping without failure. A CFRD, however, requires a higher standard due to the erodible nature of the embankment material.

Due to the large irrigation releases that will be required from all four dams (i.e. $42m^3/s$), it is expected that a large conduit will be required to meet this flow capacity. If a large conduit is to be constructed, it would seem practical that after completion of each dam the river diversion structure be converted into a combined irrigation supply and hydro-generation conduit. Given the irrigation supply flows are approximately equivalent to the annual flood from the natural inflows to the Waitohi River catchment, the incremental cost in providing additional diversion capacity in the conduit may prove economic.

Additional storage capacity through the construction of coffer dams upstream of the construction area can reduce the size of the diversion culvert as the coffer dam attenuates floods and reduces peak outflows. Furthermore, due to the large size of the reservoir storage once the dam reaches a certain height, there is the ability to store a large flood.

Other diversion options have been considered for each dam, including:

- Upstream and downstream coffer dams with a tunnel through the abutment. This is not considered a suitable option due to the associated tunnelling costs.
- Coffer dams as above, but with a diversion conduit constructed on a bench excavated onto the abutment.
- Diversion channels at high levels on the abutments (for later stages of construction). This is not considered a suitable option as temporary channels at high levels on the abutments are disruptive to construction and are not favoured unless there are no practical alternatives.

A preliminary flood routing exercise has been undertaken (refer Section 7.2.2 and Appendix A) to develop a concept for a diversion strategy for the two dam types.

8.4 Spillways

In their current configuration, all dams will have the following spillways:

- Service spillway: the service spillway will be an un-gated ogee crest spillway with a concrete-lined chute to a flip bucket and plunge pool, or stilling basin to dissipate energy. This spillway will be required to pass floods up to the 100-year flood; for events greater than the 100-year flood, this spillway will work in conjunction with the emergency spillway to pass floods.
- Emergency spillway: the emergency spillway is a channel excavated into rock that conveys high flows away from the toe of the dam. This spillway will operate at events above the 100-year flood and be able to pass the full PMF without damage to the main embankment.

These spillway arrangements will require confirmation during detailed design. For instance, gated spillways are also a possibility, particularly with the inclusion of hydro-generation capabilities at each dam.

Preliminary flood routing as described in Section 7 has been undertaken for all dams to determine preliminary spillway dimensions and to assess freeboard requirements. A summary of the results are presented in Table 7 in the earlier Section of this report.

8.5 Hydro-generation Layout

The hydro-generation aspect of the scheme utilises the available head down the Waitohi River to generate electricity at times of irrigation flow, with limited peaking available at Hurricane Gully. The key details of the hydro-generation capabilities at each dam are summarised in Table 12.

Table 12: Key Details of Hydro-Generation

Dam	Generation Head (m)	Peak Generation Outflow (m^3/s)(1)	Installed Capacity (MW)(1)
Hurricane Gully	50-100	35	26
Seven Hills	16-41	8	4
Inches Road	5-26	8	4
Lower Gorge 1	16	8	3.5

Note: (1) Concept design figures, capacities may vary slightly in final design, but will not exceed the irrigation outflow.

At each dam, following decommissioning of the diversion culvert, the culvert will be converted into a combined irrigation/hydro-generation supply conduit. The powerhouse will be situated immediately downstream of the dam. Screened intakes located in the reservoir will supply the turbines. Due to the potential for drawdown of each reservoir, a multi-level intake structure will be required.

The peak generation flow at each powerhouse will be less than the peak outflow for irrigation, so a bypass will be required to ensure full irrigation flow is available at all times.

8.6 Compensation Flow Outlet

It is envisaged that in addition to irrigation outlet works, a low flow bypass will be required for compensation flows. This will be incorporated into the combined irrigation/hydro-generation infrastructure.

8.7 Reservoir Operation

The reservoir drawdown levels are set by the scheme's irrigation demand. Table 11 gives details of the drawdown for each of the four dams and the storage volume associated with this level of drawdown. The details of reservoir operation are covered in the Project's hydrology report by PDP.

9.0 Intakes

This section outlines the preliminary detail of the four intakes that are proposed to divert water into the distribution races and into storage. Fish exclusion and sediment management are significant issues for most of the intakes. The design at this stage is conceptual only and the investigation of the intake sites has been from a visual inspection only. Four of the proposed intakes are new structures and the fifth is the existing intake for the Amuri Irrigation scheme which will be enlarged to take an increased flow. The Amuri intake is not part of the current consent but will become part of the proposed scheme.

The Intake on the Hurunui at Mandamus is part of an existing consent application by HWP which includes a dam on the South Branch of the Hurunui River and an outlet control gate on Lake Sumner.

9.1 Hurunui River at Mandamus (Intake 1)

An intake is proposed to be located on the South side of the Hurunui downstream of the Mandamus River. The Intake is included in the Resource Consent Application for a dam on the South Branch of the Hurunui River and control gates on Lake Sumner. The intake needs to be located as far upstream as possible to enable as much of the proposed irrigation area to be provided flow via gravity. Drawing 11841-230 shows a conceptual plan of the intake, sediment pond and fish screen.

The intake will include a gate to control the flows which will be directed into a sediment pond and a fish exclusion system which will most likely be a mesh screen or an infiltration gallery. The fish screen will be designed to meet the Fish Screening – good practice guidelines for Canterbury (NIWA October 2007).

It is proposed to divert up to 26.1 m^3/s of water at this intake for irrigation. An additional flow will be required to provide fish passage as part of the fish exclusion system and typically 5 to 10% of the diverted flow is recommended. Additional flows will be required as part of the sediment management system which may allow for higher flows into the intake to flush sediment during higher river flows.

For stage two flow from the Waitohi Storage will at times be required to flow back to the Hurunui upstream of the existing Amuri Intake to supply the north side of the river during very low Hurunui River flows.

9.2 Waitohi River (Intake 2)

Water will be released from storage on the Waitohi River and then will require an intake downstream to divert flow into the distribution system. In Stage 1 an intake at the location of where the main distribution canal crosses the Waitohi River will be required. This intake will be in the form of a weir and flow diversion bund across the river to take up to 3.9 m^3/s flow released from the storage dams. A concept sketch of the intake is shown on Drawing 11841-220. This intake will be decommissioned following the completion of Stage 2 of the project.

9.3 Hurunui River at Pump Station (Intake 3)

This intake will be similar to Intake 1 in requiring inflow control, sediment management and fish exclusion. A pump station will be located at the intake to pump up to 17 m^3/s to the Waitohi catchment. Drawing 11841-200 shows a potential concept layout for the intake.

The intake would include a coarse screen to prevent large debris entering the scheme with a gate to control the flows into the sluice channel. The sluice channel will provide a means to keep the larger sediment from entering the pond and along with the second stage of the intake into the pond to provide a first order means to exclude some of the fish. Flow will then pass through to a sediment pond with an infiltration gallery intake to a pump station. A fish bypass along with a separate sluice/emergency spillway back to the river will be located at the downstream end of the pond beyond the screens. An alternative to this would be to use mesh screens instead of an infiltration gallery.

It is proposed to divert up to 17 m^3/s of water from this location. Additional flow required for fish exclusion is expected to be approximately 10% of the flow. Additional flows will be required as part of the sediment management system which may allow for higher flows into the intake to flush sediment during higher river flows.

9.4 Hurunui River (North Bank) (Intake 4)

It is proposed to divert up to 8.5 m^3/s of water from this location to the existing Amuri Irrigation intake with an allowance for additional flows for fish screening. This intake is not included in the current resource consent application but will be part of later consent applications.

9.5 Waitohi River and Lower Gorge 1 Dam (Intake 5)

In Stage 2 this intake will be constructed as part of the Lower Gorge 1 Dam. A high-level intake is required to supply gravity-fed water to irrigable areas between RL340 m and RL305 m that cannot be supplied from the main distribution canal. This is expected to be a gate-controlled intake directly from the Lower Gorge 1 reservoir. A low-level intake is required to divert up to 42.4 m^3/s of water into the main distribution canal. This is expected to be diverted directly from the combined irrigation/hydro-generation tailrace channel into a canal.

10.0 Scope of Further Investigations

10.1 General

The level of detail for work undertaken to date is appropriate for a pre-feasibility assessment. In order to extend the engineering studies to full feasibility stage, further investigations are recommended. These will allow for refinement of the geological model, and elucidation of geotechnical constraints relating to dam foundation and reservoir water-tightness.

10.2 Geotechnical Investigations

Several additional stages of geotechnical assessment are recommended to investigate the feasibility of the proposed development. These should incorporate several key elements, including the following with the scope and purpose of which are summarised below in Table 13:

- Strength, compressibility, erodability, and permeability of the embankment and plinth foundation.
- For the plinth, sufficient boreholes should be drilled to enable the depth to rock of acceptable quality to be estimated for a range of plinth locations corresponding to different dam axes and different upstream slopes. Particular concern is the high hydraulic gradients across the plinth foundation and the potential for erosion of the plinth (and transition zone) foundation.
- For the embankment foundation, compressibility of the foundation rock must be assessed to ensure that compression will not lead to excessive or differential settlement that may induce cracking of the upstream concrete face. This is particularly important in the area immediately downstream of the plinth (the transition zone) that carries the highest water load.

Table 13: Scope of Investigations

Investigation Type	Primary Purposes
1. Procure 1:16,000-scale prints of aerial photos, available from Precision Aerial Ltd. Photos procured should cover the four proposed dam and reservoir sites and intake/pipe/tunnel area	Allow more detailed assessment of slope stability and landforms in dam and reservoir areas; provide good-resolution base plan images for more accurate on-the-ground mapping; aid design of additional phases of assessment.
2. Additional detailed geological mapping of the four dam and reservoir sites, and the proposed tunnel alignment.	Gather data to allow quantitative assessment of defects and potential seepage paths; refine distribution of rock and soil types; more detailed assessment of potential instability features
3. Drill holes at dam sites (some angled in bedrock) and undertake down-hole strength and permeability (lugeon) tests (in rock and soil).	Refine geological model, and correlate with geological mapping. Assess foundation condition, and treatment required for proposed dam types. Establish groundwater conditions.
4. Drill holes along tunnel alignment to develop geological model, establish appropriate tunnelling methodology, and confirm alignment.	Determine most cost-effective alignment and tunnelling methodology.
5. Mechanical test pits and trenches at dam sites, potential borrow areas, and other select areas to confirm the geological model.	Assist in geological model and obtain samples for lab testing. Assess availability/volume of suitable materials for proposed dam types e.g. filter zones, general rockfill sources, aggregate sources for RCC.
6. Assessment of suitability of local rock (sandstone/siltsone) to be used as an aggregate for the RCC concrete supply, for concrete supply for CFRD concrete face, and for conventional concrete for spillway structure.	Will determine the feasibility of RCC-type dam to be used; will determine economics of concrete supply for CFRD option.
7. Laboratory tests for grading, dispersivity, strength, durability etc.	Refine material properties and constraints on material usage; determine suitability of site materials for processing as concrete aggregate.

10.3 Hydrology and Spillways/Diversion

The spillway sizing and preliminary flood calculations to date are considered to be satisfactory to demonstrate feasibility. Further analyses may be necessary to refine options for diversion and spillway arrangements at each dam.

10.4 Effect of Drawdown Range on CFRD Dam

Confirmation of the rockfill properties will be required with regard to the potential breakdown of the sandstone during processing and transportation into ‘dirty’ rockfill that may cause a reduction in the overall permeability of the embankment.

Due to the nature of the materials CFRD are constructed from (i.e. rockfill) the embankments are typically free-draining with no pore-water pressure build-up (Section 10.7). This means that drawdown of the reservoir to the levels described above will not result in instability created by changing pore water pressures in the embankment. Confirmation of the rockfill properties is required with regard to the potential breakdown of the sandstone during processing and transportation into ‘dirty’ rockfill that may cause a reduction in the overall permeability of the embankment. Precedent for this drawdown range in CFRD is seen in the Karahnijukar Dam in Iceland completed in 2009.

10.5 Seismic Hazard

It is recommended that a preliminary-level site specific seismic hazard study is completed. It is envisaged this would involve:

- Assessment of active faulting and development of site specific spectra on the basis of a desktop study.
- Acceleration response spectra using the natural seismic hazard model will be derived, for an OBE (150 year return period) and a maximum design earthquake (MDE).
- The MDE spectra should be determined by using both probabilistic and scenario spectra for the estimated 50 and 84 percentile motions from rupture of nearby faults.

10.6 Preliminary Design and Comparison of Options

The preceding activities will provide input to a further comparison of dam types. It is envisaged that the main areas would include:

- Assessment of feasibility to a greater level of detail, with particular emphasis on foundation treatment required and constraints on particular dam types.
- Detailed assessment of the potential for local sandstone to be used as concrete, either as aggregate for an RCC dam, or for the concrete face of the CFRD option.
- Further comparison of dam types and storage range.
- Assessment of reservoir slope stability and whether risk mitigation is required.
- Preliminary analyses for CFRD dam option for the seismic loads defined, with particular reference to foundation stability.

11.0 Limitation

This report has been prepared solely for the benefit of Hurunui Water Project as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.