DAMWATCH
ENGINEERING BEYOND THE SURFACE

Wrights Road Storage Ponds

Design Report

26 October 2021

Prepared for Waimakariri Irrigation Limited

Issue 6

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Project Title	Wrights Road Storage Ponds
Document Title	Design Report
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Damwatch Project No	E2131

Document History and Status

Version No.	Issue Date	Description	Prepared by	Reviewed by	Approved by
Issue 1	May 2012	Initial Submission	PS	NC	SM
Issue 2	Dec 2012	Update for High PIC	PS/BV	NC	SM
Issue 3	Feb 2013	Incorporate comments from External Peer Reviewer	NC/BV	NC	SM
Issue 4	Dec 2015	Incorporate comments from External Peer Review	NC/BV	NC	SM
Issue 5	Sep 2021	Update to incorporate design changes and ensure alignment with 2015 NZ Dam Safety Guidelines	NC/GB/EO	NC	SM
Issue 6	Oct 2021	Update to address Independent Reviewer comments	EO	NC	SM

Current Document Approval

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Reviewed By Nigel Connell

Approved for Issue Steve McInerney Signature:

Executive Summary

Built in 1997, the Waimakariri irrigation scheme is owned and operated by Waimakariri Irrigation Ltd (WIL). It utilises water taken from the Waimakariri River. At present, the scheme irrigates about 18,000 ha in North Canterbury, with races between the Waimakariri River and the Ashley River.

Current resource consents allow maximum abstraction of 16.6 m^3/s from the river as:

- Maximum irrigation take of 10.5 m^3/s (WIL consent)
- Maximum stock take of 2.1 m^3/s (WIL consent)
- Maximum Ngai Tahu take of 4.0 m^3/s (recent Ngai Tahu consent)

WIL acquired a 120 ha property on the corner of Dixon and Wrights roads adjacent to the existing Buffer Pond and main race MR4. It is located about 12 km downstream of the Waimakariri intake at Browns Rock.

The maximum consented irrigation water take of 14.5 m^3/s is not required throughout the irrigation period. Therefore, WIL plans to construct storage to supplement irrigation supply when the take is restricted due to low flow in the Waimakariri River. Irrigation water can thus be made available at critical times of the season, when it will provide the most benefit to irrigators.

Rooney Earthmoving Ltd (REL) carried out a detailed GPS survey of the site and modelled a number of different options for storage. The option that gave the best storage and supply to the irrigated area has been adopted for detailed design. A total of nine days storage to the 18,000 ha presently irrigated has been estimated assuming that all the ponds are initially full.

The footprint of the storage is approximately a square 1 km x 1 km. The storage is divided by a middle embankment forming Pond 1 and Pond 2, with full supply level of Pond 1 (RL 226.50 m) being 3.7 m higher than that of Pond 2 (RL 222.80 m). The ponds are designed for 8.2 Mm^3 storage capacity.

The main components of Wrights Road Storage Ponds are:

- Six inlet and outlet culvert pipes G1 through G6;
- One service spillway which delivers flows from the Buffer Pond to Pond 2;
- Three emergency spillways protect the Buffer Pond, Pond 1 and the Tub;
- Fuse plug spillway in the buffer pond to prevent overtopping of the Pond 2 embankment.
- The Tub, which conveys flow from Ponds 1 and 2 into the main race MR4 through G6;
- Buffer Pond pump station to fill Pond 1;
- Pond 2 pump station to lift bottom water from Pond 2 into main race MR4 via the Tub and G6;
- A penstock which conveys water from Pond 2 to race R2, via G2, which has the potential to connect to a hydro-power station, if one is developed.
- Two flow measuring crump weirs on distribution races R2 and R3 downstream of G2 and G3 outlets;
- Ultrasonic flow measurement on the G6 culverts to measure flow into the main race MR4; and
- Control System to control gates on pond inlet and outlets to manage operation.

As noted, there is potential to construct a hydro-power station, which could utilise the available head between Pond 2 and the low elevation at Wrights Road when irrigation demand is low. A new tailrace canal would then deliver water to other proposed power stations near the Waimakariri River. This is outside the scope of the Wrights Road Storage Pond Design.

The storage ponds are designed to discharge water in three directions:

1. Into main race MR4 (irrigation water only, as stock water is supplied from the Buffer Pond)
2. Into distribution race R2 (irrigation and stock water)
3. Into distribution race R3 (irrigation and stock water)

Automation for the storage pond site will be compatible with the existing WIL scheme automation. The control system will communicate with the intake at Browns Rock; operate all the gates G1 through G6 and the Buffer Pond and Pond 2 pump stations. The gate G1 is fully automated and opened and closed to maintain the level in the Buffer Pond. Discharge from all three outlets to MR4, R2 and R3 is measured and communicated to the Control System.

Each pond is a large dam as defined by the Building Act (2004), as each pond has greater than 4 m depth of water and greater than 20,000 m^3 reservoir volume. The design and specification for construction and operation of the facility has addressed hazards that have the potential to impact on the safety of the water retaining structures. The main philosophy is that measures are put in place in design, construction and operation in order to prevent a dam breach from occurring.

The assessment of consequences from a hypothetical dam breach allows the determination of the Potential Impact Category (PIC) of a dam. PIC is a qualitative categorisation of water retaining structures according to the potential consequences of failure. The PIC links to the design and management of dams because the NZSOLD Dam Safety Guidelines use PIC to define the design load and the level of expertise and detail applied to investigation, design, construction, commissioning, surveillance and safety reviews of dams.

A fundamental principle proposed for the Wrights Road Storage ponds is that different Potential Impact Classifications can be applied to discrete lengths of the pond embankment, because the downstream consequences of breach vary depending on the section of the embankment breached. It follows that breach of the west embankment would have less consequence than breach of any of the other embankments. Accordingly, for the proposed Wrights Road storage ponds, discrete sections of the embankment can be assigned a different PIC, as is done with large canal embankments. This then results in different standards for design and construction of discrete sections of the pond embankments, as summarised in Figure 1, where the western embankment and embankment to the north of the tub are classified as Medium PIC and all other embankments are classified as High PIC.

Figure 1: Summary of PIC Assessment for Embankments of Proposed Wrights Road Storage ponds

During the period of design, the Building (Dam Safety) Regulations 2008 were revoked in 2015 and the revised NZSOLD Guidelines (NZSOLD, 2015) were released in 2015. The revised NZDSG adopt the methodology set out in the regulations for determining the potential impact classification (PIC) of a dam, which in turn determines the standard for design of the dam. Although the PIC for the Wrights Road Storage Ponds are based on the revoked Regulations, the PIC is in accordance with the revised NZSOLD Guidelines.

In other aspects of design, the revised NZSOLD Guidelines incorporate no changes or additions that materially affect the design, which is based on the 2000 NZSOLD, Guidelines.

Careful consideration has been taken with seismic design, taking into account lessons from recent major earthquakes in the Canterbury region. The report discusses the expected performance of the ponds based on a site specific seismic assessment carried out by GNS Science (GNS). Ground motion parameters provided by GNS were used to evaluate the pond stability under earthquake loading. The embankments have been designed to meet the High PIC performance requirements outlined in the NZDSG (NZSOLD, 2015).

Waimakariri River sits within the vast expanse of the Canterbury Plains. Large parts of the plains consist of outwash alluvial deposits comprising gravel, silt, clay and peat. The basement rocks are Paleozoic to Mesozoic sedimentary and metamorphic rocks. Two general soil units were encountered in the test pits and are described as:

- Finer soils encountered in the upper unit; from ground surface to 0.6 m depth, and
- Coarse soils.

Materials for pond construction are to be sourced from the footprint of the ponds. Permeability of gravels underlying the pond footprint necessitates a lining to control seepage from the ponds and this lining is to extend up the height of the pond embankments.

The groundwater table at Wrights Road Storage Ponds site was assessed from monitoring data of two nearby wells. The groundwater level at the ponds site is estimated to be at least 20 m below ground surface.

Two lining systems were considered; a three layered composite earth lining and a synthetic geomembrane. A synthetic geomembrane lining has been adopted due to:

- Limited availability of silt on site as a low permeability lining material;
- Insufficient wave protection rock of adequate size in the foundation Gravels;
- Potential leakage from a composite soil liner with a thin low permeability silt layer; and
- Superior sealing and resultant low leakage achievable with a geomembrane.

Design of the pond embankments considered the following failure modes:

- Embankment failure due to earthquake induced slumping and consequent overtopping of water at locations of reduced embankment crest levels;
- Embankment failure due to earthquake induced local deformations and consequent leakage through the damaged geomembrane liner; and
- Piping through the embankment or foundation due to leakage through the liner leading to failure, possibly initiated by earthquake shaking.

Internal embankment slopes of 1V:3H have been adopted based on experience with construction of geomembrane lined embankment slopes. Embankment outside slopes of 1V:2H have been adopted for the Medium PIC embankment, and 1V:2.5H slopes for High PIC embankments, based on experience with similar gravel embankments and stability analysis.

1.5 m of freeboard was adopted to contain wave run up and also to accommodate overflow spillway capacity. The occurrence of large earthquake induced waves (seiches) are considered very remote and, if generated, are likely to be less than 0.3 m in wave height and therefore contained within the adopted freeboard of 1.5 m.

Fourteen standpipe piezometers are included extending through the embankments into the gravels underlying the ponds to enable monitoring of the groundwater level under the lining of the ponds.

Design life for the pond embankments and appurtenant structures is for greater than 50 years. The lifetime of the embankment liner is expected to be less than 50 years. Replacement of the geomembrane liner is therefore expected over the design life of the pond, and this is accepted by the pond Owner. Longevity of all structures will be contingent on construction being implemented in accordance with the design and specification and appropriate maintenance of the structures.

Construction quality will be managed with a control program, which the Contractor will set up and operate. The quality control program includes material testing and sampling; compaction tests, soil aggregate grading's and geomembrane testing to establish correct physical properties and maintenance of these during construction. It further includes construction trials needed to establish that construction methods to be used produce construction consistent with the design.

A commissioning plan has been developed and will be finalised later in the construction phase. This is initially to test operation of the control system (dry testing) and, subsequently to initiate filling of the ponds. This will include a program for observation of performance by visual inspection, piezometer readings and observation of operation of the hydraulic structures in accordance with the NZDSG.

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Abbreviations

2D	2-dimensional
3D	3-dimensional
ACI	American Concrete Institute
ADSR	Annual Dam Safety Review
AEP	Annual Exceedance Probability
BGL	Below Ground Level
CDSR	Comprehensive Dam Safety Review
CLSM	Controlled Low-Strength Material
CME	Controlling Maximum Earthquake
DCR	Demand Capacity Ratio
D/S	Downstream
EPRI	Electric Power Research Institute
GMPE	Ground Motion Prediction Equation
GNS	Geological and Nuclear Sciences
MBIE	Ministry of Business, Innovation and Employment
MCL	Maximum Control Level
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
MHIL	Mayfield Hinds Irrigation Limited
NIWA	National Institute of Water & Atmospheric
NSHM	National Seismic Hazard Model
NZDSG	New Zealand Dam Safety Guidelines
NZS	New Zealand Standards
NZSOLD	New Zealand Society on Large Dams
OBE	Operating Basis Earthquake
O&M	Operation and Maintenance
PIC	Potential Impact Classification
PLC	Programmable Logic Controller
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PGA	Peak Ground Acceleration
REL	Rooney Earthmoving Limited
RL	Reduced Level
SEE	Safety Evaluation Earthquake
TAF	Topographical Amplification Factor
TDR	Time Domain Reflectometry
U/S	Upstream
USACE	United States Army Corps of Engineers
WIL	Waimakariri Irrigation Limited

1.0 Introduction

1.1 Principal Requirements

This docucment is considered as the Principal Requirements. It provides the purpose and performance requirements for the works and includes the detailed deisgn, technical specifications and detailed design drawings.

1.2 Background

Waimakariri Irrigation Ltd (WIL) owns and operates the Waimakariri irrigation scheme. This scheme was built in 1997. It utilises water taken from the Waimakariri River. At present, the scheme extends to about 18,000 ha in North Canterbury. This water is distributed through races between the Waimakariri River and the Ashley River (refer to Figure 2).

Current resource consents allow maximum abstraction of 16.6 m^3/s from the river as:

- Maximum irrigation take of 10.5 m^3/s (WIL consent)
- Maximum stock take of 2.1 m^3/s (WIL consent)
- Maximum Ngai Tahu take of 4.0 m^3/s (recent Ngai Tahu consent)

However, the maximum consented irrigation water take of 14.5 m^3/s is not fully required throughout the irrigation period. WIL plans to construct an off-stream irrigation water storage to supplement irrigation supply when the take is restricted due to low flow in the Waimakariri River (refer to Figure 3). Design of the storage ponds is the subject of this report.

WIL have purchased a 120 ha property on the corner of Dixon and Wrights roads and adjacent to the existing Buffer Pond and main race MR4. It is located about 12 km downstream of the Waimakariri intake at Browns Rock (refer to Figure 2).

Available take, surplus to demand, will initially be gravitated from the Buffer Pond to both Pond 1 and Pond 2 of the new development. Subsequently, when Pond 1 rises to near the maximum water level of the Buffer Pond, the Buffer Pond pump station will fill Pond 1 further to its full supply level, so that the stored water can be fed back into the scheme when supply is restricted. Irrigation water can thus be made available at critical times of the season when it will provide the most benefit to irrigators.

This storage will reduce the effect of restrictions placed on the scheme during periods of low river flow and thus improve the efficiency of the overall scheme. A feasibility study (MWH, 2007) was completed by MWH NZ Ltd in June 2007, which examined several alternative storage sites.

Subsequently, preliminary design by REL was completed in October 2011 specific to a 120 ha block pond site on the corner of Dixon and Wrights Roads.

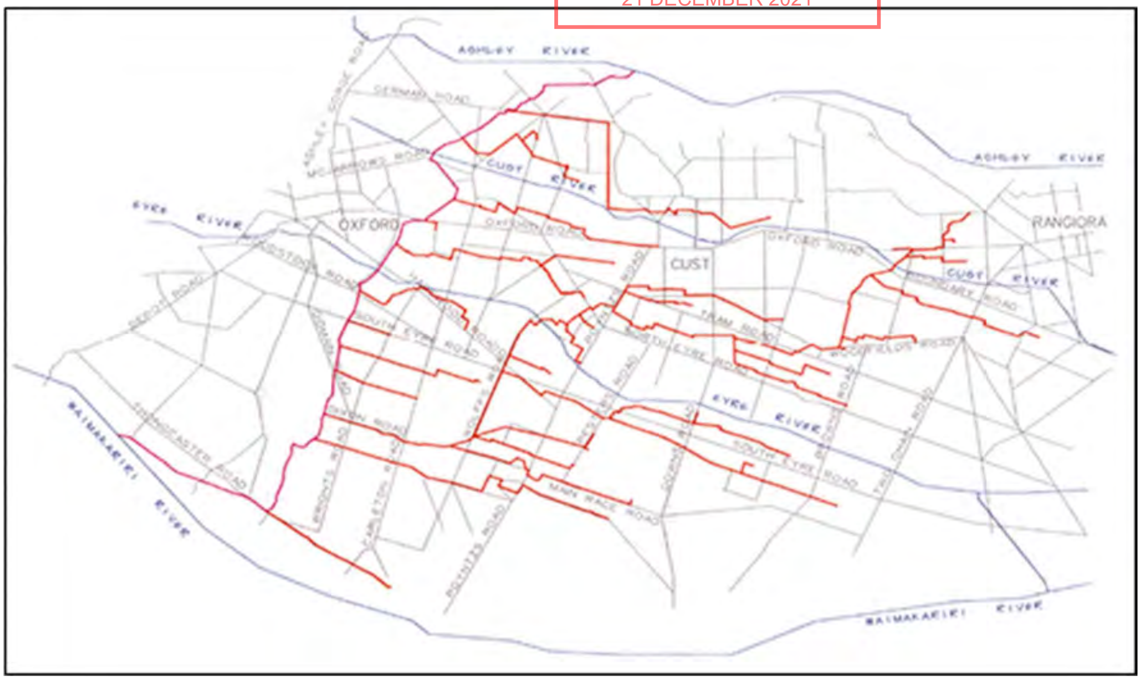


Figure 2: Storage Ponds and Scheme Layout

REL carried out a detailed GPS survey of the site and modelled a number of different options for storage. The preferred option gave the best storage and supply to the irrigated area. A total of nine days storage to the 18,000 ha presently irrigated has been estimated assuming that all the ponds are initially full.

There is potential to construct a hydro-power station in the future. The potential power station could utilise the available head between the level of Pond 2 and the low elevation at Wrights Road. This is outside the scope of the Wrights Road Storage Pond Design.

During the period of design the Building (Dam Safety) Regulations 2008 were revoked in 2015 and the revised NZSOLD, 2015, Guidelines were issued. The revised NZDSG adopt the methodology set out in the Regulations for determining the potential impact classification (PIC) of a dam, which in turn determines the standard for design of the dam. Although the PIC for the Wrights Road Storage Ponds is based on the revoked Regulations, the PIC is in accordance with the revised 2015 NZSOLD Guidelines.

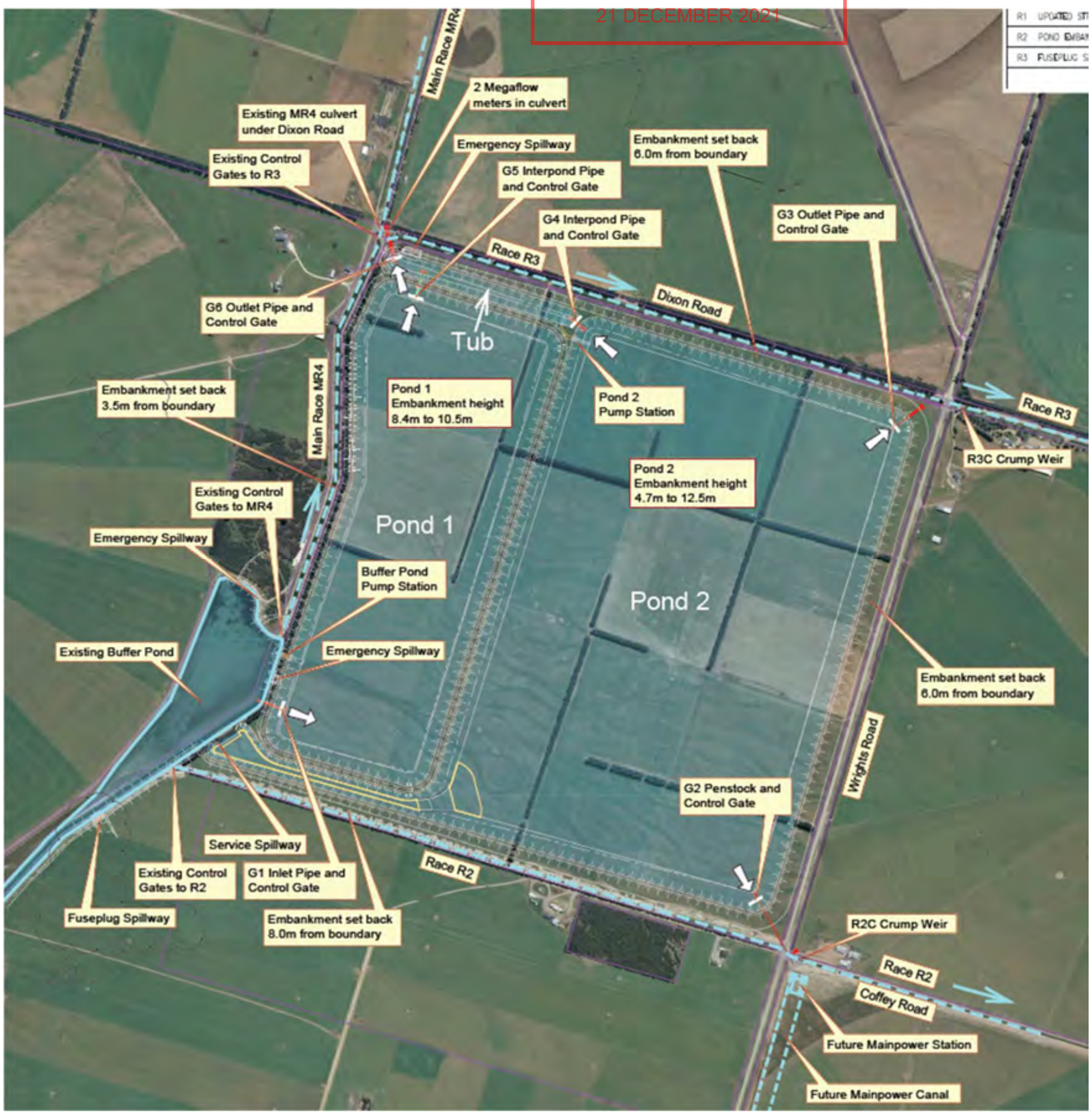


Figure 3: Wrights Road Storage Ponds Layout (from Drawing WIL1125/30/2-R4)

2.0 Description of Proposed Storage Scheme

The current maximum water take from the Waimakariri River is 16.6 m^3/s. This take is restricted when river flow is low. The proposed ponds will provide storage capacity when supply is greater than irrigation demand and conversely augment capacity when irrigation demand is greater than the permitted take from the river. The storage ponds will form an integral part of the Waimakariri irrigation scheme.

2.1 Pond Layout

The foot print of the storage is approximately a square of 1 km x 1 km. The storage is divided by a middle embankment forming Pond 1 and Pond 2 with full supply level of Pond 1 (RL 226.50 m) being 3.7m higher than that of Pond 2 (RL 222.80 m). The ponds are designed for 8.2 Mm^3 storage capacity.

The main components of the Wrights Road Storage Ponds are:

- Six inlet and outlet culvert pipes G1 through G6;
- One service spillway which delivers flows from the Buffer Pond to Pond 2;
- Three emergency spillways protect the Buffer Pond, Pond 1 and the Tub;
- A fuse plug spillway to protect the Buffer Pond from extreme rainfall induced inflow from the main canal;
- The Tub, which conveys flow from Ponds 1 and 2 into the main race MR4 through G6;
- Buffer Pond pump station to fill Pond 1;
- Pond 2 pump station to lift bottom water from Pond 2 into main race MR4 via the Tub and G6;
- A penstock which conveys water from Pond 2 to race R2, via G2, which has the potential to connect to a hydro-power station, if one is developed.
- Two flow measuring crump weirs on distribution races R2 and R3 downstream of G2 and G3 outlets;
- Ultrasonic flow measurement on the G6 culverts to measure flow into the main race MR4; and
- Control System to control gates on pond inlet and outlets to manage operation.

Major construction quantities for the proposed ponds are summarised in Table 1.

Table 1: Major Construction Quantities

Component	Quantity
Bulk embankment fill	1,485,000 m^3
Maintenance strip	9,760 m^3
Geomembrane liner	1,066,800 m^2
500 GSM Geotextile	269,150 m^2
Reinforced concrete	1,290 m^3
Precast Concrete pipes	G1: 1.8m dia. x 18

Component	Quantity
	G3: 1.6m dia. x 26 G4:1.8m dia. x 8 G5: 1.8m dia. x 19 G6: 1.6m dia. x 50
Penstock	146 m

2.2 Filling of Storage Ponds

When excess water is available, it will initially be sent into Pond 1 through gate G1. Pond 1 will have filling priority. When Pond 1 water level approaches the Pond 2 service spillway crest level (RL 222.5) gate G1 will close and the Buffer Pond pump station will pump water from the Buffer Pond to Pond 1. Although priority is given to filling Pond 1, when sufficient flow is available, both ponds will be filled simultaneously as availability of water permits.

Filling of Pond 2 will be by gravity over the service spillway, whenever the water level in the Buffer Pond is higher than the service spillway crest. Accordingly, when water in the Buffer Pond exceeds service spillway crest level, water spills into Pond 2. Pond 2 continues to fill until the level in it matches the full supply water in the Buffer Pond (RL 222.80 m). The service spill into Pond 2 is designed to cope with a maximum flow of 17.0 m^3/s.

The three pumps in the Buffer Pond are vertical shaft pumps, each with a capacity of 335 l/s. If all three pumps are pumping, approximately a total of 1,000 l/s will be transferred into Pond 1. The pumps are designed to operate continuously, and it will take 12 to 13 days to fill Pond 1 to its full supply level of RL 226.50 m. This is 3.7m above both the Buffer Pond and Pond 2 maximum water levels.

Buffer Pond pumps are set to start at different water levels in Buffer Pond in the following sequence:

		Start	Stop
• Pump 1 will start when the Buffer Pond reaches	RL	222.10	222.00
• Pump 2 will start when the Buffer Pond reaches	RL	222.30	222.20
• Pump 3 will start when the Buffer Pond reaches	RL	222.50	222.40

Staging the starting of pumps in this manner means that small amounts of excess water can be captured without excessive start/stop scenarios.

The choice of three pumps allows for one pump to be out of commission without a major impact on the filling operation.

The pumps are designed to pump solids up to 32 mm diameter. A manually cleanable louvered debris screen is provided at the inlet to the wet wells. There is to be no provision for self-cleaning mechanism for these screens.

2.3 Discharging from Storage Ponds

The following discusses discharging irrigation and stock water from the storage ponds when WIL face partial or full restrictions due to low or non-availability of take from Waimakariri River.

The storage ponds are designed to discharge water in three directions.

1. Into the main race MR4 (irrigation water only, as stock water is supplied from the Buffer Pond)
2. Into the distribution race R2 (irrigation and stock water)
3. Into the distribution race R3 (irrigation and stock water)

A summary of the maximum flow in each direction is provided in Table 2.

Table 2: Water Discharge from Ponds

Race	Irrigation Flow (l/s)	Stock Water Flow (l/s)	Total Flow (l/s)
MR4	3,633	-	3,633
R2	1,200 + 4000 Ngai Tahu	60	5,260
R3	3,543	1,000	4,543

Gates G6, G3 and G2 are controlled to deliver set flows (computed by the control system), through flow measuring devices located downstream of the gates; namely two electromagnetic flow meters for G6 (one in each barrel of the two pipes) and a crump weir for each gate G2 and G3 (Ref Drawings WIL1125/30/106, 118 and 127). The gates are opened and closed automatically to control the required flow at each of these discharge points (refer also to Appendix G, Control System).

Pond 2 supplies stored water to main race MR4 and distribution races R2 and R3. Race R2 is supplied via gate G2 outlet, race R3 is supplied via gate G3 outlet and main race MR4 is supplied via gate G4, the Tub and G6. During this period, gate G5 remains closed.

When Pond 2 water level drops to the minimum level, gate G4 is closed and water for MR4 is then supplied from Pond 1 via gates G5, the Tub and G6. This flow is also to be augmented via the Pond 2 pump station.

The Pond 2 pump station pumps draw water from Pond 2 and transfers it into the Tub. At the same time, Pond 2 continues to supply stored water into R2 and R3. The Pond 2 pump station consists of two large inclined pumps each with a capacity of between 1,000 and 1,280 l/s.

When the demand in MR4 is large enough, the two pumps supply up to approximately 2,500 l/s. When the demand is low, only one pump will operate. When the demand in MR4 is less than 900 l/s, water is supplied from Pond 1 only.

The Pond 2 pump station ensures that an even amount of storage can be supplied in all three directions (to MR4, R2 and R3).

The Pond 2 pumps are controlled by demand required from storage and also by level probes, installed inside the Tub. When the water level inside the Tub exceeds a set threshold value, then the pumps are shut off. When demand required from storage is greater than the pump station capacity then both pumps are operated, plus gate G5 is opened releasing water from Pond 1 to maintain level within the Tub. Similarly, when demand required from storage is 2x pumps > demand from storage > 1x pump, one pump is run and Pond 1 augments flow through

G5. When demand required from storage < 1x pump supply water is taken from Pond 1 through G5.

2.4 Emergency Overflow Spillways

There are three emergency overflow spillways and one fuse plug spillway (see Drawing WIL1125/30/2).

One emergency spillway is provided at the Buffer Pond near the existing outlet gates. This allows surplus water from the buffer pond to spill into the main race MR4. Through automation and careful management it is not intended that water will be spilt from the Buffer Pond. However, in the event that control systems fail and water needs to spill, then this overflow spillway will function.

An emergency spillway is provided from Pond 1 back into the Buffer Pond. Pond 1 can overfill if the buffer pond pumps fail to shut off or due to extreme rainfall.

An emergency spillway provides protection to the Tub should either the Pond 2 pumps or G5 inlet from Pond 1 overfill the Tub due to malfunction of the control system or if overfilling is caused by extreme rainfall.

A fuse plug spillway is located in the southern embankment of the Buffer Pond to limit surcharging of the Buffer Pond and Pond 2 from extreme rainfall induced inflow from the Main Canal.

2.5 Automation

The storage pond site will be automated and will be compatible with the existing WIL scheme automation. The control system will communicate with the intake at Browns Rock; and will operate all the gates G1 through G6 and the Buffer Pond and Pond 2 pump stations.

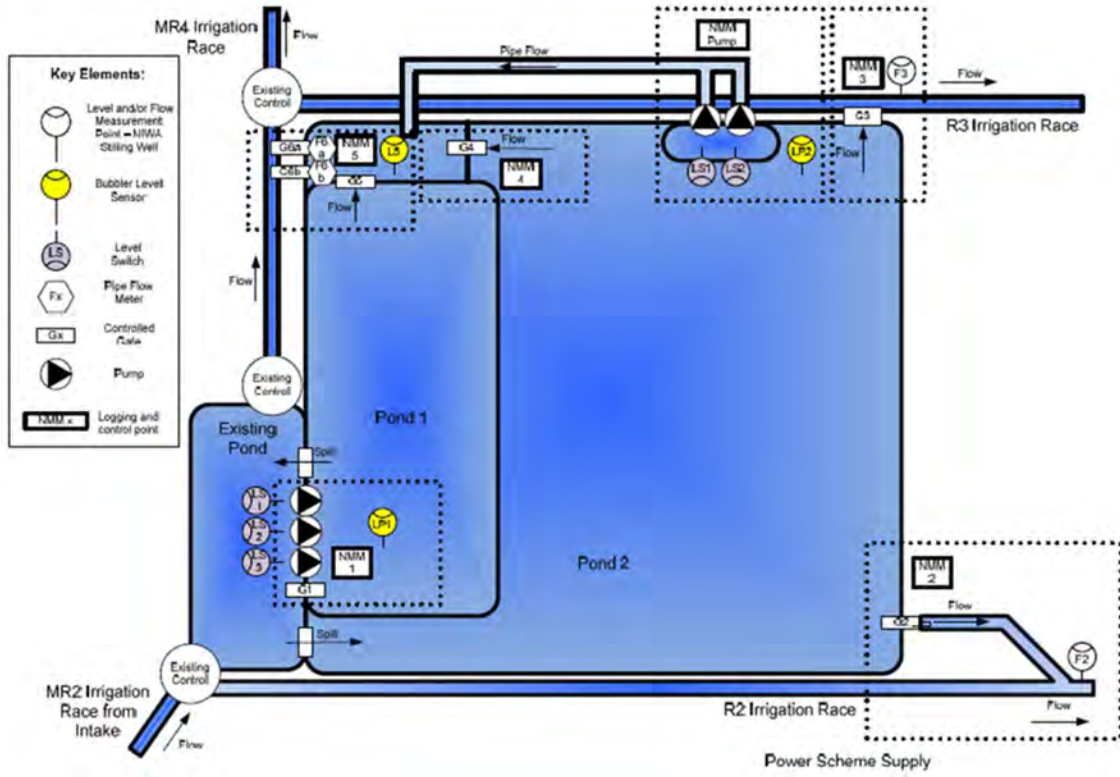


Figure 4: Storage Ponds Control System Overview

An alarm system will notify the race-man of any alarm conditions, failed gates or pumps or any overfilling scenarios. The control system schematic is shown in Figure 4 (refer also to Appendix G).

Communication between all the gates will be real time and continuous. It is proposed that a Wi-Fi system with boosters will provide the continuous feed of information between the gates and pump stations.

The automation will be run from mains power. Cables will be laid to all the automated sites. A power backup source will also be provided at all automated sites. This will consist of battery and invertor, with sufficient capacity to carry out both of the following functions, without requiring recharge:

1. Enable shutdown (if necessary) after prolonged mains power supply failure.
2. Enable opening of those control gates designated as being required for emergency dewatering in case of a dam safety emergency.

2.6 Flow Measurement

Discharges to MR4, R2 and R3 are measured and communicated to the Control System. Locations of the flow measuring devices, which include crump weirs and electromagnetic flow meters are schematically shown on Figure 4. Sections through the ponds showing crest heights and full supply levels are included in Drawings WIL1125/30/21 through 23. Details of the inlet, outlet and inter-pond pipes are shown on Drawings WIL1125/30/101 through 119.

2.6.1 Crump Weirs

Flows in the distribution races R2 and R3 will be measured using crump weirs. They are installed downstream of G2 and G3 (Ref Drawings WIL1125/30/107 & 127). The maximum outflow capacities from Pond 2 delivering into the distribution races R2 and R3 for irrigation and stock water demand are 5.26 m^3/s and 4.54 m^3/s respectively.

The Crump weirs are designed to operate without downstream submergence.

2.6.2 Electromagnetic Flowmeters

Flowmeters have been adopted to measure flow through G6 using two electromagnetic flowmeters, installed in the G6 twin pipe outlet within the reinforced concrete chamber, as shown on drawings WIL1125/30/190 to 195. Electromagnetic flowmeters at G6 were adopted as there is insufficient head available for a weir flow measurement. The two flowmeters are housed in a buried concrete chamber to provide access for maintenance or replacement of componentry over the life of the project.

2.7 Potential Power Station

There is the potential to construct a hydro-power station in the future near the intersection of Wrights and Coffey Roads (Ref Figure 3). The power station could generate power using the head between Pond 2 and the potential future power station. The power station could discharge into a future tail race canal (Ref Drawings WIL1125/30/1 & 2). The power station and tail race canal are outside the scope of this Wrights Road Storage Pond Design.

The penstock at the South East corner, which conveys water from Pond 2 to race R2, via G2, has the potential to connect to a hydro-power station, if one is developed. The penstock has been oversized to accommodate for a future hydro power development. A 1,930 mm OD steel penstock is provided under the Pond 2 embankment and has been specified with a corrosion protection system appropriate for potential future duty as a pressurised pipeline within the foundation of the embankment. In all other respects, the penstock, penstock intake and inlet gate G2 have not been hydraulically optimised for a potential future hydropower station. They have been designed, sized and specified to provide the most economic solution for the primary duty to release water for irrigation, consistent with the quality and service life requirements of the Principal's Requirements.

A tee from the penstock delivers flow to R2 through a 1,350 mm diameter steel branch pipe (Ref Drawings WIL1125/30/103-105).

An impact type energy dissipater is provided at the end of the 1,350 mm diameter branch. The flow into R2 will initially be controlled by the inlet gate. Subsequently, when the power station is constructed, flow into R2 will be controlled by a 1,350 mm diameter knife edge gate valve (Ref Drawing WIL1125/30/106).

2.8 Existing Services

A number of existing services have been identified. The services are generally located parallel to the external boundaries or as standalone power poles. They are outlined in the services Drawing WIL/1125/30/04 and standalone poles at G6 and G2 are identified as requiring relocation.

2.9 Existing Services

A number of existing structures have been identified and are shown on the construction drawings.

2.9.1 Irrigation and Stock Water through MR4

A corrugated steel pipe culvert exists at the intersection of main race MR4 and Dixons road. This has not been designed to carry unusually heavy loads from construction activities. This will cause constraints for construction traffic. The Contractor shall ensure uninterrupted irrigation water supply during irrigation times and stock water supply at all times through MR4 (Ref Drawings WILL1125/30/1, 2 & 118).

2.9.2 Irrigation and Stock Water through R3

R3 starts at the intersection of MR4 and Dixons Road. Water is diverted to R3 through three corrugated steel pipe culverts with control gates. This will cause construction constraints when excavating and installing the two outlet pipes for G6 (Ref Drawings WILL1125/30/1, 2 & 118).

Constraints will also be caused due to the following activities associated with G3 construction:

- Excavation and construction of outlet structure;
- Race bed and side slope stabilisation work; and
- Construction of crump weir, its approach channel and appurtenant structures.

The Contractor shall ensure uninterrupted irrigation water supply during irrigation times and stock water supply at all times through the R3.

2.9.3 Irrigation and Stock Water through R2

Construction constraints are likely due to construction activities associated with construction of G2:

- Installation of 1,930 mm steel penstock under R2;
- Excavation, construction and connection of outlet structure (impact type energy dissipater) with R2;
- Race bed side slope stabilisation work; and
- Construction of crump weir, its approach channel and appurtenant structures.

The Contractor shall ensure uninterrupted irrigation water supply during irrigation times and stock water supply at all times through R2.

3.0 Application of the PIC to the Design Process

The Potential Impact Category (PIC) is a qualitative categorisation of water retaining structures according to the potential consequences of failure. The PIC links to the design and management of dams because the NZSOLD Dam Safety Guidelines use PIC to define the design loads, level of expertise and detail applied to investigation, design, construction, commissioning, surveillance and safety reviews of dams.

The ponds extend over an area of 120 ha located between the Waimakariri River and the Eyre River on the Canterbury Plains. The volume and depth of the ponds are summarised in Table 3.

Table 3: Pond Volumes

Pond	Approximate Wetted Footprint (ha)	Approximate Volume ($x10^6$ m^3)	Maximum Water Depth (m)
1	30	2.0	8.0
2	72	6.2	12.0
TOTAL	**102**	**8.2**	-

Each pond is a large dam as defined by the Building Act (2004), as each pond has greater than 4 m depth of water and greater than 20,000 m^3 reservoir volume.

Dams, with their large volumes of water and related structures, have a special nature because of their scale, the water forces at work and the use of natural ground for forming the major part of the impounding embankment. The design philosophy for dams considers safety of the dam and structures as a very important component of not only the design, but also of the construction and long-term operation as well. The design, construction and operation practices must address hazards that have the potential to impact on the safety of the dam.

The philosophy is that measures are put in place in design, construction and operation in order to prevent a dam breach from occurring. The assessment of consequences from a hypothetical dam breach allows the determination of the Potential Impact Category (PIC) of a dam.

Modern water retaining structures, such as dams, have a very good safety record. This high level of safety is achieved by close attention to and management of the potential hazards and risks to which they may be subjected. An essential part of this process is the identification of the potential impacts or consequences of dam failure. Identification of potential consequences is part of the assembly of information for detailed design, and ensures that the design, construction, operation and maintenance of the dam are focused on safety. This is a well-established, standard practice for dams.

The storage ponds are off river and located about 6 km away from and about 20 m above the Waimakariri River. The main canal, flowing into the buffer pond, would intercept and be surcharged by surface runoff upstream of the buffer pond in an extreme rainfall event. Protection from this hazard is addressed with the buffer pond emergency spillway (Refer to Drawings WIL1125/30/2 and WIL1125/30/122) and the fuse-plug spillway located near the upstream limit of the buffer pond (Refer to Drawings WIL1125/30/2 and WIL1125/30/147). There are no other sources of local tributary floodwater to cause an overtopping of Wrights Road pond embankments.

With no external flood threats the potential failure modes for an off-river earth embankment storage would be:

1. Large leak through lining defect leads to piping or washing out of fill and sloughing of the embankment
2. Overfilling of pond leads to overtopping failure
3. Seismic induced settlement leads to either piping (large leak) or overtopping failure
4. Higher elevation pond (Pond 1) breach leads to overtopping of lower pond (Pond 2)

The proposed storage scheme consists of two separate reservoirs. For the purpose of the PIC assessment it is assumed that the breach mechanism for Pond 1 is internal leak through a lining defect and piping failure of a length of embankment. It is assumed that failure could occur at any location on the Pond 1 embankments due to earthquake damage or a localised weakness in the embankment. Overtopping failure was used as the breach mechanism for Pond 2 resulting from Pond 1 failure into Pond 2 and causing Pond 2 to overtop and fail. These failure mechanisms are chosen based on documented examples of historical embankment dam failures. Details of the hypothetical breach and outflow modelling are included in Appendix H.

Four embankment failure locations, located for the greatest impact on nearby houses, were analysed and flood inundation maps were developed using a two-dimensional computational hydraulic model. A full description of the hydraulic modelling study is provided in Appendix H. The four assumed failure locations are labelled as “East”, “South”, “North” and “West” and the resulting flood inundation from embankment failure at these locations are also provided in Appendix H.

There are currently two methods for determining the Potential Impact Classification (PIC) of dams in NZ. Both require an assessment of consequence of dam breach. The methodologies are contained in:

- The NZSOLD Dam Safety Guidelines (2015) (NZDSG); and
- The Building (Dam Safety) Regulations 2008 supporting the Building Act 2004.

The Building (Dam Safety) Regulations were released in 2008, providing amongst other things, the methodology for PIC determination to be applied to the Dam Safety Scheme, which was to take effect from 1 July 2014. These regulations were revoked in 2015, however, the 2015 NZDSG, adopt the same methodology for PIC determination. Accordingly, the PIC assessment described below remains appropriate, despite the revision of the NZDSG and the revocation of the regulations on which the PIC assessment was based.

Dams are classified as high, medium or low PIC, depending on the potential impact of the dam breach outflow. Both the NZDSG and the Building Regulation methods for determining PIC are strongly influenced by the likelihood of fatalities resulting from breach outflow. This in turn is dependent on the population at risk (PAR), the amount of warning that is provided to the people exposed to dangerous flooding and the severity of the flooding.

During August 2012, a topographic survey of households within the dam breach flood flow path and surrounding area was undertaken. An assessment of the threshold level at which water can start to inundate each property was obtained by surveying the doorstep level of each household.

Appendix H shows the location of each of the households surveyed. In total, 270 households were surveyed. This survey data was used to determine the population at risk (PAR) for the four dam breach scenarios modelled. A summary of the PAR evaluated for each dam breach scenario is provided in Table 4 – refer to Appendix H for further details of the PAR assessment.

Table 4: Population at risk (PAR) summary

Parameter		Eastern Breach	Southern Breach	Northern Breach	Western Breach
A	Area of flood zone (km^2)	73	59	66	15
B	Number of households within flood zone	176	75	198	50
C	Households at risk [1]	40	19	35	1
D	Number of people per household	2.5	2.5	2.5	2.5
E	Number of additional people in flood zone (farm workers, people on footpaths or in vehicles, etc) [2]	7	6	7	2
F	Population at Risk (PAR) [3]	107	54	95	4

Notes:
(1) "At risk" is defined as a property in the dam breach flood zone and inundated by 0.5 m or greater
(2) Based on assumption of 1 person per 10 km^2 within the area of the flood zone
(3) PAR = (C x D)+E

Based on the PAR and dam breach hazard maps (refer to Appendix H), Table 5 summarises the assessed PIC for each dam breach location based on both Building (Dam Safety) Regulations, 2008 and the 2000 NZDSG. Full details of the PIC assessment are provided in Appendix H.

Note that the maximum dam breach peak outflow for the type of breach analysed is approximately 2,500 m^3/s. This is less than twice the mean annual flood in the Waimakariri River. Peak flow entering the Waimakariri River would be significantly less than the peak breach outflow due to routing effects in the overland flow before flow enters the river. Accordingly, a breach outflow would not cause significant flooding in the Waimakariri River downstream with either normal low flow or flood flow in the river.

The Building (Dam Safety) Regulations were scheduled to come into force in 2014 and the 2000 NZDSG were revised to align with the Building Regulations with respect to PIC assessment.

The NZDSG set out the standards for design construction and operation of large dams such as the Wrights Road Storage ponds and this in turn would be determined by the PIC assessed in accordance with the Guidelines. The NZDSG set out the design, construction and operation standards for large dams in New Zealand.

In anticipation of the increase in building of homes in the Wrights Road Storage Ponds inundation area, we consider that the Potential Impact Classifications determined in accordance with the Building (Dam Safety) Regulations should be adopted for determining the standards for design and construction of the Wrights Road Storage ponds set out in the NZDSG.

Table 5: Assessed PIC for Modelled Breach Scenarios

Category	Eastern Breach	Southern Breach	Northern Breach	Western Breach
Building (Dam Safety) Regulations (2008)	HIGH	HIGH	HIGH	MEDIUM
NZSOLD Dam Safety Guidelines (2000)	HIGH	MEDIUM	MEDIUM	LOW

The PIC is consequence driven and the consequences relate to breach peak outflow, volume and development in the area inundated. This is different for the west embankment from the other, north, east and south embankments particularly with respect to breach peak outflow and volume. This is because only Pond 1, which has less volume than Pond 2, is breached in that scenario. In the other scenarios, Pond 1 breaches into Pond 2 in turn causing Pond 2 to breach, providing breach outflow from the contents of both ponds.

A fundamental principle proposed for the Wrights Road Storage Ponds is that different Potential Impact Classifications can be applied to discrete lengths of the pond embankment, because the downstream consequences of breach vary depending on the section of the embankment breached. It follows that breach of the west embankment, and the embankment north of the tub, would have less consequence than breach of any of the other embankments. Accordingly, we submit that for the proposed Wrights Road storage ponds, discrete sections of the embankment can be assigned a different PIC as is done with large canal embankments. This then results in different standards for design and construction of discrete sections of the pond embankments, as summarised in Figure 5.

Figure 5: Summary of PIC assessment for embankments of proposed Wrights Road Storage Ponds

The PIC evaluation is based on the current distribution of houses in the district. In the future, further development could occur and housing density increase. If so, the population at risk around the Wrights Road Storage Ponds could increase with a future PIC evaluated as high also for the western embankment. The design intent is that such an outcome should have relatively insignificant impact on operation of the ponds due largely to the features of the design (e.g. free draining outwash gravels forming the foundation and used in forming the pond embankments along with the properties (strength and elongation) of the lining adopted). Assessment of earthquake shaking on performance of the pond embankments, particularly their ability to contain the water stored if subjected to such earthquake shaking, is addressed further in Section 7.

Appurtenant structures are defined in the Building Act and the NZDSG as those structures required for the safe containment and control of the reservoir contents under any, and all, loading conditions. The 2015 NZDSG provide more detailed guidelines for appurtenant structures than the 2000 NZDSG or the Building Act and are also referenced for purposes of verifying compliance and dam safety requirements.

The principal appurtenant structures that form part of the storage scheme and have dam safety-related functional requirements are the emergency spillways and the control gates/associated intake structures. The dam safety critical function of the emergency spillways is as described in Section 2.4. The two pump stations do not have a dam safety critical function as they are not required for emergency dewatering and overtopping protection is provided by emergency spillways in the event of pump malfunction (failing to stop).

The dam safety critical function of each control gate is shown in Table 6 below.

Table 6: Dam Safety Critical Functions of Control Gates

<table>
<tr><th>Gate</th><th>Emergency dewatering (open function)</th><th>Overtopping protection (open function)</th><th>Isolate culvert in case of culvert leakage (close function)</th></tr>
<tr><td>G1</td><td>No</td><td rowspan="6">No</td><td rowspan="6">Yes</td></tr>
<tr><td>G2</td><td>Yes</td></tr>
<tr><td>G3</td><td>Yes</td></tr>
<tr><td>G4</td><td>No</td></tr>
<tr><td>G5</td><td>Yes</td></tr>
<tr><td>G6a/b</td><td>Yes</td></tr>
</table>

It can be noted that a failure of any of the control gates while closed against reservoir head will not result in the unsafe release of the reservoir contents as the size of the openings and capacity of the inlets will in all cases control the rate of discharge to levels that do not have unsafe consequences[1].

Generally, all control gates that discharge to a location outside of the ponds (G2, G3 and G6) are designated as having an emergency dewatering function. This requires the gates to be capable of opening following a SEE event and any associated aftershocks. Gate G5 between Pond 1 and the Tub is also required for emergency dewatering.

None of the control gates have a dam safety function to prevent overtopping. Gates G1, G5 and G6 are not required to prevent this function as Pond 1 and the Tub are protected by emergency spillways. Pond 2 overtopping is prevented by the emergency spillway and the fuse plug spillway in the Buffer Pond, which is hydraulically connected to Pond 2 above RL 222.5 m, as discussed in Section 4.4.5 below.

None of the culverts operate under significant internal pressure and any leakage from the culverts into the dam or foundation in the event of culvert damage would not result in internal erosion or instability of an embankment. The culvert haunching and backfill is robustly and specifically designed to control any seepage, as detailed in Section 7.10 below.

All gates are considered to have a dam safety critical function to close for culvert isolation. This is based on the potential need to close the gates in the event of pipe dislocation following an SEE earthquake. Gates 4 and 5 may also serve an isolation function if the tub or one of the G6 gates were damaged.

If a hydropower station is connected to penstock G2 in future, its intake gate will be required to have a safety isolation function. Gate G2 is not intended to be designed for that purpose at present and would require modification or replacement in future if a hydropower station is added.

[1] Referenced from Section E.4.1, Appendix E of the 2000 NZDSG, which can be taken to also apply to the meaning of uncontrolled discharge in the 2015 NZDSG.

All gates with a dam safety critical function are required to have an appropriate level of redundancy in the design of their operating systems and a suitable emergency backup power supply.

4.0 Design Philosophy

4.1 Location of the Ponds

The ponds are located on land previously purchased by WIL to accommodate the ponds. They are located North West of the intersection of Dixons Road and Wrights Road; some 20 m above the Waimakariri River level. The design maximises the stored volume within the constraints of land area and construction cost.

The pond embankments are set back from land boundaries and the adjacent roads as follows:

- 4 m setback from the Western boundary; parallel to main race MR4;
- 5.4 m setback from Northern and Eastern road boundaries, parallel to Dixons and Wrights roads respectively; and
- 8 m setback from the Northern edge of race R2.

4.2 Design of Ponds

Materials for construction of the pond embankments are to be sourced from the inverts of the ponds as availability permits. Design has set the levels of the constructed pond inverts in order to source sufficient materials for the embankments of each pond on a pond by pond basis.

Lining of the ponds is necessary to control seepage due to the high permeability of the embankment and foundation material. Two lining systems were considered; a three layered composite earth lining and synthetic geomembrane. A synthetic geomembrane lining has been adopted due to:

- Limited availability of silt on site as a low permeability lining material;
- Insufficient wave protection rock of adequate size in the foundation gravels;
- Potential leakage from a composite soil liner with a thin low permeability silt layer; and
- Superior sealing and resultant low leakage achievable with a geomembrane.

Selection of geomembrane for use in this application is addressed subsequently in Section 7.5.

Construction of the inlet, the ponds and the outlets has been engineered so that construction is largely an earthworks exercise. Adequate control of seepage, provision of filters and compatibility of soil materials is fundamental to the successful performance of the project. Design is based on a finite number of test pits and soils testing investigations. It is important that the embankment materials are placed and compacted in accordance with the drawings and specification and that quality management of construction is well set up, managed and documented in order to demonstrate that this is so. Quality management of excavation and placement is required in order to check, assess and adjust as necessary based on laboratory and field test results during the construction where materials, for foundation or embankments differs from initial test pit results on which this design is based. Construction shall be in accordance with the NZDSG, which for High PIC requires full time supervision and control testing of construction should be a mandatory requirement.

The criteria adopted for design of the Wrights Road Storage ponds are summarised in the following sections.

4.3 Flood Hydrology

For a Medium PIC the NZDSG recommend:

- Design to withstand the 1:1,000 to 1:10,000 annual exceedance probability (AEP) flood in the Waimakariri River. This implies that the ponds must also be able to absorb or pass a 1:1,000 to 1:10,000 AEP rainfall event; and

For a High PIC the NZDSG recommend:

- Design to withstand the 1:10,000 AEP to the Probable Maximum Flood (PMF) flood in the Waimakariri River. This implies that the ponds must also be able to absorb or pass a 1:10,000 AEP to PMP (probable maximum precipitation).

The ponds are located six kilometres from the Waimakariri River and are approximately 20 m above the river bank level. This is significantly above extreme flood levels and beyond the flood plain of the Waimakariri River.

Preliminary flood inundation mapping by Waimakariri District Council indicates that major flooding in the Eyre river catchment north of the ponds would extend to the North embankment of the ponds and would not endanger the integrity of the ponds. The ponds are also required to withstand a rainfall event with a 1:10,000 AEP or with the PMP.

The ponds occupy almost the entire site so that most precipitation will fall into the ponds. In turn the ponds will act as detention basins, and the freeboard of the ponds allows them to accommodate approximately 1.5 m of precipitation if the ponds are assumed to be at the maximum operating level during the rainfall event. It would therefore be an extreme rainfall event when the ponds are full and overtopping of embankments becomes a potential risk.

The NIWA program HIRDS was used to determine precipitation of the area for up to a 1:100 AEP. This was extrapolated for more extreme events to obtain values summarised in Table 7. The Probable Maximum Precipitation (PMP) was calculated in accordance with Tomlinson and Thompson (Thomlinson and Thompson, 1992).

Table 6 shows that the ponds can contain well in excess of a 1 in 10,000 AEP rainfall event. To prevent overtopping of embankments, the inlet gate can be closed and the inter pond and outlet gates opened to discharge water. The emergency spillways are also available to prevent pond overtopping.

The canal flowing into the buffer pond would intercept and be surcharged by surface runoff upstream of the buffer pond in an extreme rainfall event. To prevent erosion of the toe of Pond 2 embankment due to overtopping of the buffer pond during extreme rainfall events, provision for the main canal embankment to be raised to RL 224.0 m over a reach of 230 m upstream of the S1 service spillway has been included (refer to Drawing WIL1125/30/21). This will divert overtopping flood water away from the toe of Pond 2, and prevent potential erosion of the Pond 2 embankment toe. Further protection is provided with the buffer pond emergency spillway, and also the fuse-plug spillway in the main canal immediately upstream of the buffer pond (Refer to Drawing WILL1125/30/11).

Table 7: Extreme Rainfall Depths (mm) for Wrights Road Storage Ponds Site and Rise of Water Depth Due to Buffer Pond Pump Stop Failure

AEP	Duration				
	60 min	2 hr	6 hr	12 hr	24 hr
1:1,000	102	130	186	236	296
1:5,000	180	220	300	364	442
1:10,000	228	276	368	438	526
PMP	-	-	-	-	400

4.4 Hydraulic Design of Inlet, Outlets, and Inter-pond Culverts and Spillways

4.4.1 Pond 1 Inlet G1

Pond 1 is initially filled by gravity via the inlet gated culvert G1 (refer to Figure 6). G1 has a capacity of 3.0 m^3/s with a minimum differential head of 0.1 m. The priority of filling is to Pond 1 as it has the facility to augment water in the main race MR4.

Gated inlet culvert G1 is a single barrel 1,800 mm diameter with concrete inlet and outlet transitions shown on Drawing WIL1125/30/101. G1 is fully automated, managed by the control system to maintain the water level in the Buffer Pond. This ensures that there is always sufficient water in the Buffer Pond to supply the main race MR4. Initially, Pond 1 is filled by gravity until it approaches Buffer Pond water level.

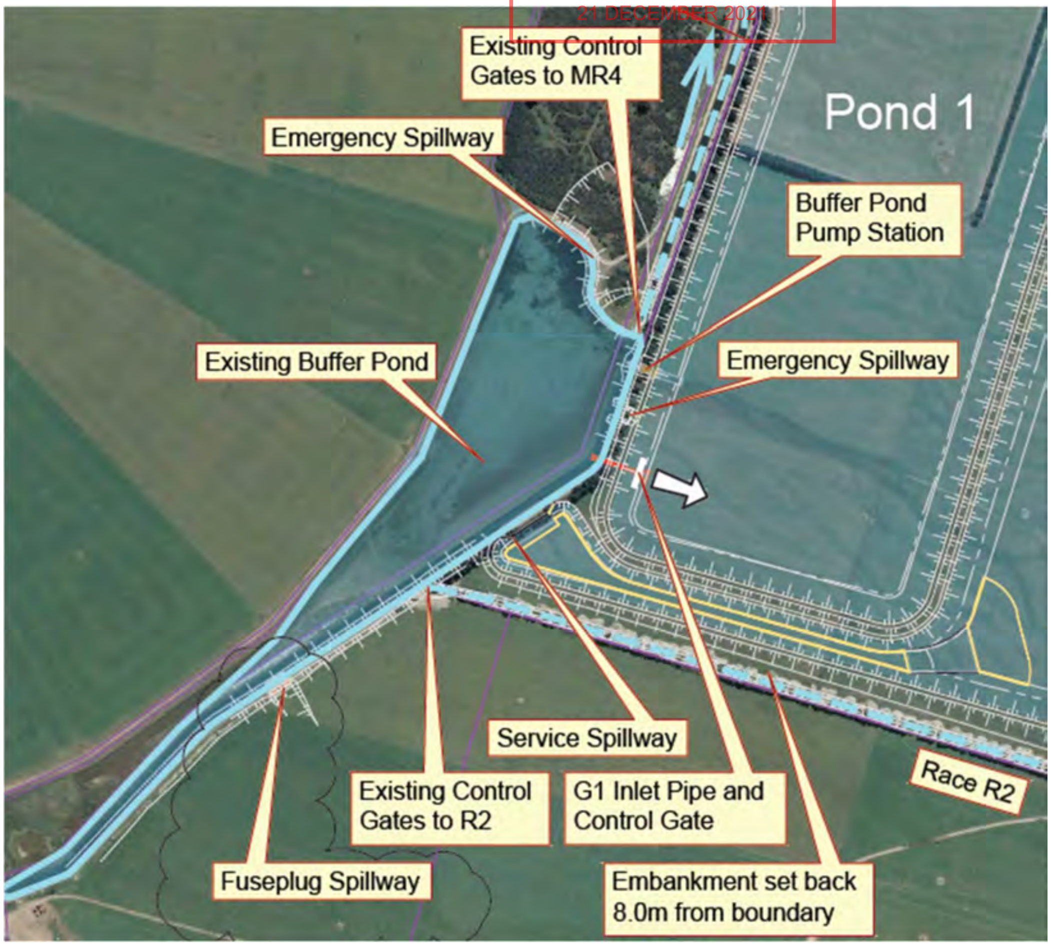


Figure 6: Layout at Buffer Pond

Then, gate G1 will close and the Buffer Pond pump station will pump available flow from the Buffer Pond into Pond 1 until it reaches full supply level RL 226.5 m or as long as excess water is available in the Buffer Pond.

4.4.2 Inlet Service Spillway to Pond 2

The service spillway to Pond 2 is designed to pass a maximum flow of 17.0 m^3/s. It is a concrete lined channel. A concrete stilling basin is included to dissipate energy when the upstream section of Pond 2 (southern Tub) is empty (Ref Drawing WIL1125/30/124 through 126).

Then water flows along the 245 m long conveyance channel between spillway and Pond 2. The invert of the conveyance channel is protected with 0.4 m thick riprap of Zone 1 material. At the end of the conveyance channel a 65 m long reinforced concrete chute, 0.2 m thick, passes water from RL 220.10 m down to the 55 m long bed protection riprap of Pond 2 at RL 213.54 m (refer Drawing WIL1125/30/123).

4.4.3 Log Boom

Trees have become established along the true left of the main canal, upstream of the buffer pond. In inclement weather conditions, there is a risk of the floating debris entering Pond 2 via the Buffer Pond and service spillway, potentially impacting on the safe operation of the scheme. To mitigate this risk a log boom has been proposed across the main canal, immediately upstream of the buffer pond, as shown on drawing WIL1125/30/187. The log boom design is outlined in the following sub sections:

4.4.3.1 Design Considerations

The log boom design is based on the following design considerations and assumptions:

- The anchor ground conditions has been assumed to comprise of medium to dense sandy gravel based on the test pit investigation shown on Figure 12. Relevant logs and test data are included in Appendix N.
- The elevation of the anchor point on the true right hand side (RHS) of the canal is at RL 224 m.
- The anchor point elevation on the true left hand side (LHS) has been assumed to be at RL 223 m.
- Access to the LHS of canal will be provided to enable anchoring.
- The log boom is intended to capture floating debris only, not submerged debris.
- Floating debris is expected to comprise of tree logs, with a maximum dimension of up to 5 m long and 1 m thick.
- Maximum flow velocities within the canal at the log boom location are expected to be 0.6 m/s.
- Water level fluctuation within the pond will not be more than 2 m i.e. RL 220.8 to RL 222.8 m.
- Wave loads on the boom are negligible and have not been considered.
- A minimum safety factor of 2.0 is adopted for the load capacity design of the log boom and its anchorage.

4.4.3.2 Log Boom Design

The log boom will be installed at about a 40° angle across the canal, as shown on drawing WIL1125/30/187. This will direct debris towards the true right bank of the canal, where it can be removed by an excavator.

The boom details are shown on drawing WIL1125/30/187. It consists of two 100 mm dia PVC pipes filled with expanded polystyrene, which are held together by stainless steel brackets at about 2.2 m spacing. The steel brackets are connected together by a 16 mm diameter backbone wire rope. The backbone wire rope is extended across the canal and connects to the anchor points on either bank. As per the standard practice, 1% of extra rope length is allowed for each 10 m up to a maximum of 5%. The total length of the wire rope will be about 31 m.

The drag load on the boom from a 1 m thick debris mat is estimated to be about 2 kN/m. This would generate a maximum tension of about 50 kN in the wire rope. A stainless steel wire rope of 16 mm nominal dia (SWR 6x36 type as included in Appendix N) is proposed which has a Minimum Breaking Load (MBL) of 143 kN. The wire rope capacity is considered 80% of the

MBL (114 kN) to allow for the connections. This would give a safety factor of 114/50 = 2.3 , which is considered acceptable (> 2.0).

The anchor points are proposed to be of Manta Ray type anchors. The Manta Ray anchors are driven into the ground and after reaching the required depth, the driving tool (drive steel) is removed. The anchor is then tipped and proof tested to present its bearing area to the soil. This is called “load locking” and provides an immediate proof test of each anchor. Assuming the anchor ground condition comprises of medium to dense sandy gravel, it is expected that the Manta Ray MR-SR anchors will provide a minimum ultimate load capacity of about 100 kN (see Appendix N). This would provide a safety factor of at least 2.0 for the anchorage.

4.4.3.3 Design Confirmation on Site

The following design assumptions should be confirmed on site prior to the boom fabrication:

- The elevation of the anchor points are at RL 223 m and RL 224 m at the LHS and RHS of the canal, respectively.
- Final length of the boom (including wire rope) should be confirmed on site following confirmation of the boom anchor points.
- The proposed Manta Ray anchors can provide an ultimate anchor capacity of at least 100 kN. The anchor capacity should be tested on site with an anchor locker.

4.4.3.4 Operation and Maintenance Requirements

- The log boom will require regular clearance of accumulated debris to prevent excessive build up and stress forming on the boom.
- The log boom will require regular visual inspection, at least on an annual basis. The PVC pipes, backbone wire rope and the shackle connections should be checked for any sign of deterioration or corrosion. Any components that are damaged sufficiently to impair the performance or structural integrity of the boom will need to be replaced as soon as possible.
- All wire rope terminations will have heavy duty stainless steel thimbles. The stainless steel thimbles at the anchor points are in contact with the galvanised mild steel shackles and thus, the thimble and wire rope are wrapped by Denso Tape to ensure no galvanic reaction would occur between the mild steel and the stainless steel components. The connection points should be inspected annually to check the Denso Tape, shackle and steel rope conditions.
- The Manta Ray anchors on the LHS and RHS of the canal should be inspected (at least annually) to ensure their load capacity is not compromised. Due to the angled orientation of the log boom with respect to the canal, the LHS anchor point is expected to take more rope tension than the RHS anchorage. The LHS could be partially or fully submerged under high canal water levels and the inspection may need boat access.
- A concrete pad is provided on the RHS bank of the canal for temporary placement of the floating debris. It is anticipated that the debris will be removed soon after drying out and that debris accumulation will be avoided at the concrete pad.

4.4.4 Emergency Overflow Spillways

An emergency over-flow spillway is provided from Pond 1 into the buffer pond. This is required to protect embankments of Pond 1, should the Buffer Pond Pump Station fail to stop and as a consequence over fills Pond 1. The emergency spillway discharges water into the Buffer Pond (refer to Drawing 1125/30/120).

A grassed surface, over-flow emergency spillway is also provided for the Buffer Pond to discharge up to 12 m^3/s into MR4 should the Buffer Pond water level rise above RL 223.20 m (refer to Figure 6 and Drawing 1125/30/122).

The third emergency over-flow spillway is provided from the tub to the distribution race R3. This is required to protect the Tub, should the Pond 2 Pump Station pumps fail to shut off or gate G5 remains fully open due to a control system failure, discharging water from Pond 1 to the Tub, then this overflow spillway will function. This spillway has a maximum capacity to discharge 14 m^3/s (refer to Drawing 1125/30/121).

The emergency spillways discharging from the Pond 1 and the Tub are geomembrane lined channels. The emergency spillways are detailed on Drawings WIL1125/30/120 to 122.

4.4.5 Fuse Plug Spillway

A fuse plug spillway, 20 m long (refer to Figure 6 and Drawing WIL1125/30/1, 2, 3 and 147), is provided to limit overfilling of the Buffer Pond and Pond 2 should extreme rainfall runoff enter and overload the main canal discharging into the Buffer Pond. The extreme rainfall runoff into the main canal from the upslope catchment, is estimated to be greater than 20 m^3/s for the 24 hour PMP event. The fuse plug spillway will fuse if the Buffer Pond water level rises above RL 223.40 m as a consequence of inflow exceeding the buffer pond emergency spillway (which has a capacity rising to 12 m^3/s as the Buffer Pond water level rises to RL 223.40 m) plus inflow to Pond 2.

The fuse plug will discharge the upper reservoir volume of Pond 2 (between RL 222.5m and RL 222.8m) and the Buffer Pond above the natural ground level at the fuse plug (approx. RL 222.0 m) along with continuing rainfall runoff. The fuse plug has internal pipes which, when the Buffer Pond water level rises to RL 223.40 m, will flow to induce erosion of the embankment downstream shoulder fusing the embankment.

The estimated peak outflow from the fuse plug spillway is approximately 60 m^3/s with a total volume released in the order of 700,000 m^3. This outflow will discharge downslope and result in shallow, overland flooding to the south-east of the ponds as outlined in the flood inundation map shown in Figure 7. The flood inundation shown in Figure 7 was developed with a two-dimensional topographic model, which used the same model parameters as the dam break model described in Appendix H. The flood extent shown in Figure 7 is based on no background flooding from rainfall runoff. As the fuse plug will only operate in extreme rainfall conditions the outflow from a fuse of the plug will be much more dispersed as it will be superimposed on rainfall induced sheet flow over the area shown in Figure 7.

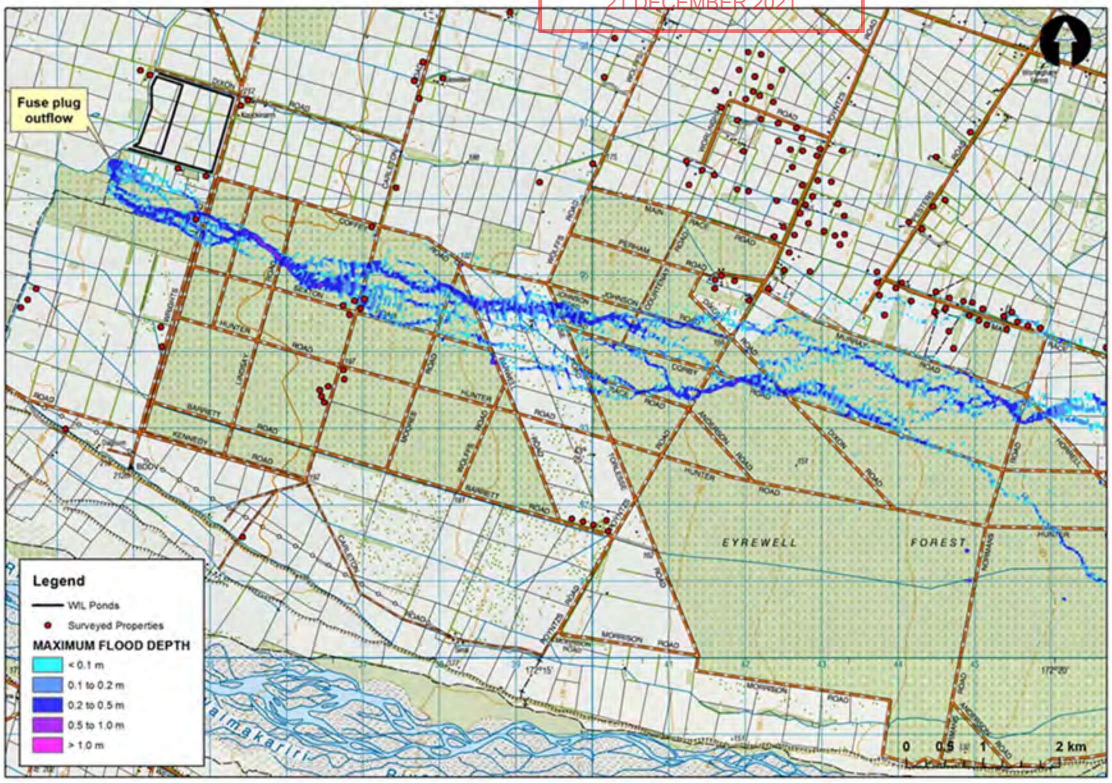


Figure 7: Fuse Plug Outflow Inundation

4.4.6 Inter-pond Gated Culverts

There are two inter-pond gated culverts sized to convey inter-pond flows managed by the control system. They are G4; discharging water from Pond 2 to the Tub and; G5 discharging water from Pond 1 to the Tub. Both are single barrel gated culverts shown on Drawings WIL1125/30/113 to 116.

The inter-pond downstream transitions are designed to prevent flow eroding the pond lining ballast when the ponds are initially empty. As with the gated inlet to Pond 1, the control system will be programmed so that the start of filling an empty pond will be limited to not less than 30 minutes. This is so that the initial flow does not erode the pond bottom lining ballast. As with the inlet, time to initiate flow will be programmable so that during commissioning, the rate of increase in initial flow into the ponds when empty can be adjusted based on initial observed performance.

4.4.7 Pond Outlets

As explained previously in Sections 1.0 and 2.7, allowances are included for;

- Future development of Ngai Tahu land; and
- Future hydro-power development.

The pond outlets are designed for capacities shown in Table 8. The outlet to MR4 releases only the irrigation discharge of 3.63 m^3/s from the Tub. Outlets to R3 and R2 discharge both irrigation and stock water of 4.54 m^3/s and 5.26 m^3/s, respectively. The R2 discharge includes 4.0 m^3/s for Ngai Tahu consented flow.

Table 8: Pond Outlet Capacities

Outlet	Description	Capacity (m^3/s)				
		Stock Water	Peak Irrigation		Potential future hydro-power	Total
			Present	Ngai Tahu		
G2	Outlet from Pond 2 to R2 & Hydro Power	0.06	1.2	4.0	14.0	5.26/14.0
G3	Outlet from Pond 2 to R3	1.0	3.54	-	-	4.54
G6	Outlet from Tub to MR4	-	3.63	-	-	3.63

The maximum irrigation and stock water discharge with Ngai Tahu flow to R2 is 5.26 m^3/s. The future potential hydro-power discharge 14.0 m^3/s (max) would also be through G2. As explained in Section 2.7, G2 discharge into R2 will initially be by upstream gated control. Subsequently, as part of a hydro-power development, control would move downstream to the valve installed at the outlet to R2 (Ref Drawing WIL1125/30/106).

4.4.8 Flow Measurement

Two crump weirs are included on both R2 and R3 to measure the flow release from Pond 2. The crump weirs are designed to measure the canal flows as shown in Table 8 for both R2 and R3.

Flow from the Tub to main race MR4 is through outlet G6 and is measured by an electromagnetic type flow-meters installed in the G6 culverts, upstream of the outlet. Reasons for selection of electromagnetic flowmeters on G6 are detailed in Section 2.6.2.

4.5 Seismic Hazard Assessment

Damwatch subcontracted GNS Science (GNS) to prepare two reports concerning the seismic hazard of the Wrights Road Storage Ponds site. The first report (June 2012) presents the current knowledge of seismic hazards between Oxford and Darfield in Canterbury. The report provides the latest thinking on fault locations, local Canterbury seismicity and the implications on the National Seismic Hazard Model (Stirling et al, 2008), (Stirling et al, 2010). This report is provided in Appendix K. The second report (October 2012) provides ground motion recommendations for use in design of the pond embankments and ancillary structures. This report is provided in Appendix L. The following discussion is based on these reports as it relates to the seismic hazard assessment to Wrights Road Storage Ponds.

In late 2015, GNS reviewed the impact on earthquake probabilistic hazard results of entertaining shorter recurrence intervals for the Hororata Fault and four other fault sources in the region; the potential impact of "hanging wall" effects on the deterministic scenario hazard result for the Hororata Fault and comment on horizontal accelerations adjacent to intersections of reverse/strike-slip faults. The finding of this review with respect to seismic hazard and design of the pond embankments is described subsequently in Section 4.5.6.

4.5.1 Active Faults

There are two main types of faults in the Oxford-Darfield area. Large active faults located some distance to the west capable of magnitude (M) ~8 earthquakes. This includes the Amberley Fault Zone and Alpine Fault. The other type are tectonic structures located in the Canterbury plains which trend northeast-southwest (NE-SW) and east-west (E-W). For example the Greendale Fault trends E-W as shown on Figure 8 (reproduced from the GNS report provided in Appendix K). These tectonic structures have a low probability of an earthquake occurring, but could generate high ground accelerations if they move, due to their close proximity to the ponds site. A summary of active faults mapped in the Oxford area on GNS database and discussion of ground motions measured from recent earthquakes in Canterbury is presented in Appendix C.

The most significant known active tectonic structure close to the project site is the Hororata Fault and its associated anticline (Racecourse Hill Anticline) to the south of the proposed pond site. The Hororata fault is capable of M=7.2 earthquake according to the National Seismic Hazard Model (Stirling et al. 2008, 2010) and has an average recurrence interval of 17,000 years (Appendix K). The active fault database (GNS) shows the Hororata Fault approximately 13 km south of the irrigation ponds. The GNS Christchurch Geologic Map (2008) shows the northern tip of Racecourse Hill Anticline approximately 2.8 km southeast of the irrigation ponds.

The Springbank Fault located approximately 20km northeast of the irrigation ponds on GNS active fault database was also considered as an earthquake scenario in evaluating pond embankment performance. The Springbank fault is estimated to be capable of M=7.0 earthquake (Stirling et al. 2008, 2010).

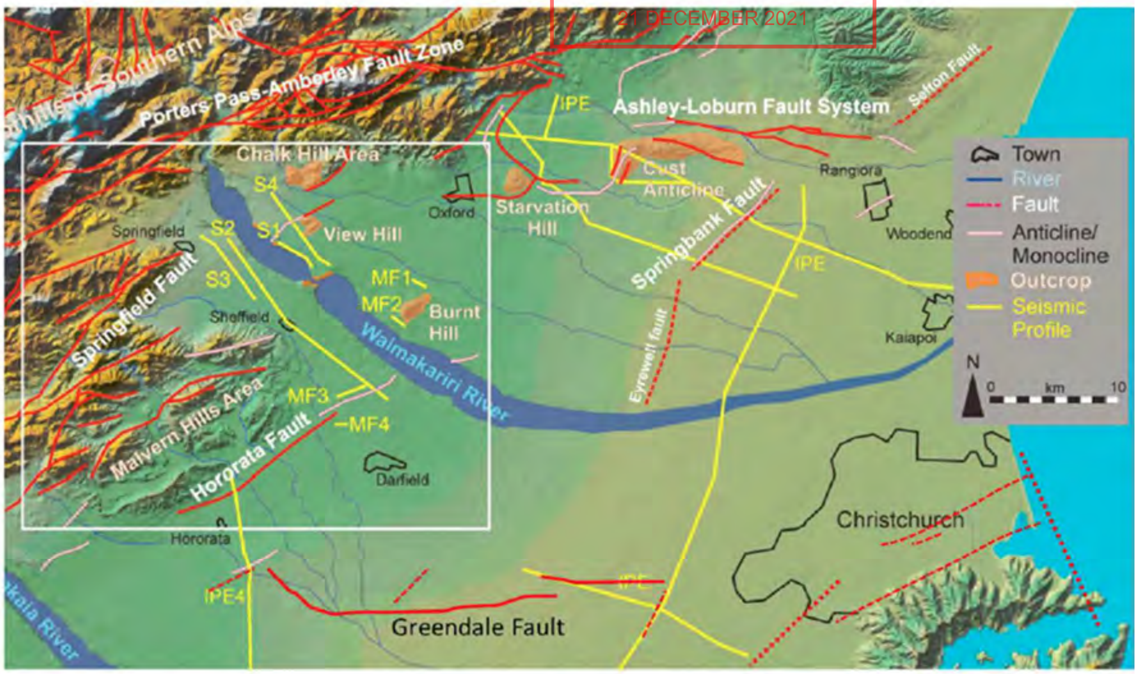


Figure 1 Map of the northwest Canterbury Plains (modified from Dorn et al. 2010) showing the location of seismic lines (yellow lines) and mapped faults and folds (red and pink lines, GNS active faults database; dashed lines indicate locations of known significant buried structures inferred from seismic lines, geomorphology and/or recent seismicity). White box denotes extent of zoomed area in Figure 2.

Figure 8: GNS June 2012 Report Figure 1 showing faults and seismic lines in Canterbury

4.5.2 Surface Fault Rupture

GNS report (Appendix K) cites no known surface fault traces directly through the proposed Wrights Road Storage Ponds site. As shown in Figure 2 of their report (Appendix K) the closest mapped fault is the Racecourse Hill Anticline to the southeast of the project and identified on the northern river bank. Tectonic structures in the Canterbury region are partially traceable in the foothills to the west and masked by thick alluvium gravels (outwash) in the plains.

The Greendale Fault was previously unidentified prior to the Darfield earthquake, 20 September 2010 and caused surface rupture during that event. Post-earthquake investigations performed in May 2011 (GNS, 2011) found some evidence of pre-existing surface expressions by comparing historic aerial photographs and LiDAR datasets. This indicates that surface movement had occurred previously from this fault but had not been recognised. No similar surface expressions have been identified within the proposed pond site. However, surface evidence may not exist due to current land use activities. Evidence of low-rate buried fault structures is discussed in the following Section 4.5.3.

4.5.3 Subsurface Fault Evidence

A number of geophysical seismic survey lines (Figure 2 of the GNS 2012 report) were performed by the University of Canterbury (Campbell, 2010), (Finnemore, 2004) and the Institute of Geophysics, Switzerland (Dorn et al. 2010) to investigate buried tectonic structures in Canterbury. No significant buried structures are inferred at the Wrights Road Storage Ponds site from the seismic lines performed (dashed lines shown). Seismic line S2 just south west of Waimakariri River provides the most information regarding potential NE-SW trending fault traces relative to the pond site. The output image shows strain or dips in the subsurface ground, which are primarily concentrated in the northern foothill area. The Racecourse Hill Anticline is visible. Other fault traces are identified adjacent to the Hororata Fault and southwest of Burnt Hill. There is no evidence of surface rupture from the fault traces identified from these seismic lines close to the project site.

GNS provided a map of earthquake epicentre locations in the Oxford area (refer Figure 9, GNS 2012 Report Figure 3), which does not support a trend towards the project site that would indicate presence of a causative fault trace. Therefore, we cannot foresee fault rupture displacing the pond embankments leading to breach. The earthquakes shown are all events from the GeoNet catalogue, recognized by GNS to have well-constrained locations, between September 2010 and January 2012. Earthquake events in the GeoNet catalogue that are poorly-constrained, either in location or detection (phase picks), were excluded. We are advised by GNS that the website www.canterburyquakelive.co.nz shows poorly constrained events and therefore they will not endorse its accuracy.

4.5.4 Earthquake Design Criteria

The NZDSG provide performance criteria for dam design under seismic loading. For Medium and High PIC dams, embankments are to be designed for two levels of earthquake loading as follows:

- The Operating Basis Earthquake (OBE)
- The Safety Evaluation Earthquake (SEE)[2]

The OBE is the minimum service level of ground motion to evaluate the design and performance of the dam. Performance criterion under the OBE loading allows only minor damage to occur. Following the OBE, there should be either no damage, or minor repairable damage to the dam. The performance criterion under the OBE loading condition applies to all PIC dams. NZDSG recommend ground motions for the OBE represent a 1 in 150 AEP.

[2] The Safety Evaluation Earthquake was formerly referred to as the Maximum Design Earthquake (MDE) in the NZSOLD 2000 Guidelines.

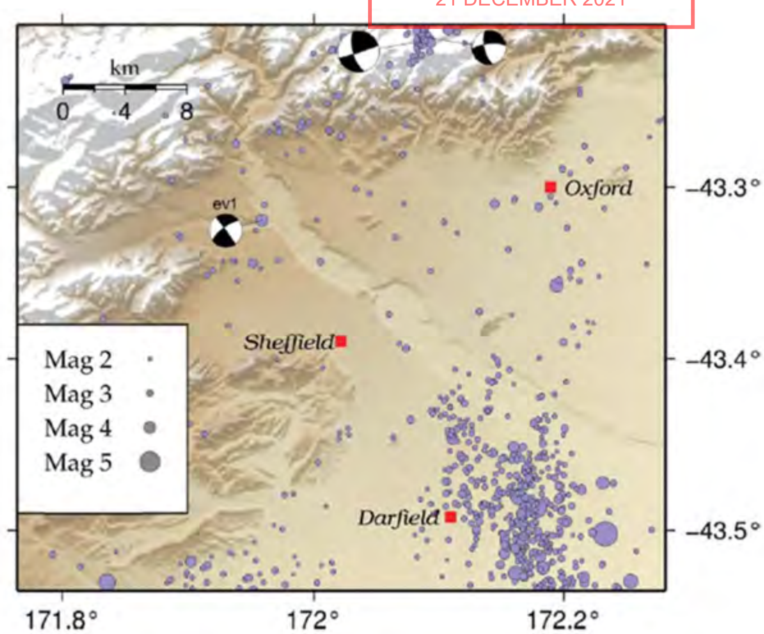


Figure 3 The location of earthquakes in the Oxford area detected by GeoNet between September 2010 and January 2012, and subsequently relocated using a 3D velocity model developed for the Canterbury region. Note, that only well-constrained earthquake relocations are shown here; events in the GeoNet catalogue with insufficient phase information or that were poorly constrained after relocation have not been included. Moment tensor solutions, shown for three events, indicate right-lateral strike-slip faulting. The event labelled 'ev1', (M_w 3.7, on 2011-01-28) is close to the surface expression of the Springfield fault (Figure 2), suggesting some activity on that fault. There is also some focused activity near and on the Porters Pass-Amberley fault zone.

Figure 9: GNS June 2012 Report Figure 3 showing earthquake locations in Oxford area

The SEE is the earthquake that would result in the most severe ground motion, which a dam must be able to endure without the uncontrolled release of the reservoir. The SEE is based on the PIC of the dam. For High PIC dams, NZDSG recommend the 84th percentile level for the Controlling Maximum Earthquake (CME) and need not exceed the 1 in 10,000 AEP ground motions. For Medium PIC dams, the guidelines recommend the 50th to the 84th percentile level for the CME and need not exceed the 1 in 2,500 AEP ground motions. The PIC classification for the proposed Wrights Road Storage Ponds is Medium for the western embankment, and High for all other embankments, as summarised in Section 3.

4.5.5 Design Ground Motions

Recent seismic activity in the region has resulted in changes to the National Seismic Hazard Model (NSHM) with consideration of time dependent seismic activity function. The Building Code NZS 1170.5:2011 Amendment 10 reflects this change for the Canterbury region with an increase in the spectra hazard factor (Z). For the Wrights Road Storage Ponds the spectra hazard factor Z increased from 0.30 to 0.34 (Appendix K).

GNS provided site specific recommendations for peak ground accelerations (PGA) and spectral accelerations parameters based on recommendations from The Expert Elicitation Group, which provides guidance on current practice for Canterbury design ground motions. The response spectra are constructed by using the most conservative PGA values (period, T=0) obtained from the unweighted Bradley (2010) model. For period T≥ 0.1 sec, the response spectra envelop values obtained from the weighted McVerry (McVerry, 2006) model and unweighted Bradley by using GNS recommended Building Code NZS1170.5 Deep Soil spectra parameters. Specifically, a Z of 0.34 and return period factor (R) of 0.67, 1.8 and 2.4 corresponding to 1 in 150 year, 1 in 2,500 year and 1 in 10,000 AEP, respectively.

The recommended response spectra for the Wrights Road Storage ponds is shown in Figure 10 and the PGA corresponding to T=0 sec (shown on Figure 9 at T=0.03 sec) are listed in Table 9.

The OBE (1 in 150 year AEP) response spectrum applies to the Medium and High PIC embankments. The two SEE response spectra shown represent the 1 in 2,500 AEP for the Medium PIC embankment and the 1 in 10,000 AEP for the High PIC embankment. Damwatch performed engineering assessments using these scenario ground motions and spectra as recommended by GNS.

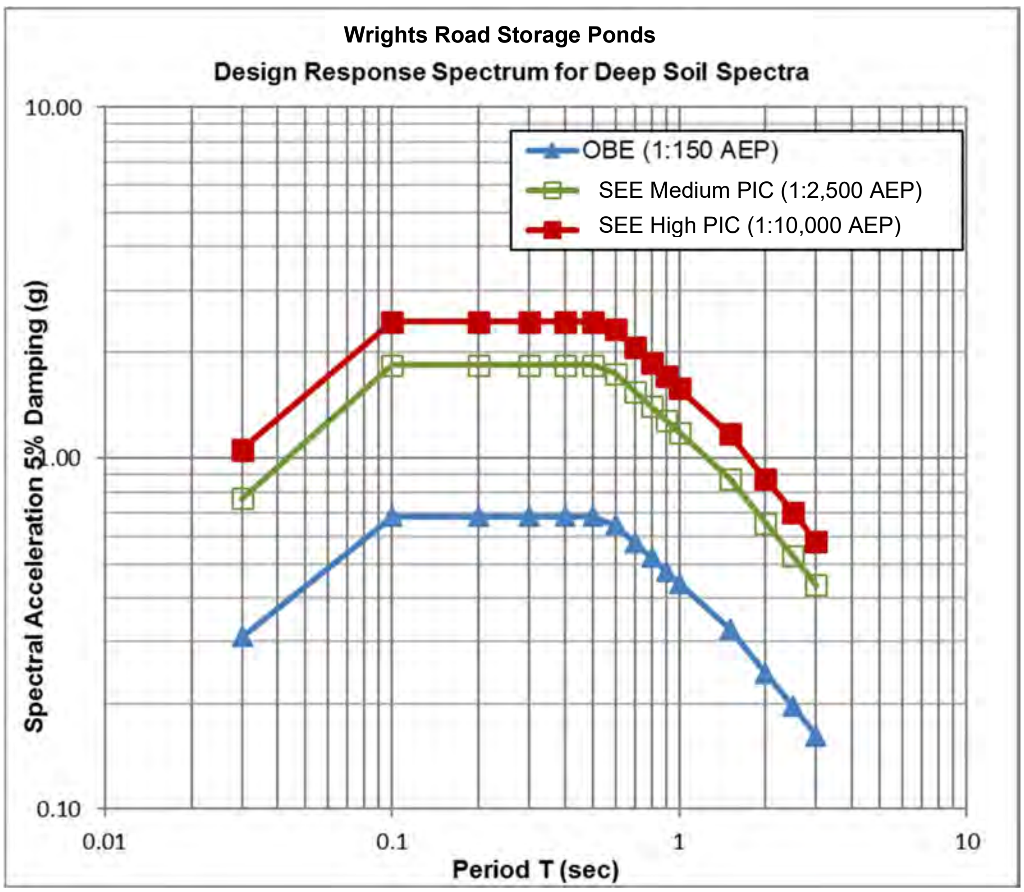


Figure 10: GNS recommended response spectra for design ground motions

Table 9: Design ground motions for Wrights Road Storage Ponds

Design Case	Moment Magnitude (M_w)	Peak Horizontal Ground Acceleration (g)
Main Shock		
OBE	N/A	0.31
SEE - Medium PIC	7.0	0.77
SEE - High PIC	7.0	1.06
Aftershock		
OBE	N/A	N/A
Medium PIC	6.0	0.40
High PIC	6.0	0.50

An earthquake M=7.0 was used to evaluate the embankment performance under the SEE ground motions (Table 9)
This magnitude corresponds to 84th percentile level earthquake scenario with a PGA of ~1.06g on the Springbank Fault located 3km from the site using the Bradley model. An earthquake magnitude was not determined for the OBE loading condition based on satisfactory performance of embankment design as discussed in the slope stability assessment provided in Section 7.8.

Aftershocks earthquake magnitude and PGA were used to evaluate embankment performance following the main shock (SEE). These values are listed in Table 9.
The aftershock M=6.0 is one magnitude less than the main shock CME as recommended by NZDSG (2015). Aftershock PGAs of 0.40g and 0.50g for the Medium and High PIC loading conditions, respectively, correspond to conservative values expected for Deep Soil site with M=6.0 reverse-mechanism earthquake (i.e. the Springbank Fault) and epicentral distance being less than 10 km using the McVerry (2006) model.

4.5.6 GNS 2015 Review of Earthquake Hazard

Additional studies of PGA ground-shaking hazard and ground surface deformations were conducted by GNS subsequent to the Expert Witness Conference on seismic effects on 16/09/15. The GNS report of these studies is included in Appendix I. This section addresses the PGA ground-shaking hazard findings. The investigation of ground surface deformations is addressed in Appendix I.

The ground-shaking hazard issues that were assessed relate to:

- the sensitivity of both probabilistic and deterministic (“scenario”) ground-shaking estimates to the recurrence intervals of several of the local faults,
- the modelled location of the Hororata and Springbank Fault with respect to the ponds, and
- the effects of including hanging-wall factors for the Hororata Fault.

The PGA sensitivity studies were performed only for the McVerry stress-drop enhanced ground-motion prediction equation (GMPE), because the previous calculations showed that this model gave stronger PGAs than the Bradley GMPE (which was adopted for design).

The GNS assessment showed that the 1 in 10,000-year PGAs are insensitive to large changes in fault-rupture recurrence intervals for five near-by faults affecting the site, reductions in the source-to-site distance and enhancement of PGAs by hanging-wall (or other) effects. The greatest increase found from the sensitivity studies was less than 5% for the unweighted 10,000-year PGA, for a case combining the simultaneous application of multiple conservative assumptions.

On the basis that the impact of increased fault rupture frequency, fault location relative to the ponds and hanging wall effects, cumulatively impact the SEE PGA less than 5%, we consider that these issues have negligible impact on design of the embankments using PGAs listed in Table 9, which were adopted for design.

5.0 Concrete Structures

The reinforced concrete culvert transitions and crump weirs have been designed generally in accordance with the design methods and criteria provided in NZS 1170:2004 (Structural Design Actions) and NZS 3101:2006 (Design of Concrete Structures). The seismic design loads are provided in Appendix L.

5.1 Design Life

Design is based on a design life for the pond embankments and appurtenant structures of greater than 50 years. Longevity of these structures will be contingent on construction being implemented in accordance with the design, specification, workmanship and appropriate maintenance of the structure during its operating life.

5.2 Earthquake Design Loads

The concrete culvert transitions are appurtenant structures to the pond embankments. If structural components are critical to the safety of the associated embankment, they need to satisfy dam safety criteria. If they are non-critical, they need to comply with the 2004 Building Act.

The critical structural components are the gated headwalls. These are to withstand 10,000-year SEE (1.06 g PGA) shaking without damage or deformation that would prevent post-earthquake operation of the gates. This ensures that outflows from the ponds can be made immediately after the 10,000-year event if required.

All other structural components, such as the transition and crump weir sidewalls, are considered as non-critical. These are assumed to be Importance Level 2 structures in accordance with NZS 1170 with a design life of 50 years. The SEE seismic design load in this case is the 500-year (0.47 g PGA) event. The non-critical concrete walls are designed to withstand the 500-year earthquake as an elastically responding structure. This implies that the walls would likely be able to withstand the 10,000-year shaking without collapse, but with considerable, permanent structural damage.

With the above SEE seismic performance criteria, the structures easily meet the 150-year OBE performance criterion of minor damage which would not prevent usual operation of the irrigation ponds. The OBE has a PGA of 0.31 g (refer to Table 9).

5.3 Ductility

A ductility factor μ of 1.0 is assumed for all concrete design.

5.4 Loadings

5.4.1 Static Water Pressure Loading

No net water pressures are assumed to act on the walls of the transitions (land side), whether submerged or dewatered, as the ponds are lined and the embankment shoulder material is free-draining.

5.4.2 Surcharge Loading

For the design of the G6 outlet traffic loading behind the wall is assumed to be a uniform surcharge loading q_s = 12 kN/m^2 (12 kPa). No surcharge is included for other transitions.

5.4.3 Static Earth Pressure Loading

The assumed angle of friction (Φ) of the sandy gravel backfill used for backfill of the walls is 38º. Zero cohesive strength is assumed. The active earth pressure is:

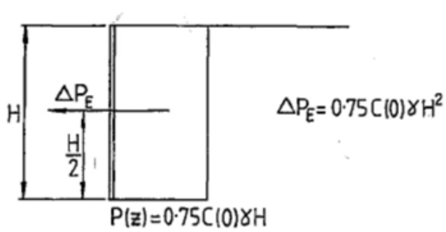
- $P_A = 0.5 \times K_A \times \gamma \times H^2$
- The active pressure coefficient (K_A) is determined in accordance with Rankine's method, taking account of sloping backfill if present.
- The assumed unit weight (γ) of the backfill is 20 kN/m^3;
- Height of the retaining wall (H).

For the rigid headwalls, an at-rest pressure K_o coefficient is assumed.

5.4.4 Earthquake Loadings

The walls are assumed to be 'stiff' walls (that is, the displacement at the top of the walls under earthquake loading will be less than 0.2% of the wall's height. The earthquake surcharge loading on stiff walls due to the earthquake PGA is (NZSEE, 1980):

- $\Delta PE = 0.75 \times PGA \times \gamma \times H^2$.



For earthquake loading, all transition structures are assumed to be under normal operating conditions at the time. That is, all structures are assumed to be submerged except for the outlets to the G2, G3 and G6 structures.

5.4.5 Uplift (Flotation)

Where structures are subjected to uplift, a factor of safety of 1.5 under usual load conditions has been adopted to provide a suitable margin of safety against hydrostatic uplift (flotation) and the self-weight of the structure.

5.5 Load Combinations

The combination of loads (or actions) and load factors used for limit state design of the reinforced concrete structural elements is in accordance with AS/NZS 1170.0:2002.

The design load is computed so that the ultimate strength capacity of the section is sufficient. The design load combinations are:

- Usual Loading $U_1 = (1.25 \times EP) + (1.5 \times SL)$
- Earthquake loading $U_2 = (1.25 \times EP) + (1.0 \times EQ)$

Where: U = ultimate load

EP = static earth pressure

SL = surcharge load

EQ= earthquake earth pressure

5.6 Sliding and Overturning Stability

The following factors of safety against rigid body sliding or overturning have been adopted for normal loading:

- 2.0 safety factor against overturning;
- 1.5 safety factor against sliding.

For earthquake loading, the following factors of safety have been adopted:

- 1.5 safety factor against overturning;
- 1.2 safety factor against sliding.

5.7 Wall Design

The headwalls have been designed as one-way slabs spanning horizontally between sidewall supports. The sidewalls have been designed as two-way slabs (combination of cantilever and beam action). All horizontal reinforcing is made continuous through wall corners to ensure full potential beam action of the walls can be developed under earthquake loading.

5.8 Material Properties

The specified compressive strength of the concrete is f'c is 30 MPa. However, for the inverted “L” beam of impact type energy dissipater of R2 concrete, f'c is 50 MPa (Ref Drawing WIL1125/30/106).

The specified yield strength of the reinforcing bars fy is 500 MPa.

5.9 Concrete Cover

The specified concrete cover for all exposed conditions is 50 mm. For buried structures not exposed to groundwater, the specified concrete cover is 50mm.

5.10 Contraction Joint Spacing in the Service Spillway

The 200 mm thick invert slabs for the spillway are reinforced with 661 mesh. This provides a 0.15% reinforcing steel content, meeting the 0.14% minimum code requirement for temperature and shrinkage reinforcement. The slab is laterally unrestrained, and on a gravel foundation (rather than rock). It can therefore undergo relatively high thermal contraction or drying shrinkage without cracking of the concrete. A contraction joint spacing of 7 m should prevent possible cracking of the slab from volume change effects. All contraction joints are provided with waterstops, as shown in the drawings.

5.11 Precast Concrete Pipes

Precast concrete pipes have been designed according to the design methods and criteria included in AS/NZS 3725:2007 - Design for the Installation of Buried Concrete Pipe and Humes Concrete and Drainage Structure Manual.

5.12 Steel Pipes

Steel pipes have been designed according to the design methods and criteria included in AS/NZS 2566:1998 - Buried flexible pipelines Part 1: Structural Design, and Steel Pipe New Zealand Manual, as applicable.

5.13 Inline Flowmeter Chamber

A buried reinforced concrete chamber has been provided downstream of the G6 control structure and embankment, to accommodate a flowmeter in the outlet pipes. Details for the inline flowmeter chamber are provided on drawings WIL1125/30/190-195. The length of outlet pipes inside the chamber comprise steel pipes that mate with the concrete precast pipes upstream and downstream of the chamber. The chamber lid comprises 3 removable precast concrete slabs, 2 of which are provided with manholes. Inside the chamber, the steel pipes are supported on steel supports that can be fabricated off-site and then positioned and installed inside the chamber. A steel landing (platform) and steel steps have been provided inside the chamber to enable access from one side of any pipe to the other. However, there is no permanent access ladder or stair flight to gain entry to the chamber. This has been done in the interest of health and safety, so that each entry to the chamber necessitates a confined space entry plan and relevant hazard ID and health and safety protocols are adhered to.

The chamber is a buried structure surrounded by free-draining gravel fill on all sides. The groundwater level at the location of the chamber is some 20m to 25m below the surface. The structure is therefore stable against sliding because of the considerable restraint from backfill on all sides and is not expected to be subject to any loads that would induce flotation.

The purpose of the flowmeter chamber is to enable installation and inspection/maintenance of the meters inside it and hence, it does not fulfil a dam safety function. The structure has therefore been designed to Importance Level 2 according to NZS 1170.0, with a design life of 50 years. The SEE for seismic design of the chamber is the 500-year event (PGA = 0.47 g). The chamber can withstand seismic loads induced by the 500-year earthquake without undergoing any structural damage. This implies that it is likely to withstand 10,000-year shaking without collapse, although with considerable structural damage.

6.0 Site Conditions, Foundations, and Material Assessment

6.1 Geologic Setting

The Waimakariri River sits within the vast expanse of the Canterbury Plains which comprise of coalesced floodplains. Large parts of the plains are abandoned braided-river flood plains that consist of outwash alluvial deposits comprising gravel, silt, clay and peat. The composite terrane mainly comprises of thick, deformed packages of greywacke. The basement rocks are Paleozoic to Mesozoic sedimentary and metamorphic rocks, termed Torlesse composite terrane that were originally part of the Gondwanaland supercontinent.

6.2 Groundwater

The regional groundwater system (reference Forsyth et al. 2008, Geology of the Christchurch Area, Institute of Geological & Nuclear Sciences 1:25,0000 Geological Map 16) for Canterbury Plains is described typically to include shallow unconfined aquifer with the water table at less than 20 m depth and hydraulically connected to nearby surface water (i.e., Waimakariri and Eyre Rivers). Water yield in shallow soils may vary laterally over short distances in more permeable channel deposits. Below the water table, deeper aquifers are present below 30 m depth. Groundwater movement is generally downward through the permeable Gravels (Coarser soils) of the Canterbury Plains and eastward toward the Pacific Ocean.

The groundwater table at the Wrights Road Storage Ponds site was assessed from monitoring data of two nearby wells. Monitoring data of groundwater well levels is maintained by Environment Canterbury Regional Council and may be accessed from their website at http://ecan.govt.nz/services/online-services/monitoring/groundwater-levels. The locations of the two groundwater wells relative to the proposed ponds are shown on Figure 11 and are as follows: Well L35/0051 with a ground surface at RL 243.1 m is 5 km to the northwest and well M35/0174 with a ground surface at RL 160.5 m is 7 km to the east. The proposed ponds have a corresponding ground surface roughly between these two wells, from RL 220 m at the western perimeter to RL 212 m at the eastern perimeter. Both wells are hydraulically connected to the Eyre and Waimakariri Rivers. Well L35/0051 is approximately 3 km south of the Eyre River and 9 km north of the Waimakariri River. Well M35/0174 is more centrally located between these two rivers, at approximately 5.5 km to the south and north, respectively. This is similar to the location of the proposed ponds, which is approximately 6.5 km south of Eyre River and north of Waimakariri River. Unfortunately, no water stage levels are recorded for the Waimakariri River in the project vicinity to evaluate water table levels.

The groundwater level for the proposed ponds is estimated to be at least 20 m below ground surface based on the ground surface level of Waimakariri River and groundwater levels in wells L35/0051 and M35/0174. The ground surface of Waimakariri River's outer banks south of the proposed ponds vary from 198 to 186 m MSL., which corresponds to 22 to 26 m below ground surface at the project site. This is consistent with the average groundwater depth of 25 m recorded in the two wells. The maximum groundwater level recorded in these wells is 14.4 m and 13.3 m below ground surface, which was not maintained and most likely corresponding to extreme flood events.

Figure 11: Locations of Wrights Storage Ponds and Groundwater Wells

6.3 Site Investigation

Characterisation of the foundation soils is based on twenty test pits (TP01 through to TP20) excavated across the footprint of the proposed irrigation ponds by Rooney Earthmoving Ltd (REL) in October 2011. Location of these test pits are shown as red and pink boxes in Figure 12.

During the feasibility stage, 21 test pits were excavated inside and outside of the proposed footprint of the irrigation ponds under the supervision of MWH in April 2007 (MWH, 2007). Location of test pits relevant to (inside or adjacent to) the proposed footprint (i.e., TPP01, 02, 03, 04, 05, 06, 07, 08, 09, 10, 12, and 13) are shown as blue boxes in Figure 12.

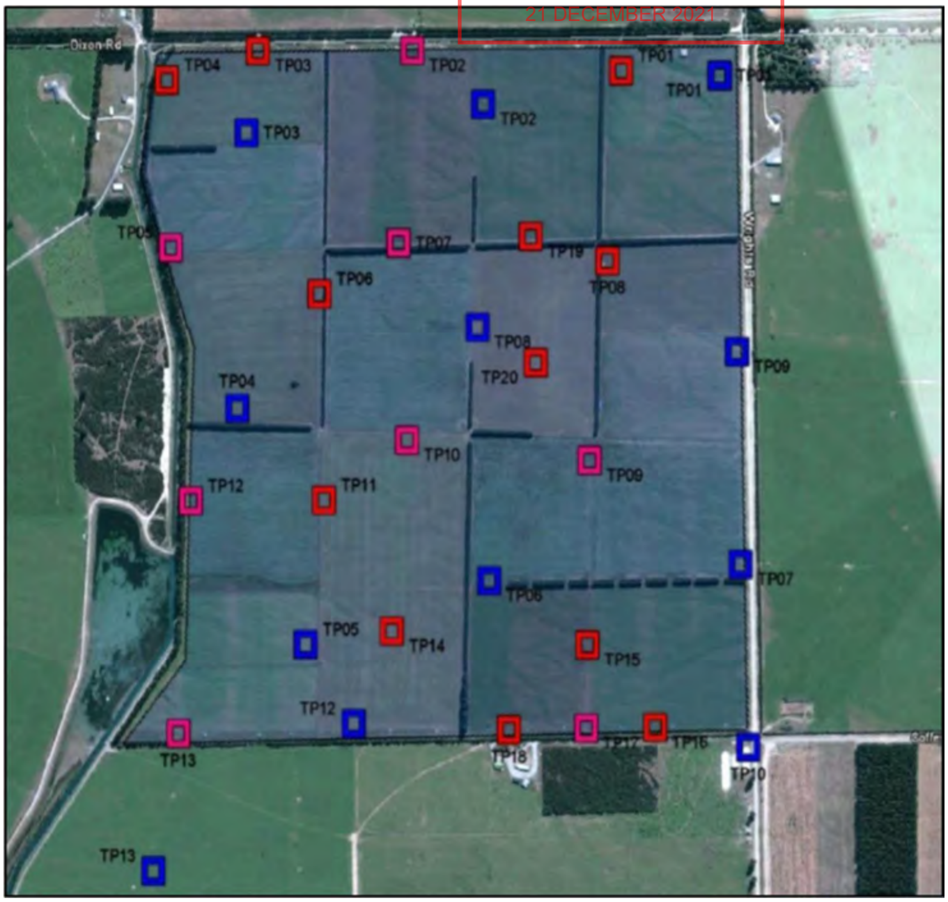


Figure 12: Locations of Test Pits from REL and Previous Investigation

The classification of foundation materials is based on samples taken from test pits, field inspection, logging of test pits and laboratory test results. Two general soil units were encountered in the test pits and are described as:

- Finer soils; and
- Coarse soils.

Finer soils were encountered in the upper unit from ground surface to about 0.6 m depth and overlie the coarser soils. Finer soils generally consist of sandy Silt Topsoil and Sandy Silt. Underlying coarser soils consist of silty sandy fine to coarse Gravel to sandy Gravels with some cobbles and boulders and interbedded with lenses of clean Gravels and locally, with layers of cobbles and boulders to the maximum depth excavated. The REL test pits were excavated to depths ranging between 3.2-5.2 m deep using a 20 tonne hydraulic excavator. The MWH test pits ranged between 3.5-4.2 m deep using a 21 tonne hydraulic excavator. Individual logs of REL test pits are in Appendix A – Test Pits Data.

No groundwater was encountered in any of the test pits. Soils encountered were generally dry to moist in upper layers and damp to wet below 2.0 m depth in REL investigation and below 2.9

m depth in the previous MWH investigation. Locally, inter-bedded loose clean Gravel lenses were described as wet (as discussed subsequently).

Bulk (“grab”) soil samples were obtained from the test pits for laboratory testing. A total of nine dry sieve analyses, three particle size analyses (i.e., hydrometer method), three Atterberg Limits (i.e., Liquid Limit, Plastic Limit, and Plasticity Index), and seven compaction tests were performed on samples obtained from REL’s test pits. Previous laboratory testing performed on samples obtained from MWH’s test pits consisted of six dry sieve analyses, one Atterberg Limits (i.e., LL, PL, and PI), and six compaction tests. Individual reports of laboratory tests performed on samples from REL test pits are included in Appendix B - Soil Laboratory Test Results.

The grading analyses for both the finer and coarser soils are shown on Figure 13. Further discussion of each of the soil categories is detailed subsequently.

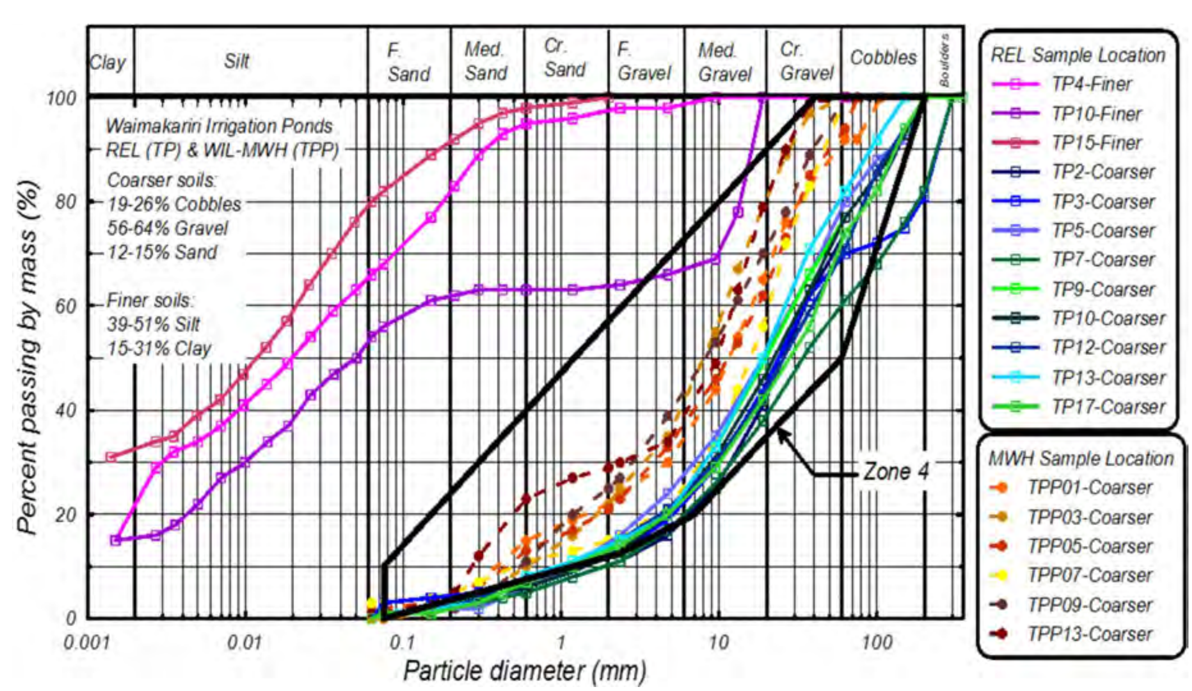


Figure 13: Particle Grading Distribution of Test Pit Samples

6.4 Finer Soils

Finer soils are of limited depth and characterised as fine sandy Silt (Topsoil) from ground surface to 0.2-0.3 m depth and overlie, locally, fine sandy Silt from 0.3 to 0.6 m depth. These soils classify as sandy clayey Silt to clayey Silt with some Sand as per the NZ Geotechnical Society’s “Field Description of Soil and Rock” (NZGS, 2005) and as sandy Clay to Clay (CL) with sand as per the Unified Soil Classification System (ASTM Standard D2487-11, 2011). Organic materials consisting of grass and rootlets were encountered in this unit.

Results of particle size analysis tests on finer soils obtained from REL test pits TP04, 10 and 15 are shown in Figure 13. These samples consist of 54-80% fines, 10-32% sand (primarily fine-sized sand), and 0-36% Gravel (primarily medium-sized Gravel). The fines (<0.06 mm) consist of 15-31% clay and 39-51% Silt.

Results of the four Atterberg Limits tests performed on the same three REL samples obtained from TP04, 10 and 15 and one MWH sample obtained from TPP04 are shown on a plasticity chart in Figure 14. These samples are of low plasticity with a PI of 9 to 17 (average 12) and LL of 28 to 41 (average 33).

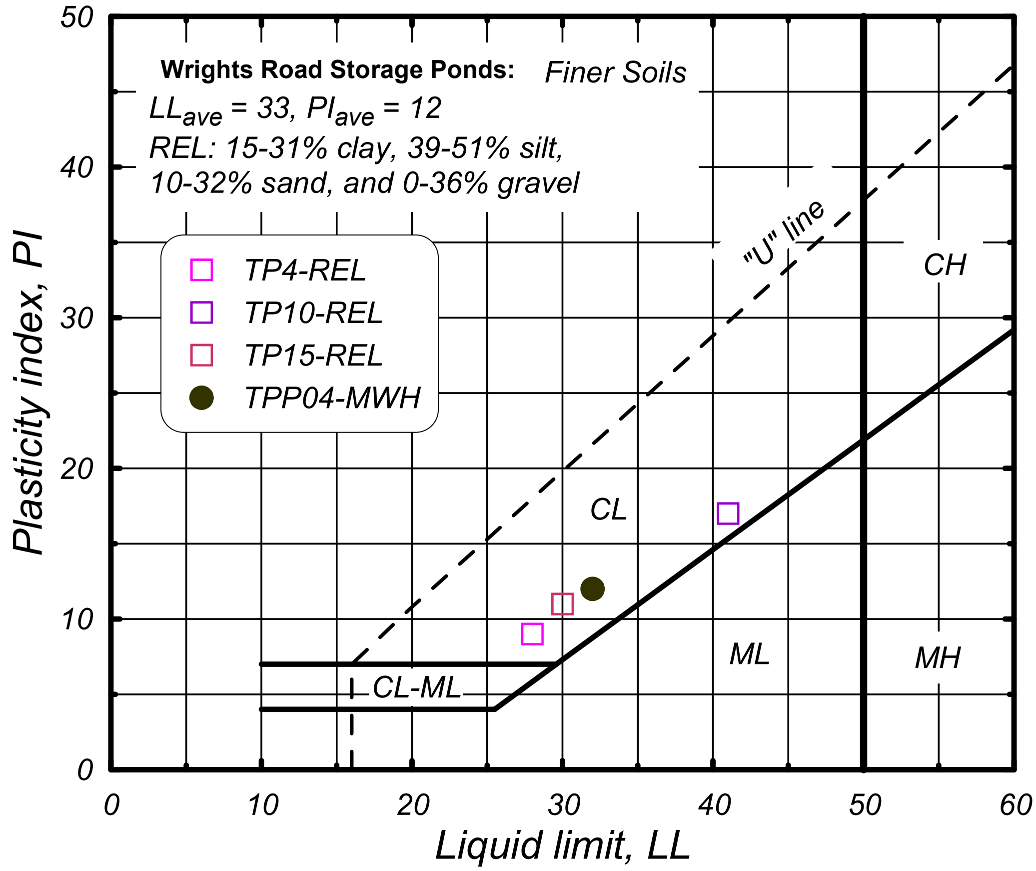


Figure 14: Results of Atterberg Limits Tests of Finer Soils

6.5 Coarser Soils

The coarser soils consist of outwash Gravels in an aggrading profile with varying amounts of fines, sands and oversized cobbles and boulders and were encountered below finer soils to the maximum depth investigated of 5.2 m. Coarse soils generally consist of silty sandy fine to coarse Gravel overlying sandy fine to coarse Gravel interbedded with gravel lenses with varying amount of silt, as listed in Table 10. Cobbles and boulders were encountered within coarse soils throughout the depth of excavation and in 0.15-0.5 m thick discrete layers. The maximum particles size ranged between 125-200 mm and up to 300-400 mm diameter. Interbedded

Gravel lenses were typically 50-250 mm and up to 500 mm in thickness. In the footprint of Pond 2, loose clean Gravel lenses were encountered as discussed subsequently. Locally, a layer of tree roots was encountered at 1.2 m depth in TP12.

Coarser materials were easy to excavate in REL investigation with the test pits walls being generally stable above 2.5 m depth. Below 2.6 m depth, caving of side walls was observed where materials were damp-wet. Materials encountered in the test pits performed by MWH were generally described as tightly packed, except where loose gravel lenses were encountered.

Table 10: Coarse Soil Profile in Test Pits

Coarse Soils	Thickness	Depth	
		From (m)	To (m)
Silty sandy fine to coarse Gravel with some cobbles and boulders	0.2-1.1 m & 1.9-2.3m	0.3-0.6	0.4-1.1 up to 2.1
		1.2	3.5
Sandy fine to coarse Gravel with some cobbles and boulders	0.3-1.6	0.4-1.1	1.0-2.3
Sandy fine to coarse Gravel with cobbles and boulders and interbedded clean gravel lenses	0.20-1.3	1.0-2.3	1.8-2.8
Sandy fine to coarse Gravels with cobbles and boulders and interbedded gravel in sandy silt	1.0-3.0	1.8-2.8	3.2-5.2

6.5.1 Loose Gravel Lenses in Coarser Soils

Inter-bedded loose clean Gravel lenses between 100 and 500 mm thick were encountered under the footprint of Pond 2 during both MWH and REL field investigations. Loose Gravels were encountered between 2.3 and 3.1 m depth (RL 215.7 to 214.9 m in the centre of Pond 2 and at slightly shallower depths between 1.5 and 2.2 m depth (RL 213.2 to 211.8 m) along the eastern perimeter. At the southern perimeter, a loose clean Gravel layer was encountered from 0.7 to 1.2 m depth (RL 219.3 to 218.8 m) in TP18 and multiple loose Gravel layers of limited thickness were encountered in TPP12 between 0.5 and 2.7 m depths (i.e., RL 216.5 and 214.3 m RL.). Similar multiple loose Gravel lenses were encountered in TPP13 located outside the footprint of the proposed ponds between 2.0 and 3.6 m depth.

6.5.2 Results of Laboratory Tests on Coarser Soils

Results of dry sieve analyses on coarser soils obtained from REL test pits TP02, 03, 05, 07, 09, 10, 12, 13, and 17 and MWH test pits TPP01, 03, 05, 07, 09, and 13 are shown in Figure 13. REL samples were generally obtained from 0.7 to 2.2 m depth, but as shallow as from 0.25 to 0.7 m depth and up to 5.2 m depth. Gradation curves of REL samples include oversized materials consisting of cobbles (60 mm ≤ diameter < 200 mm) and boulders (200 mm ≤ diameter). MWH samples were obtained from 0.75 to 2.3 m depth and do not include oversized materials. Coarse-grained soils to be used for construction of the pond embankments are identified in the gradation curve as Zone 4.

Samples classify as well graded Gravel (GW) per Unified Soils Classification System (USCS). REL samples consist of 11-16% sands, 50-67% Gravel, 11-29% cobbles and 0-19% boulders.

The REL test pits encountered boulders generally with a maximum size of 250 mm but up to 400 mm in diameter were reported in MHW test pits. Gradation of MWH samples consist of 0-3% fines, 13-29% sands, 68-84% Gravels, and 0-8% cobbles. Gradation curves of REL samples are similar to those of the MWH samples when the oversized materials are not included. This may reflect the sampling method adopted and exclusion of the cobbles from the samples taken for laboratory analysis. Samples obtained from test pits performed by MWH to the south of the proposed ponds (not shown in Figure 12) were generally similar with slightly higher fines (+1%) and Gravel (+3%) content and lower sand (-5%) content.

Table 11: Summary of Compaction Tests on Coarser Soils

REL Test Sample	Maximum Dry Density, MDD (kN/m^3)	Optimum Water Content, ω_{opt} (%)	MWH Test Sample	Maximum Dry Density, MDD (kN/m^3)	Optimum Water Content, ω_{opt} (%)
TP2	21.1	4.8	TPP02	20.5	6.0
TP3	19.0	5.0	TPP06	21.4	7.0
TP5	20.2	3.6	TPP10	21.1	6.0
TP7	20.8	4.6	TPP13	19.7	3.5
TP9	20.6	3.4	TO17	20.6	5.0
TP10	20.9	3.6	TP21	21.2	7.0
TP12	20.9	5.0			
Average	20.5	4.3		20.7	5.8
Average of All		MDD = 20.6 kN/m^3 and ω_{opt} = 5.0%			

Coarse soils have an overall average maximum dry density (MDD) of 20.6 kN/m^3 at an optimum water content (ω_{opt}) of 5.0%. Results of the 13 compaction tests on samples obtained during both field investigations are summarized in Table 11. The REL samples have an average MDD of 20.5 kN/m^3 and ω_{opt} of 4.3%, which is slightly lower than MDD of 20.7 kN/m^3 and ω_{opt} of 5.8% obtained from MWH samples.

Soil and strengths properties of embankment and foundation material (i.e., River Gravels and Loose Gravels) used in the static and pseudo-static (seismic) stability analyses are shown in. Embankment material properties were based on results of compaction tests on all samples (i.e., 95% of average of all MDD). Embankment fill was assigned typical strength values with an angle of internal friction (ϕ) of 38° and cohesion (c) of zero representative of a compacted well graded free-draining Gravel with similar density specified in the field based on published values (Kulhawy and Mayne, 1990), (USBR, 1987) and laboratory testing on similar gravels fills (Jones 1965, Pender 1972). The River Gravels and loose Gravels properties were conservatively estimated from typical values (Kulway & Mayne, 1990) for free-draining Gravels of medium and loose consistency, respectively.

Table 12: Soil properties used in static and pseudo-static stability analyses

Material	Unit Weight, γ (kN/m^3)	Static		Pseudo-static	
		Angle of internal friction, ϕ' (degree)	Cohesion, c (kPa)	Angle of internal friction, ϕ (degree)	Cohesion, c (kPa)
Embankment	19.6	38	0	38	0
River Gravels	18.5	35	0	35	0
Loose Gravels	18.0	32	0	32	0

6.6 Assessment of Liquefaction Potential

The potential for liquefaction of the loose, unsaturated, clean gravel is considered low based on the location of groundwater table approximately 20 m below the base of the lowest layer, being highly permeable, and the limited thickness and lateral extent to which they were encountered. The clean Gravel layers are approximately 0.1 to 0.5 m thick (average 200 mm thick) and were encountered at variable depths, with the majority of these materials being removed as part of Pond 2 excavation to RL 213.14 m. Below Pond 2 external embankment, loose materials with a maximum thickness of 0.5 m thick was encountered at 0.7 m depth along the southern perimeter (TP18). This layer tapers to 0.1 m thickness going eastward (TPP10) and is of similar thickness along the eastern perimeter at an average depth of 1.8 m (TPP07 and 09). Going westward along the southern perimeter, three discrete loose lenses were encountered. Two of them were approximately 0.1 m and the other was 0.3 m thick in TPP12, but not in adjacent test pits TPP05, TP14 and TP13. All loose layers are located above the groundwater table estimated at least 20 m below the pond embankment foundation (as previously discussed in Section 6.2). In addition, limited fines were encountered in these gravels, which are of high permeability and likely to not develop high pore pressures during earthquake shaking.

The potential for liquefaction in the coarse soils, consisting of silty and sandy gravels, is also considered low based on the description of being “tightly packed” and its unsaturated condition (typically dry to moist). Strengths of these Gravel layers are difficult to assess using common in-situ test methods, such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). Measured blow counts (N) of SPTs are higher than expected due to the influence of larger-sized particles (i.e., gravels, cobbles, and boulders) interfering with driving the sampler, which has an inside diameter opening of 34.9 mm. CPTs on the other hand cannot penetrate into these coarse-grained materials. The Becker-hammer Penetration Test (BPT) with a 168 mm inside diameter sampler is better suited to evaluate gravel and cobbles, but similar to SPTs would not provide representative results, due to large size cobbles and boulders present at the site.

7.0 Design of Storage Ponds

7.1 Concept

The ponds have been located and sized to:

- Store a total of 8.2 x 10^6 m^3;
- To have a maximum normal operation water depth of 11 m;
- Be supplied from the existing Buffer Pond and deliver irrigation and stock water supply to existing WIL main race MR4 and distribution races R3 and R2; and
- Supply water for hydro power generation in the future.

The design utilised previous experience gained from the following similar irrigation projects, which were categorised as Medium PIC dams:

- Rangitata South Irrigation storage ponds;
- Carew Ponds which are owned and operated by Mayfield Hinds Irrigation Ltd (MHIL).

As the proposed Wrights Road Storage Pond embankments are categorised as both Medium and High PIC, a design standard consistent with NZDSG (2015) has been adopted.

Materials for pond construction are to be sourced from within the footprint of the ponds and design is managed to achieve this wherever possible. In this regard, a composite soil liner option was investigated. Experience with the projects listed above strongly influenced the decision to adopt a geomembrane liner.

Design of the pond embankments considered the following failure modes:

- Embankment failure due to earthquake induced slumping and consequent overtopping of water at locations of reduced embankment crest levels;
- Embankment failure due to earthquake induced local deformations and consequent leakage through the damaged geomembrane liner; and
- Piping through the embankment or foundation due to leakage through the liner leading to failure, possibly initiated by earthquake shaking.

7.2 Pond Embankment Design

Permeability of Gravels underlying the pond footprint necessitates a lining to control seepage from the ponds and this lining is to extend up the full height of the pond embankments.

Internal embankment slopes of 1V:3H have been adopted based on experience with construction of geomembrane lined embankment slopes. Embankment outside slopes of 1V:2H for the western embankment and 1V:2.5H for all other embankments have been adopted based on embankment stability analysis provided subsequently in Sections 7.8 and 7.9. Refer to Drawings

WIL1125/30/22, 23, and 101 for typical pond embankment sections and positioning of the geomembrane lining adopted.

7.3 Setting Crest Level Based on Freeboard

7.3.1 Wind Speed & Critical Fetch

The site wind speeds were determined from the regional wind speeds for all directions based on three second gust wind data given in AS/NZ 1170.2:2011, Structural Design Actions Part 2: Wind actions. Two design wind speeds were determined for minimum and normal freeboards. As per USBR (1992) and US Corps of Engineers (1984), minimum and normal freeboards were determined following two combinations of parameters and conditions:

- Minimum freeboard with maximum water level; i.e. full supply level plus probable maximum precipitation (PMP), and adding the sum of wind set up and wave run up due to a wind expected with extreme rainfall (1 in 10 year recurrence interval);
- Normal freeboard with normal water level (full supply level) plus the sum of wind set up and wave run up that would be generated by the highest 10 minutes sustained wind speed with a 1% AEP (1 in 100 year recurrence interval).

The regional three second gust wind speed for minimum freeboard with 1 in 10 year recurrence interval was 34 m/s. The highest regional three second gust wind speed for normal freeboard with 1 in 100 year recurrence interval was 41 m/s. They were assumed to act in all directions across the Waimakariri Ponds. The maximum fetch for Pond 1 is 940 m and for Pond 2 is 1,180 m.

7.3.2 Wave Run Up and Wind Set Up

Wave height (trough to crest), run-up and wind setup were estimated in accordance with Fell, et al (2015), subject to the condition that the reservoir is relatively deep compared to the wind generated wave length (L), i.e. water depth > 0.5L.

The significant wave heights; the average of the highest one third of waves in the wave spectrum, were calculated as 0.60 m and 0.48 m for normal and maximum freeboards respectively.

The maximum wave run-up was calculated as 1.10 m (1.08 wave run-up plus 0.02 wind setup) and 1.21 m (0.79 wave run-up plus 0.02 wind setup plus 0.40 m PMP surcharge) for normal and minimum freeboards respectively.

A 1.5 m freeboard, up to the top of 300 mm top surface was adopted which adequately contains wave run up and wind setup and rain reduced freeboards respectively, applicable to the maximum fetch of 1.18 km. This freeboard also accommodates head required for the overflow spillway and provides storage to contain an extreme rainfall event (PMP) as discussed in Section 4.3.

7.3.3 Other Influence of Freeboard Setting

The crest settlement expectations from an earthquake are discussed in Section 7.9. Maximum crest settlement is expected in the range of 243 to 409 mm with SEE and aftershock ground shaking, which is well within the freeboard adopted.

Earthquake induced ground surface tilting was addressed by GNS subsequent to the Expert Witness Conference on seismic effects on 16/09/15. The GNS report of these studies is included in Appendix I. GNS determined that ground surface tilting induced by a rupture of the Hororata fault would be less than 30 cm/km and in a NW SE direction.

First shock settlement is estimated to be 233 to 388 mm (see Table 32), which reduces freeboard to between 767 to 612 mm if the 300 mm crest topping is neglected. The greatest NW pond length is 800 m in Pond 2 resulting in an after settlement tilt of 77 to 96 cm/km for the pond to retain the reservoir if full. Such earthquake tilting would cause seiching in the pond which would overtop the embankment. However, although expected to erode the 300 mm crest topping material, would not cause failure of the pond embankment. The estimated tilt which Pond 2 can withstand without sustained overtopping which would cause embankment failure is significantly greater than earthquake induced tilting, of less than 20 cm/km, estimated by GNS resulting from rupture of the Hororata fault.

GNS also estimated that rupture on the Hororata fault could cause ground surface horizontal strain less than 10^{-5}. Which equates to a couple of centimetres over a line-length of 10 km. Such small strains can be accommodated by the pond embankments, the geomembrane lining and appurtenant structures without risk of damage.

7.4 Reservoir Slope Protection

Reservoir inside slope protection is provided by the geomembrane. Wave action may cause uplift tension stresses on the geomembrane by suction. However, these stresses are expected to be smaller than the stresses imposed by wind loading when the pond water level is low. Wind uplift on the geomembrane liner is addressed in Section 7.6.2.

7.5 Geomembrane Selection

7.5.1 Review of Industry Experience

In selecting the most appropriate geomembrane for the Wrights Road Storage Ponds, we first reviewed industry experience and practice in use of geomembranes in dams.

In the past, there have been a number of different geo-synthetic materials that have been used as the impervious layer in dams and canals. The International Commission on Large Dams, Bulletin No. 135 (ICOLD, 2010) lists 10 different polymers used in the geomembranes in more than 240 large dams around the world (refer to Table 13). In addition, ICOLD lists a further 23 dams in which bituminous products have been used as the impervious layer.

The use of PVC membranes accounts for 65% of the installations cited. Of these roughly equal numbers are installed as covered and exposed membranes. By contrast the other two popular membranes (LLDPE and HDPE) are typically installed in covered arrangements to provide protection against environmental conditions and mechanical damage or vandalism.

It is noteworthy, however, that there are many applications of exposed, particularly HDPE lining, used for pond liners. Table 13 summarises geomembrane types and usage in dams internationally.

Table 13: Use of Geomembranes in Dams (reproduced from ICOLD, 2010)

Material	Abbreviation	Total number of dams			Total
		Exposed	Covered	Unknown	
Polyvinylchloride – Plasticised	PVC-P	80	73	3	156
Linear Low Density Polyethylene	LLDPE	0	29	1	30
High Density Polyethylene	HDPE	3	12	1	16
Butyl rubber	IIR	5	4	2	11
Polyisobutylene	PIB				
Ethylene-propylene-diene monomer	EPDM				
Chlorosulfonated polyethylene	CSPE	3	5	1	9
Geotextiles impregnated with polymers	In situ membrane	2	7	0	9
Polyolefin	PP	3	3	0	6
Chlorinated polyethylene	CPE	0	3	0	3

The integrity of the seams between geomembrane panels is critical to the behaviour of the impervious layer. There are various means of providing the seals at the seams. Each is dependent on the properties of the geomembrane and the requirement for quality assured integrity. Each of the three common geomembranes can be sealed using thermal fusion. However, it is only PVC that permits the use of other jointing methods.

The advantages and disadvantages of the commonly used materials, namely HDPE, LLDPE and PVC, are summarised in general terms in Table 14.

It should be noted however that, because the PVC formulation can be enhanced with additives such as plasticizers, stabilisers and ageing retardants, it can be specifically formulated to meet different service conditions, such as resistance to UV, specific contaminants and very low temperature environments.

7.5.2 Geomembrane Durability and Longevity

ICOLD (2012) provides a summary of information of geosynthetic installations on dams as outlined in Table 15.

Table 14: Advantages and Disadvantages of Commonly Used Geomembranes (Fourie 2010)

Geomembrane	Advantages	Disadvantages
HDPE	Broad chemical resistance Good weld strength Good low temperature properties	Potential for stress cracking High degree of thermal expansion Poor puncture resistance Poor multi-axial strain resistance
LLDPE	Better flexibility than HDPE Better lay flat than HDPE Good multi-axial strain properties	Inferior UV resistance to HDPE Inferior chemical resistance to HDPE
PVC	Good workability and lay-flat behaviour Easy to seam Can be folded so fewer field fabricated seams	Poor resistance to UV and ozone unless specially formulated Poor resistance to weathering Poor performance at high and low temperatures

Table 15: Geomembrane Longevity in Dams (reproduced from ICOLD, 2012 Table 12)

Material	Abbreviation	Oldest installation (year)	
		Exposed	Covered
Polyvinylchloride – Plasticised	PVC-P	1974	1960
Linear Low Density Polyethylene	LLDPE	-	1970
High Density Polyethylene	HDPE	1994	1978
Butyl rubber	IIR	1982	1959
Polyisobutylene	PIB		
Ethylene-propylene-diene monomer	EPDM		
Chlorosulfonated polyethylene	CSPE	1981	1986
Geotextiles impregnated with polymers	In situ membrane		
Polyolefin	PP	1995	2000
Chlorinated polyethylene	CPE	-	1970

7.6 Physical Properties of Geomembranes

Based on assessment of cost and the physical properties listed in Table 16, HDPE has been adopted for the Wrights Road Storage Ponds.

Table 16: Geomembranes and Physical Properties Assessed

Geomembrane Type	Geomembrane Material	Physical Properties Considered
Polymeric	PVC-P LLDPE HDPE FPP EPDM Polymer coated geotextile	Impermeable to water Burst strength Puncture resistance Tear strength Frictional properties Jointing Durability
Bituminous	Not considered	

The design developed herein is based on 1.5 mm thick, smooth HDPE manufactured by Solmax Ltd in Malaysia. As Solmax have recently adopted white faced smooth liner, which provides improved laying and performance characteristics, as the standard liner that they produce, this Solmax 460W-9000 1.5 mm thick liner will be used. The physical properties of HDPE liner are summarised in Table 17.

Solmax regularly produce HDPE geomembrane with tensile properties (i.e. strength and elongation) greater than those shown on Solmax specification sheets for standard 1.5 mm thick smooth HDPE. The geomembrane liner for placement at the pond corners requires the enhanced tensile and extension properties shown in Table 17, and detailed in the Specification - Appendix F and Drawing WIL1125/30/24-R1. Samples from rolls received at site will be tested and rolls with enhanced tensile and extension properties will be selected for placement in the pond corners.

Table 17: Geomembrane Physical Properties

Property	Test Standard	Solmax standard value	Enhanced values for placement at pond corners	
		HDPE, smooth 1.5 mm thick	HDPE, smooth 1.5 mm thick	
Density (minimum)	ASTM D1505 or ASTM D792	0.940 g/cc	0.940 g/cc	
Tensile Properties (minimum based on 5 test specimens in each direction)	ASTM D6693 Type IV		Machine Direction	Cross Direction
Yield Strength		22 kN/m	24 kN/m	26 kN/m
Break Strength		42 kN/m	49 kN/m	50 kN/m
Yield Elongation		13%	18 %	16 %
Break Elongation		700%	800 %	800 %
Tear Resistance (minimum)	ASTM D1004	187 N	187 N	
Puncture Resistance (minimum)	ASTM D4833	540 N	540 N	

7.6.1 Geomembrane Installation

Solmax HDPE is manufactured in standard width of 8.0 m wide and 140 m long.

Placement of HDPE geomembrane is a well-established process, and is carried out by preparing the subgrade; then,

1. On the reservoir embankment slopes, laying initially a layer of Ecotech 500 g/m^2 geotextile, with lapped joints, over the prepared subgrade and into the anchor trench. The purpose of the geotextile is to protect the geomembrane from puncturing on embankment slopes.

2. Secondly laying the HDPE geomembrane liner over the geotextile and into the anchor trench, welding the panels with hot-wedge welding machines, backfilling of the anchor trench and ballasting the base.
3. On the pond bottoms, laying the liner over the subgrade, welding the panels with hot-wedge or extrusion welding machines; and application of ballast.

The embankment and pond bottom surface to be lined should be smooth and free of angular rocks, roots and debris. The embankment surface will be proof rolled. The prepared surface shall conform with the requirements summarised in Table 18. Voids created from the removal of protruding angular rocks shall be filled with finer AP20 screened material and the surface shall be proof rolled a second time.

Table 18: Surface Preparation Requirements

Requirement	Standard
Requirement for line and level	Undulations (up or down) shall be less than 100 mm
Requirement for no items likely to damage the liner	No foreign objects such as timber or steel and no broken greywacke rocks visible in the surface
Requirement for maximum rock fragment size	On pond bottom maximum bare rounded rock face < 6 mm On embankment slope maximum bare rounded rock face in the plane of the finished surface < 75 mm
Requirement for surface conditions	No pools or standing water

7.6.2 Wind up-lift of the Geomembrane

The highest three second gust wind velocity at the Wrights Road Storage Ponds site is 41 m/s, corresponding to a 100 year return period occurring in the north-westerly direction. This 41 m/s velocity was used to assess the wind uplift of the 1.5 mm thick HDPE geomembrane liners.

The calculated uplift requires the geomembranes to have support as the weight of the HDPE liner alone is not enough to counteract the site wind uplift. A soil cover layer over the embankment section of the geomembrane liner is not practical as it could be eroded by wind induced waves. The alternative liner support is to anchor the liner at the embankment toe and crest.

The tension-strain relationship of the geomembrane lining, and the ponds withstanding the effective wind suction on the slope lengths of the leeward and windward sides of embankments of each pond were found to be acceptable. It was found that each liner can withstand the effect of wind suction, if it is anchored at the embankment toe and crest without needing additional anchoring.

7.6.3 Groundwater Effect on Geomembrane

The Wrights Road Storage Ponds are not expected to have an effect on the groundwater level at the site as foundation soils are permeable and any leakage from the liner will have a general downward movement. Groundwater, at the estimated depth of 20 m, is not considered to impact the geomembrane lining. Piezometers are recommended to monitor for any change in groundwater conditions to evaluate the potential (negative) impact on the lining and thus embankment stability.

7.6.4 Anchorage of the Geomembrane

The geomembrane will be ballasted to anchor the geomembrane over the invert of the ponds and at the toe of the embankment slope. To prevent machinery tracking on the geomembrane, the most efficient placement of ballast was found to be by providing mounds of ballast on the pond invert and a strip of ballast at the toe of the embankment slopes (refer to Drawing WIL1125/30/21 to 23). An anchor trench, with a required area of 0.4 m^2, (0.5 m wide by 0.75 m deep trench) will anchor or hold the liner at the embankment crest.

7.6.5 Geomembrane Detail at Pond Lining Penetrations

The geomembrane will be fixed to the inlet, outlet and inter pond concrete transitions using the detail shown in Drawing WIL1125/30/129. This membrane fixing method will be adopted for all penetrations through the lining. Surface ballast is included at culvert penetrations to prevent potential damage by wind uplift at these locations, refer to Drawing WIL1125/30/130.

7.6.6 Lining Performance

HDPE

Lifetime performance of HDPE geomembrane depends significantly on whether the geomembrane is covered or exposed. The predicted lifetime for covered HDPE is 446 years when exposed to temperatures of 20 degrees (GRI, 2005). Exposed geomembrane lifetime is assessed based on the field performance of installed HDPE liners. Industry experience indicates that exposed HDPE liners are predicted to have lifetimes greater than 36 years (GRI, 2005).

Ten A3 paper size coupons of geomembrane liner material shall be welded to the geomembrane surface at the full supply level of the lining in Pond 1, allowing for 55 years of monitoring with testing of a coupon sample every 5 years, commencing at end of year 5.

A further, two A3 size coupons shall be welded to the geomembrane surface at the bottom of Pond 1 allowing for testing over 5 years at the time samples from full supply level result in a 20% decrease in strength and elongation properties, in turn indicating the need for liner maintenance.

The laboratory testing will include tensile properties (strength at yield, elongation at yield, strength at break and elongation at break, in accordance with ASTM D-6693), tear resistance in accordance with ASTM D-1004 and puncture resistance in accordance with ASTM D-4833. A decrease in any of these test results by greater than 20% from the specified value in Table 16 will signal the need for testing one of the tokens from the pond bottom. Based on these laboratory test results, which indicate the need for liner maintenance, a recommendation to this effect will be included in the CSR report and subsequently acted on by the Dam Manager.

7.7 Foundation Preparation

Foundation preparation shall require stripping of the overlying Silt or Organic Materials, and any other unfavourable materials as may be encountered, prior to placement of embankment fill. Removal of the Silt materials is required to ensure:

- Embankment settlements are minimised;
- No materials are left beneath the embankment, within which a potential failure surface could develop;
- No materials which are potentially liquefiable are left beneath the embankment.

With respect to the latter, loose and/or uniformly graded sand and gravel deposits are also to be considered as unfavourable material which should be removed from the footprint of the embankments.

7.8 Embankment Slope Stability

Two-dimensional limit equilibrium stability analyses were performed for the Wrights Road Storage Pond embankments with the computer program SLOPE/W using Morgenstern-Price method and entry/exit failure surfaces. Stability of the embankment was evaluated under static (steady state seepage) and pseudo-static (seismic) conditions. The embankment stability after the earthquake (post-seismic) was also evaluated. Factors of safety for theoretical failure surfaces that would intercept the embankment and foundation materials were investigated. The five critical slip surfaces are shown for each analysis along with contours of factors of safety. Results of stability trials are presented in Appendix D.

The four cross-sections of embankment profiles evaluated in the stability analyses are listed in Table 19. The “External Downstream Shoulder (Pond 1 western embankment)” cross-section (Case 1) is classified as a Medium PIC dam (refer to Figure 1 page v). All other embankment cross-sections (Case 2, 3, and 4) are classified as High PIC embankments. Embankment cross-section stability was evaluated in accordance with NZSOLD (2015) guidelines for a Medium and High PIC dams, respectively.

Table 19: Cross-sections evaluated in stability analysis

Case	Cross-section Location	Drawing Ref	PIC	Height (m)	Embankment Slope	
					Water side	Land side
1	External Downstream Shoulder (Western embankment only)	WIL1125/30/23 Section 5-5	Medium	8.0	1V:3H	1V:2H
2	Internal Upstream Shoulder	WIL1125/30/23 Section 7-7	High	9.4	1V:3H	-
3	Internal Downstream Shoulder	WIL1125/30/23 Section 7-7	High	14.6	Upper slope: 1V:2H Lower slope: 1V:3H	-
4	External Downstream Shoulder (All other external embankments)	WIL1125/30/23 Section 8-8	High	12.0	1V:3H	1V:2.5H

The western embankment (Medium PIC) consists of a 1V:3H upstream (water side) slope and a 1V:2H downstream (land side) slope, as indicated in Section 5-5 of Drawing WIL1125/30/23. All other external embankments (High PIC) have a 1V:3H upstream (water side) slope and a 1V:2.5H downstream (land side) slope. The internal embankment (High PIC) is located between Ponds 1 and 2, refer to Section 7-7 of Drawing WIL1125/30/23. The upstream slope in contact with Pond 1 is 1V:3H and the downstream slope in contact with Pond 2 is a composite consisting of a 1V:2H upper section and 1V:3H lower section.

Soil properties used in the stability analyses are presented in Table 11, as previously discussed in Section 6.5. The static stability analysis modelled the embankment at maximum storage capacity with a leak in the geomembrane liner for steady state seepage conditions. The pseudo-static analysis modelled the embankment at maximum storage capacity and no leakage in the geomembrane liner. Thus, a surcharge load of 10 kN/m^3 was used to represent the ponded water without applying pore water pressures (i.e. buoyant weights) to underlying soils.

7.8.1 Static (Steady State Seepage) Stability

Estimated FOS for the four embankment profiles are presented in Table 20 and shown in Figures D1, D2, D3 and D4, respectively (refer to Appendix D). Results of the static stability analyses show the estimated factor of safety (FOS) for the four embankment profiles is greater than 1.5. For embankment dams under long-term operating conditions with maximum storage pool and steady state seepage (static), NZDSG specify a minimum FOS of 1.5 for the downstream slope. Results of the static stability analyses, indicate the four embankment profiles meet the minimum NZSOLD criteria. The phreatic surface is based on the hydraulic connection of stored water between Pond 1 and Pond 2 for the internal shoulder embankment and hydraulic connection of stored water in Pond 1 and 2 to the loose gravel layer downstream for the external shoulder embankments.

Table 20: Factors of Safety for Static (Steady State Seepage) Condition

Case	Cross-Section Description	F_{Static}
1	External Downstream Shoulder (Medium PIC)	≥ 1.88
2	Internal Upstream Shoulder (High PIC)	≥ 2.51
3	Internal Downstream Shoulder (High PIC)	≥ 2.08
4	External Downstream Shoulder (High PIC)	≥ 1.96

7.8.2 Pseudo-Static Stability

Estimated factors of safety for critical failure surfaces were determined from pseudo-static stability analyses of the four embankment profiles. The pseudo-static analyses used the seismic coefficients; Peak Ground Acceleration (PGA) listed in Table 8 for Medium and High PIC embankments. For embankment dams under earthquake loading conditions, NZDSG specify dam performance criteria of: only minor damage is acceptable at the OBE loading condition, and under the SEE loading condition, some damage may occur, but the reservoir must be retained and damage must not lead to catastrophic failure.

OBE Loading

The factors of safety for the OBE loading condition are presented in Table 21 for the four profiles. Typical critical failure surfaces for the four embankment profiles under OBE loading condition are shown in Figures D5, D6, D7 and D8 (refer to Appendix D).

Table 21: Estimated Factors of Safety for Pseudo-Static Stability under OBE Loading Conditions

Case	Cross-Section Description	F_{Static}
1	External Downstream Shoulder (Medium PIC)	≥ 1.02
2	Internal Upstream Shoulder (High PIC)	≥ 2.31
3	Internal Downstream Shoulder (High PIC)	≥ 1.55
4	External Downstream Shoulder (High PIC)	≥ 1.05

The results show all the embankments performance with a FOS ≥ 1.00 meet the minimum NZSOLD criteria for the OBE loading condition. The critical slip surfaces shown with a FOS greater or equal to 1.00 indicate no or only minor damage will occur during the OBE earthquake. It may be noted that when the ponds are empty, lower factors of safety than those tabulated would apply to the stability of the internal embankment profiles. However, there is no hazard of uncontrolled release of impounded water associated with these cases. and thus are not considered further. The external embankments with steeper slopes at 2.0 - 2.5H:1V also provide a lower bound on the stability of the internal slopes at 3H:1V without ponded water.

Sensitivity of embankment stability to slip surfaces within upper embankment was also analysed under the OBE loading as a check of the dam performance. As discussed in Section 7.8.5 the potential failure surfaces within the upper half and one-third of the embankment were analysed for the external High PIC embankment and upper one-half of the external Medium PIC embankment. Figures D21 and D22 in Appendix D show the estimated failure surfaces within the upper half of the embankment at k_y for the external Medium PIC and High PIC slopes, respectively. Figure D23 shows the estimated failure surface within the upper third of the embankment at k_y for the external High PIC slope.

The k_y estimated for the upper failure surfaces are summarized in Table 22 and Table 22 and were used to estimate deformations under the OBE loading. Potential deformations from the upper slip surface were estimated by considering the dynamic response of the embankment and the maximum average acceleration (k_{max}) along the slip surface instead of PGA. The k_{max} is based on amplification factor of 2 times that at the crest (Yu et al. 2012). Values are summarised in Table 22 for the different embankment and slip surfaces depths.

No deformations are estimated from the upper slip surfaces subject to the OBE loading based on results of these sensitivity analyses. Table 22 provides a summary of the upper slip surface yield acceleration (k_y) and maximum average acceleration along the upper slip surface (k_{max}) for the OBE loading condition with a PGA of 0.31g. For all slip surfaces the k_y is higher than k_{max} which indicates no yielding and deformation along the upper slip surfaces. Thus, no deformation is estimated which meets NZSOLD criteria of no damage for the OBE.

Table 22: Summary of Sensitivity of Upper Embankment Stability for OBE

Case	Section	Depth	k_{yield} (g)	k_{max} (g)
1	External Downstream Shoulder (Medium PIC)	Upper half	0.45	0.37
4	External Downstream Shoulder (High PIC)	Upper half	0.48	0.37
		Upper third	0.57	0.5

SEE loading

For the SEE loading condition, NZDSG criteria require that some damage is allowable, but it must not lead to a catastrophic failure and release of the stored contents of the dam. Figure D9 to D12 show the critical slip surface at the yield acceleration (k_y) for each cross-section corresponding to a factor of safety equal to 1.0. The k_y for each cross-section are listed in Table 23 and compared to the SEE loading condition for each cross-section. The k_y estimated are lower than the SEE of 1.06 g for the High PIC dams and 0.77 g for the Medium PIC dam which indicates that deformation will occur under the SEE earthquake.

Table 23: Peak Ground Accelerations under the SEE and corresponding yield acceleration

Case	Cross-Section Description	SEE Loading (g)	k_y (g)
1	External Downstream Shoulder (Medium PIC)	0.77	0.32
2	Internal Upstream Shoulder (High PIC)	1.06	0.82
3	Internal Downstream Shoulder (High PIC)	1.06	0.59
4	External Downstream Shoulder (High PIC)	1.06	0.36

A k_y less than the SEE indicates a factor of safety less than 1.0 for the SEE loading condition. Therefore, some displacement is expected under the SEE loading. Performance of the embankment slopes under SEE loading has been assessed using estimated deformations based on the yield acceleration determined in the slope stability analyses, as discussed in Section 7.9.

7.8.3 Post-Seismic Aftershock Stability

Damage to the geomembrane liner is considered for the post-seismic (earthquake) static condition. Therefore, the embankment profiles include a phreatic surface for modelling seepage through the geomembrane. This is consistent with the static steady state seepage embankment profiles. Soils are not susceptible to liquefaction and therefore are not expected to lose strength thus the critical failure surfaces and minimum factor of safety are the same as shown in Figures D1, D2, D3 and D4 for the four embankment profiles considered.

Stability of the embankments during aftershocks was also evaluated. For these cases the seepage through the embankment was modelled. The k_y corresponding to a factor of safety equal to 1.0 was determined as shown in Figure D13, D14, D15 and D16. The k_y are listed in Table 24 and are used to estimate additional embankment deformations from aftershock earthquakes. The performance of the embankments due to aftershock deformation is discussed further in Section 7.9.

Table 24: Post-aftershock yield accelerations

Case	Cross-Section Description	k_y
1	External Downstream Shoulder (Medium PIC)	0.29 g
2	Internal Upstream Shoulder (High PIC)	0.21 g
3	Internal Downstream Shoulder (High PIC)	0.18 g
4	External Downstream Shoulder (High PIC)	0.29 g

7.8.4 Sensitivity of Embankment Stability to Shear Strength

Additional analyses were performed to evaluate sensitivity of embankment stability to fill shear strengths. Shear strength of the fill was decreased by 2° for a f' of 36° and cohesion (c) of zero was assigned to the embankment gravels. Sensitivity analyses were performed on the external Medium and external High PIC downstream slopes as they provide lower bound results than the internal embankment slopes. Loading conditions of static (steady state seepage) conditions and mainshock earthquake loading to determine the k_y were considered. Results of sensitivity analyses are summarised in Table 25.

Static stability of the embankment slopes with the lower shear strength meet NZDSG with FOSs greater than 1.5. The embankments were analysed to have a FOS of 1.85 for the Medium PIC slope and 1.90 for the High PIC slope, which is a slight decrease in FOS by 0.03 - 0.06. Figures D17 and D18 in Appendix D show the estimated failure surfaces for the external Medium PIC and High PIC slopes, respectively.

Results of the pseudo-static stability of the embankment slopes with the lower shear strength also had a corresponding slight decrease in estimated k_y. Analyses estimated a k_y of 0.30 g for the Medium PIC slope and 0.33 g for the High PIC slope. For the Medium PIC embankment with a k_y of 0.30g only minor repairable damage is expected to meet NZDSG performance criteria for the OBE. For the High PIC embankment with a k_y of 0.33g none to only minor repairable damage is also expected to meet NZDSG performance criteria for the OBE. The slight decrease in k_y were also considered in estimating deformations under the SEE, as discussed further in the following section. Figures D19 and D20 in Appendix D show the estimated failure surfaces at k_y for the external Medium PIC and High PIC slopes, respectively.

Table 25: Summary of Sensitivity Stability Analyses to Shear Strength

Case	Cross-Section Description	Static FOS	SEE k_y
1	External Downstream Shoulder (Medium PIC)	1.85	0.30 g
4	External Downstream Shoulder (High PIC)	1.90	0.33 g

7.8.5 Slip Surfaces within Upper Embankment

Additional analyses were performed to evaluate consequences of potential failure surfaces extending within the upper embankment under SEE loading. Potential failure surfaces within the upper half and one-third of the embankment were analysed for the external High PIC embankment and upper one-half of the external Medium PIC embankment. The approach taken was to determine the k_y for slip surfaces intercepting the upstream slope near storage level and extending to one-half and one-third the embankment height.

Result of k_y for estimated failure surfaces within the upper embankment are summarised in Table 26. The k_y was used to estimate deviatoric deformations for comparison against deformations estimated for full slope slip surface. The dynamic response of the embankment was accounted for in the estimated deformations of the upper slip surface as discussed in the following Section 7.9. Figures D21 and D22 in Appendix D show the estimated failure surfaces within upper half of the embankment at k_y for the external Medium PIC and High PIC slopes, respectively. Figure D23 shows the estimated failure surface within the upper third of the embankment at k_y for the external High PIC slope.

Table 26: Summary of k_y for slope stability analyses within upper embankment height

Case	Cross-Section Description	Slip Surface	SEE ky
1	External Downstream Shoulder (Medium PIC)	Upper half	0.45g
4	External Downstream Shoulder (High PIC)	Upper half	0.48g
		Upper third	0.57g

7.8.6 Summary

Stability analysis of the Wrights Road Storage pond embankments was performed with SLOPE/W. Four typical embankment profiles were considered, and results from the analysis are presented in Appendix D. The key conclusions from slope stability trials are that:

- Factors of safety for static (steady state seepage) stability trials were all greater than 1.5, and meet NZDSG, which specify that a minimum FOS of 1.5 is required for the downstream slope under operating conditions with maximum storage level.
- Factors of safety for pseudo-static stability under OBE loadings were all greater than 1.0, and meet NZDSG which specify that only minor damage is acceptable at the OBE loading condition.
- Factors of safety for pseudo-static stability under SEE loadings were all less than 1.0, indicating that deformation of the embankments is expected under the SEE loading. NZDSG state that under SEE loadings some damage is allowable, but it must not lead to catastrophic failure and release of the stored contents of the dam. The performance of the embankment design with a geomembrane lining is shown to meet the NZDSG criteria as discussed further in Section 7.9.
- Sensitivity of embankment stability to shear strengths illustrated the slopes are stable and meet NZDSG for static (steady state seepage) conditions and performance criterion for earthquake loading at the OBE level. When embankment material shear strengths were decreased to f of 36° and c of zero, the embankment slopes had a static FOS ≥ 1.85 and none to only minor repairable damage is expected under earthquake loading at the OBE level.
- Sensitivity of embankment stability to shear strengths resulted in a slight decrease in k_y under the SEE. This k_y was used to estimate seismic-induced deviatoric deformations and compared against estimated deformations used in design.
- Additional stability analyses were performed to evaluate consequences of potential slip surfaces within the upper half and one third of the embankment. Higher k_y were estimated for these upper slip surfaces. Seismic-induced deviatoric deformations were estimated as presented in the next section which account for the dynamic response of the embankment and were compared against estimated deformations used in design.

7.9 Earthquake Induced Embankment Settlement & Cracking

The estimate for settlement and cracks described in this section is based on experience with embankment dams which are rather different from the Wrights Road Storage ponds in the following ways:

- The dams are typically across valleys with variation in height along the length of the dam;
- The dams are typically zoned earth embankments with central low permeability high fines or clay cores and saturated upstream shoulders; and
- The low permeability core of the zoned embankments has less favourable strength characteristics than the free draining sandy gravels used in the Wrights Road Storage pond embankments.

With lining on the upstream side of the Wrights Road Storage pond embankments, the embankments are similar to concrete-face rock-fill dams which have an upstream water barrier and free draining embankment.

As a consequence of these differences the settlement and cracking estimates which follow are conservative.

The following sub-sections present:

- Assessment and analysis of earthquake induced embankment crest settlement and cracking for the Wrights Road Storage Ponds.
- Analysis of the embankment and geomembrane liners' ability to withstand earthquake induced embankment settlement and cracking.
- Mitigation design for transverse cracking to maintain embankment stability post-earthquake.

Current practice dictates considering an aftershock in addition to the main shock (the design earthquake). Aftershocks were taken as one moment magnitude (Mw) lower than the main shock per NZDSG (2015). Aftershock PGAs were estimated based on Mw and the epicentral distance being less than 10 km.

Earthquake induced embankment crest settlement and cracking for the external Medium PIC and High PIC embankments were estimated as they represent an upper bound on deformations. Deviatoric deformations for the internal embankments were estimated as presented in the following section and are significantly less than the external Medium PIC and High PIC embankments. Thus, assessment and analysis of earthquake induced crest settlement and cracking are provided for the external Medium PIC and High PIC embankments.

Earthquake induced embankment crest settlement and cracking were estimated using the following steps:

1. Determine the earthquake mainshock yield acceleration (k_y) for each section.
2. Estimate crest settlement from mainshock:

a. Estimate seismic mainshock induced deviatoric deformations using the semi-empirical method by Bray and Travasarou (Bray and Travasarou, 2007). Determine vertical component of deviatoric deformations for calculating crest settlement.

b. Estimated seismic compression settlement due to mainshock loading using method by Tokimatsu and Seed (Tokimatsu, 1984) for calculating crest settlement.

c. Calculate crest settlement from mainshock as the sum of the vertical component of deviatoric deformation and compression settlement.

3. Repeat steps 1 through 2 for aftershock loading.
4. Sum the crest settlements from mainshock and aftershock.
5. Determine the "Damage Class" based on crest settlement normalised by embankment height.
6. Estimate the longitudinal crack widths based on "Damage Class" using Pells and Fell's method (Pells and Fell, 2002).
7. Estimate maximum transverse cracking dimensions based on "Damage Class" using the method outlined in Geotechnical Engineering of Dams, 2nd edition (Fell et al, 2015).

7.9.1 Crest Settlement

Seismic induced crest settlement from the earthquake loading consists of two components: (i) deviatoric-induced deformations and (ii) volumetric-induced compression. Deviatoric-induced deformations along the failure surface were estimated by methods by Bray & Travasarou (Bray & Travasarou, 2007). Deviatoric deformations include a horizontal and vertical component of slope movement. The vertical component of deviatoric deformations corresponds to crest settlement and was determined based on a portion of the slopes at 2H:1V for the external Medium PIC embankment, 3H:1V for the internal High PIC embankment and 2.5H:1V for the external High PIC embankment. The volumetric-induced deformation is the seismic compression and settlement under earthquake shaking. Methods by Tokimatsu and Seed (Tokimatsu, 1984) for sands were used to estimate the earthquake-induced compression settlement.

Deviatoric Deformations by Bray & Travasarou (2007)

Methods by Bray & Travasarou (2007) were used to estimate deviatoric-shear deformations along a failure surface because it is based on the Newmark model, but also captures the dynamic nonlinear performance of earthfill embankments. The method uses the k_y, earthquake magnitude (M), embankment fundamental period (Ts), and the spectra acceleration at a degraded embankment period [Sa(1.5Ts)] using Figure 15 for the mainshock deformations based on the embankments having a Ts ≥ 0.05s and the PGA for aftershock deformations. The k_y used are listed in Table 26 and 27 for the mainshock and aftershock, respectively. The earthquake M is 7.0 for the SEE mainshock and 6.0 for the aftershock. Table 27 lists the Bray and Travasarou input parameters for the external Medium PIC (case 1), internal upstream and downstream High PIC (case 2 & 3) and High PIC (case 4) embankment sections. Input parameters for the internal embankment sections are the same based on the same PIC category and Ts.

The embankments Ts is equal to 2.6*(H/Vs) where H is the embankment height and Vs is the average shear wave velocity. A representative Vs value of 300m/s for compacted gravels was used in this assessment to estimate Ts of 0.07 to 0.13s for the short embankments.

Table 27: Parameters Used in Methods by Bray and Travasarou (2007)

Case	Description	Height (m)	Ts (s) *(Note 1)*	1.5Ts (s)	Mainshock Sa (1.5Ts), (g)	Aftershock PGA, (g)
1	External Downstream Shoulder (Medium PIC)	8	0.07	0.10	1.18	0.40
2 & 3	Internal Upstream & Downstream Shoulder (High PIC)	14.6	0.13	0.19	2.45	0.50
4	External Downstream Shoulder (High PIC)	12.0	0.10	0.16	2.45	0.50
Notes: *(1) based on V_s of 300 m/s*						

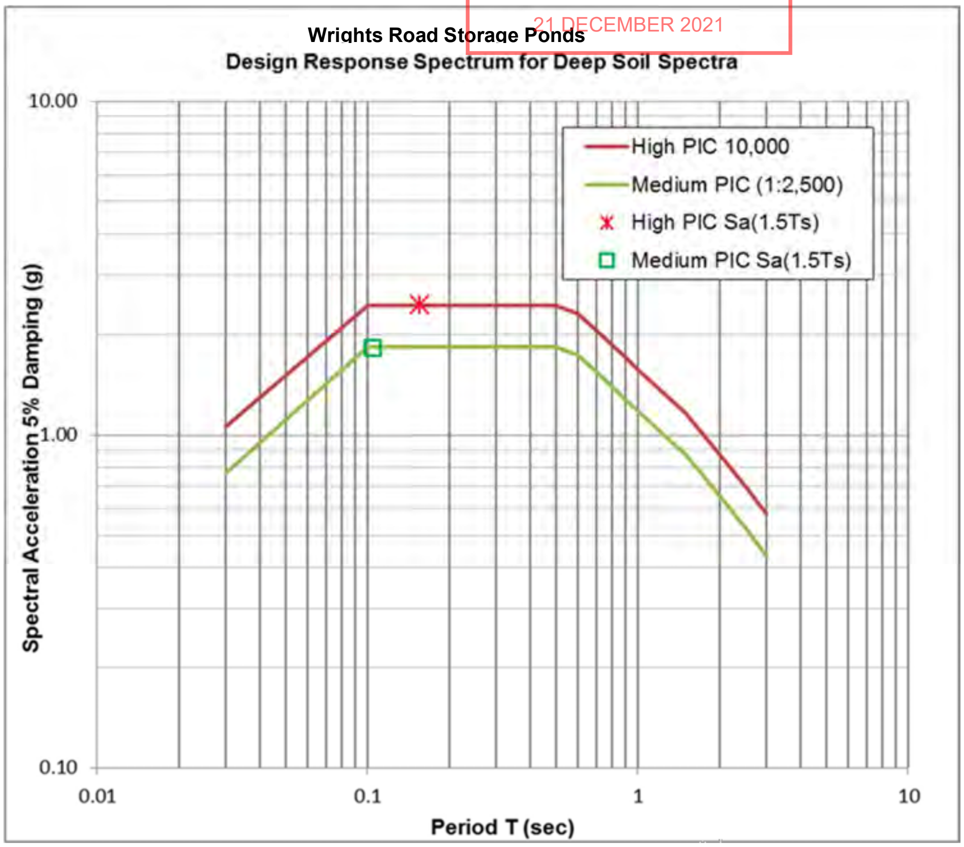


Figure 15: Spectral Accelerations for Estimating Deformations of High and Medium PIC Embankments

Table 28 and Table 29 summarise the mainshock and aftershock deviatoric deformations estimated using Bray & Travasarou (2007) method for the embankment sections. Estimates are provided as a mean (m), and mean plus and minus one standard deviation (m ±s). Estimated deformations at (m +s) were considered appropriate for assessing the embankment performance under the SEE mainshock and aftershock loading.

It was necessary to consider whether the embankment becomes saturated by the main shock resulting from rupture of the geomembrane on the upstream face. The embankment was assumed to be saturated below a phreatic surface in the results presented in Table 29. The saturated embankment tended to lower the yield acceleration in the aftershock stability trials. This resulted in additional deviatoric deformations, especially for the internal embankments with lower k_y.

Table 28: Summary of Mainshock Deviatoric Deformations

Case	Section	Deviatoric Deformations (mm)		
		(m - s)	(m)	(m + s)
1	External Downstream Shoulder (Medium PIC)	190	370	714
2	Internal Upstream Shoulder (High PIC)	41	79	152
3	Internal Downstream Shoulder (High PIC)	81	156	302
4	External Downstream Shoulder (High PIC)	122	236	457

Table 29: Summary of Aftershock Deviatoric Deformations

Case	Section	Deviatoric Deformations (mm)		
		(m - s)	(m)	(m + s)
1	External Downstream Shoulder (Medium PIC)	6	12	22
2	Internal Upstream Shoulder (High PIC)	23	45	86
3	Internal Downstream Shoulder (High PIC)	32	62	120
4	External Downstream Shoulder (High PIC)	11	21	41

Vertical Component of Deviatoric Deformations by Bray & Travasarou (2007)

The vertical component of deviatoric deformations listed in Figure 7.23 corresponds to crest settlements based on the embankment slope of 2.0H:1V for the external Medium PIC embankment, 3.0H:1V for internal High PIC embankments, and 2.5H:1V for external High PIC embankment. The vertical component is 37%, 32% and 45% of the total (m + s) deviatoric deformation, respectively.

Based on the vertical component of the deviatoric deformations, the external embankments provide an upper bound for estimating earthquake induced crest settlements and cracking. The total vertical component of deviatoric deformations from mainshock and aftershock loading of the internal upstream and downstream High PIC embankments were compared. The internal upstream deformations are 27 to 60% of the external embankment deformations. The upstream embankment deformation is considerably lower than that for the external embankment deformation. Thus, the following sections provide results of the assessment of earthquake induced crest settlements and cracking for the external embankments (case 1 & 4) only.

Table 30: Summary of Mainshock and Aftershock Vertical Component of Deviatoric Deformations

Case	Section	Vertical Component (mm)(m + s)	
		Mainshock	Aftershock
1	External Downstream Shoulder (Medium PIC)	206	10
2	Internal Upstream Shoulder (High PIC)	48	27
3	Internal Downstream Shoulder (High PIC)	96	38
4	External Downstream Shoulder (High PIC)	264	15

Seismic Compression Settlement using Tokimatsu and Seed (1984)

Seismic compression settlement was estimated using methods by Tokimatsu and Seed (1984). The amount of seismic compression depends on the density of material, the amplitude of the cyclic shear strain induced in the material and the number of cycles of shear strain applied during the earthquake.

The following calculations were performed to estimate the seismic compression settlement:

1. Calculate the maximum shear modulus Gmax based on Vs = 300m/s and embankment unit weight of 20 kN/m^3.
2. Divide embankment into sublayers, for example, the external High PIC embankment was divided into six layers each 2 m thick for a total of 12 m height.
3. For each sublayer:
    a. Calculate mean confining stress (s'm) for each layer based on unsaturated embankment conditions, coefficient of earth pressure at rest (K_0) equal to one and embankment unit weight of 20 kN/m^3.
    b. Calculate parameter (geff)(Geff/Gmax) =0.65*(PGA/g)(s'm*rd)/(Gmax) where rd = 1 for shallow surfaces and PGA corresponds to the spectral acceleration (Sa) at each layer using a linear interpolation from ground surface (PGA) to the crest (Sa,crest) to account for dynamic response of the embankment.. The Sa,crest was estimated using a crest amplification factor of 2 times based on relationship by Yu et al. (2012). For the High PIC embankment Sa for each layer was a linear interpolation from PGA of 1.06g to Sa,crest of 2.12g. For the Medium PIC embankment Sa for each layer was a linear interpolation from PGA of 0.77g to Sa,crest of 1.54g.
    c. Estimate the cyclic induced shear strain (geff) using Figures 16 and 17 for the external Medium and High PIC embankments, respectively.
    d. Estimate the volumetric strain (e_c) at M7.5 based on cyclic shear strain using Figures 18 and 19 for the external Medium and High PIC embankments, respectively.

   e. Adjust volumetric strain ($e_{c,M=7.5}$) from M7.5 to $e_{c,M=7.0}$ at M7.0 for mainshock using Table 30.

   f. Double $e_{c,M=7.0}$ to account for multidirectional shaking.

   g. Calculate each sublayer settlement by multiplying $2*e_{c,M=7.0}$ by sublayer thickness.

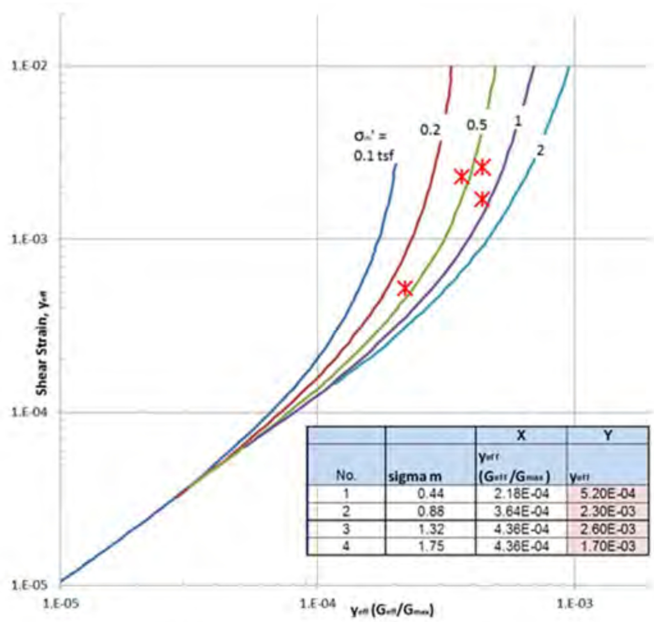
4. Estimate compression settlement by summing each sublayer settlement.

The calculated compression settlements for the mainshock loading are listed in Table 31 for the embankment sections. Differences in estimated compression settlement between the High PIC embankment and Medium PIC from embankment height and PGA values corresponding to 1:10,000AEP and 1:2,500AEP, respectively.

Influence of Earthquake magnitude on Volumetric Strain is summarised in Table 31.

The estimated total crest settlement (deviatoric and compression) for the mainshock loading is summarised in Table 32.

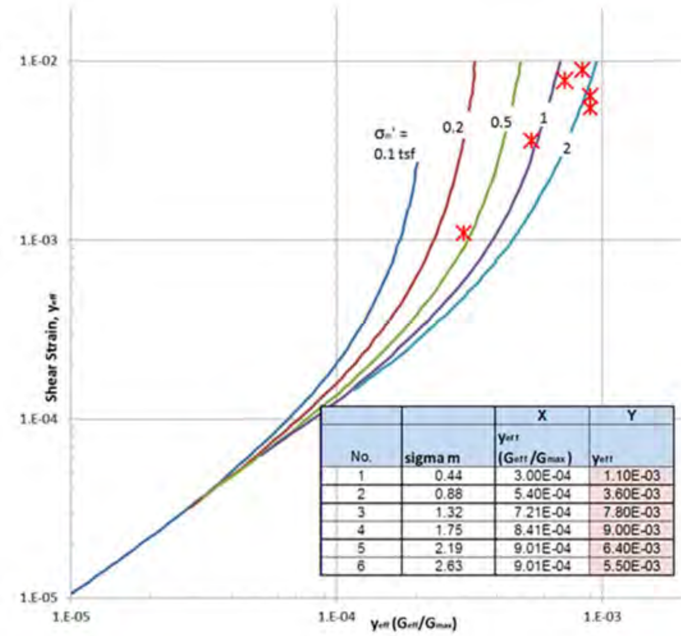
The seismic compression settlement from aftershock loading was estimated using similar methodology and aftershock PGA values of 0.40 and 0.50 for the external Medium PIC and High PIC embankments, respectively. Estimated seismic compression settlement from the aftershock loading is summarised in Table 33.



		X	Y
No.	sigma m	γeff (Geff /Gmax)	γeff
1	0.44	2.18E-04	5.20E-04
2	0.88	3.64E-04	2.30E-03
3	1.32	4.36E-04	2.60E-03
4	1.75	4.36E-04	1.70E-03

Determination of induced strain (modified Figure 12 Tokimatsu and Seed 1984)

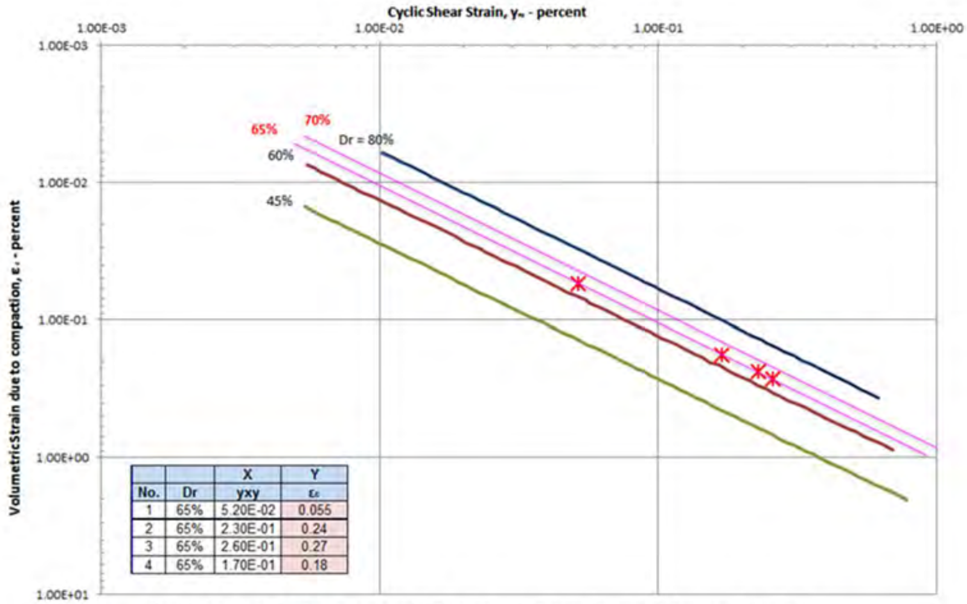
Figure 16: Determination of Induced Cyclic Strain for External Medium PIC Embankment (modified fromTokimatsu and Seed 1984)



		X	Y
No.	sigma m	γ_{eff} (G_{eff}/G_{max})	γ_{eff}
1	0.44	3.00E-04	1.10E-03
2	0.88	5.40E-04	3.60E-03
3	1.32	7.21E-04	7.80E-03
4	1.75	8.41E-04	9.00E-03
5	2.19	9.01E-04	6.40E-03
6	2.63	9.01E-04	5.50E-03

Determination of induced strain (modified Figure 12 Tokimatsu and Seed 1984)

Figure 17: Determination of Induced Cyclic Strain for External High PIC Embankment (modified fromTokimatsu and Seed 1984)



		X	Y
No.	Dr	yxy	εc
1	65%	5.20E-02	0.055
2	65%	2.30E-01	0.24
3	65%	2.60E-01	0.27
4	65%	1.70E-01	0.18

Modified Figure 13 of Tokimatsu and Seed 1984 Relationship between Volumetric Strain and Shear Strain

Figure 18: Relationship between Volumetric Strain and Shear Strain for External Medium PIC Embankment (modified fromTokimatsu and Seed 1984)

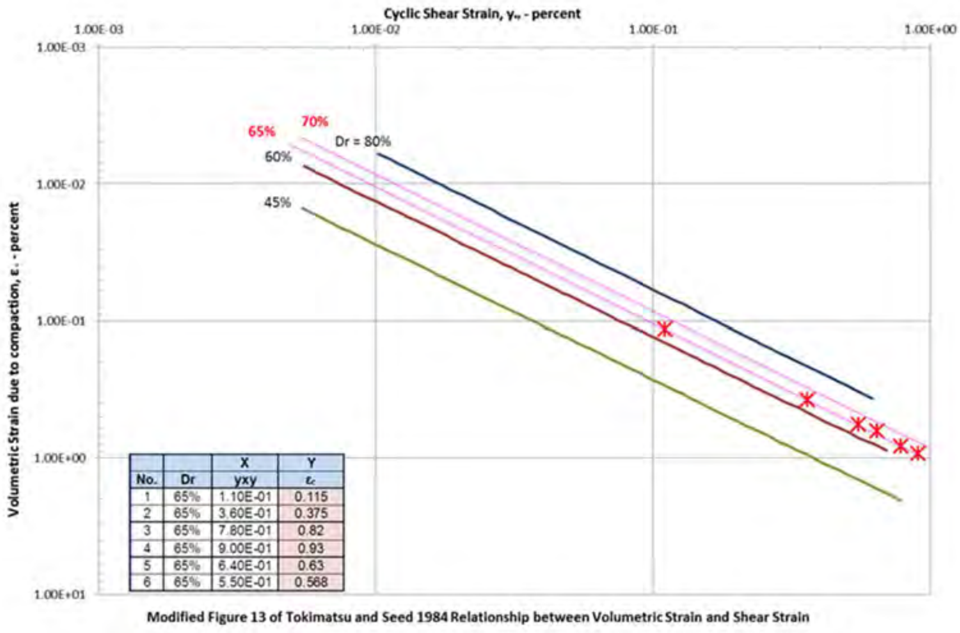


Figure 19: Relationship between Volumetric Strain and Shear Strain for External High PIC Embankment (modified fromTokimatsu and Seed 1984)

Table 31: Influence of Earthquake magnitude on Volumetric Strain (modified from Tokimatsu and Seed 1984)

Earthquake Magnitude, M	$\varepsilon c,M/ \varepsilon c, M=7.5$
5.25	0.4
6.0	0.6
6.75	0.85
7.0	0.90
7.5	1.0
8.5	1.25
Notes: Linearly interpolated	

Table 32: Summary of Crest Settlement from Mainshock

Case	Section	Vertical Deviatoric Component (mm)	Compression Settlement (mm)	Mainshock Crest Settlement (mm)
1	External Downstream Shoulder (Medium PIC)	206	27	233
4	External Downstream Shoulder (High PIC)	264	124	388

Table 33: Summary of Crest Settlement from Aftershock

Case	Section	Vertical Deviatoric Component (mm)	Compression Settlement (mm)	Aftershock Crest Settlement (mm)
1	External Downstream Shoulder (Medium PIC)	10	0	10
4	External Downstream Shoulder (High PIC)	15	6	21

Crest settlement estimates are reported in Table 34. SEE (main shock), aftershock and total crest settlement estimates are reported therein. The normalised crest settlement as a percentage of dam height is also reported because this is the parameter with which longitudinal and transverse cracking are empirically estimated. No crest settlements or cracking were estimated for the OBE scenario because the pseudo-static factor of safety was greater than 1.00 as previously discussed in Section 7.8.1.

Earthquake induced crest settlements should be considered conservative for the reasons listed previously at the beginning of this section.

Table 34: Earthquake Crest Settlement, Longitudinal and Transverse Crack Estimates

Case (refer to Table 18)	1	4
PIC	Medium	High
Moment Magnitude, Mw (Main shock / Aftershock)	7.0 / 6.0	7.0 / 6.0
PGA (g) (Main Shock / Aftershock)	0.77 / 0.40	1.06 / 0.50
Case	1	4
Embankment Height (m)	8.0	12.0
Slope Angle	1V:2H	1V:2.5H
Slope Type	External D/S	External D/S
Crest Settlements (Bray and Travasarou 2007, Pells and Fell 2002)		
SEE Crest Settlements (mm)	233	388
Aftershock Crest Settlements (mm)	10	21
Total Crest Settlements (mm)	243	409
Settlement/Height (%)	3.04	3.41
Damage Classification	4	4
Longitudinal Cracks (Pells and Fell 2002)		
SEE Main Shock Crack Width (mm)	291	323
Main and Aftershock Separation Width (mm)	304	341
Transverse Cracks (mainshock and aftershock considered)		
Crack Top Width [UNICIV R 446 2008] (mm)	175	175
Selected Crack Depth within Upper Third (m)	2.67	4
Design Freeboard to FSL (m)[1]	1.2 *(Note 1)*	1.2 *(Note 1)*
Post-SEE & Aftershock Freeboard (m)	0.97 (Note 1)	0.82 (Note 1)
Estimated Post-SEE & Aftershock Water Depth in Crack (m)	1.90	3.39
Estimated Mean Submerged Crack Width (mm)	62	74
Notes: *(1) Omits 300 mm capping*		

[1]Not including 300mm maintenance strip on crest

Sensitivity of Embankment Stability and Estimated Deformations to Shear Strength

Results of k_y from slope stability sensitivity analyses with the embankment having lower shear strengths of f=36° were used to estimate deformations from SEE loading as a check against those presented in Table 34 developed for design. Table 35 shows this comparison of the estimated vertical component of deviatoric deformations. There is a corresponding increase in deformations from the lower shear strength and estimated yield acceleration for a net change of 23mm for the Medium PIC embankment and 40mm for the High PIC embankment.

The net increase in crest settlement from the lower embankment shear strengths under the SEE loading would not result in loss of freeboard. Similarly, aftershock deformations would increase slightly. However, given the design estimates are on the order of 10 to 15mm, a slight increase in these values would not result in a significant change in estimated deformations and no loss of freeboard is expected.

The incremental increase in normalised crest settlement (DD/H) from lower embankment strengths would not cause a change in the damage class used to estimate longitudinal and transverse cracking. The net increase in vertical component of deviatoric deformations normalised by embankment height is 0.30% for the Medium PIC embankment and 0.33% for the High PIC embankment as listed in Table 35.

Table 35: Sensitivity of Vertical Component Deviatoric Deformations to Shear Strength

Case	Section	Vertical Component (mm) (m + s)		Net Difference DD (mm)	(DD/H)
		Design (f=38°)	Sensitivity (f=36°)		
1	External Downstream Shoulder (Medium PIC)	206	229	+23	+0.30 %
4	External Downstream Shoulder (High PIC)	264	304	+40	+0.33 %

Sensitivity of Embankment Stability and Estimated Deformations to Slip Surface

Results of k_y from stability analyses of the upper embankment slip surfaces were used to estimate deformations from SEE as a sensitivity check against those presented in Table 34 developed for design. Deviatoric deformations were estimated using Newmark methods by Bray & Travasarou (2007) based on the upper embankment having a Ts<0.05s. The maximum average acceleration (k_{max}) along the slip surface was used instead of PGA and estimated by accounting for the dynamic response of the embankment using methods by Makdisi and Seed (1977) and relationships by Harder et al. (1998) and Yu et al. (2012).

The k_{max} along the slip surface at a specified depth is a function of the maximum crest acceleration (u_{max}). The u_{max} was estimated using an amplification factor of 1.5 to 2 times the PGA based relationships developed by Yu et al. (2012) and Harder et al. (1998), respectively, from case histories of dam performance under earthquake loading.

Methods by Makdisi and Seed (Makdisi et al, 1977) were used to determine k_{max} of the potential slip surface at the specified depths. The k_{max} was based on the maximum acceleration ratio (k_{max}/u_{max}) at depths corresponding to the upper half (0.5) and upper third (0.3) using the average of all data curve in Figure 20. The k_{max}/u_{max} are 0.6 and 0.8 at the respective depths.

Estimated vertical component deviatoric deformations from slip surfaces within the upper embankment are significantly less than those estimated for full slope slip surfaces used in design. Table 36 provides a summary of dynamic properties of acceleration and calculated vertical component of deviatoric deformations from slip surfaces within the upper half and upper third.

For slip surfaces within the upper half of the embankment estimated vertical component of deviatoric deformations range from 44 to 71mm for the external Medium PIC embankment and

High PIC embankment, respectively, with a crest amplification factor of 2 times applied. These deformations are 21-27% of full slope vertical component deviatoric deformations.

For slip surface within the upper third of the embankment, estimated vertical component of deviatoric deformations ranged from 47 to 100 mm for the external High PIC embankment with a crest amplification factor of 1.49 and 2 applied, respectively. These deformations are 23-38% of the crest amplification factor of full slope vertical component deviatoric deformations.

Table 36: Summary of Vertical Component Deviatoric Deformations to Slip Surface for SEE

Case	Section	Depth	Crest Ampli-fication	kmax (g)	Vertical Component (mm) (m + s)	% of Full Slope (mm)
1	External Downstream Shoulder (Medium PIC)	Upper half	2	0.92	44	21%
4	External Downstream Shoulder (High PIC)	Upper half	2	1.27	71	27%
		Upper third	1.49-2.0	1.26-1.70	47-100	23-38%

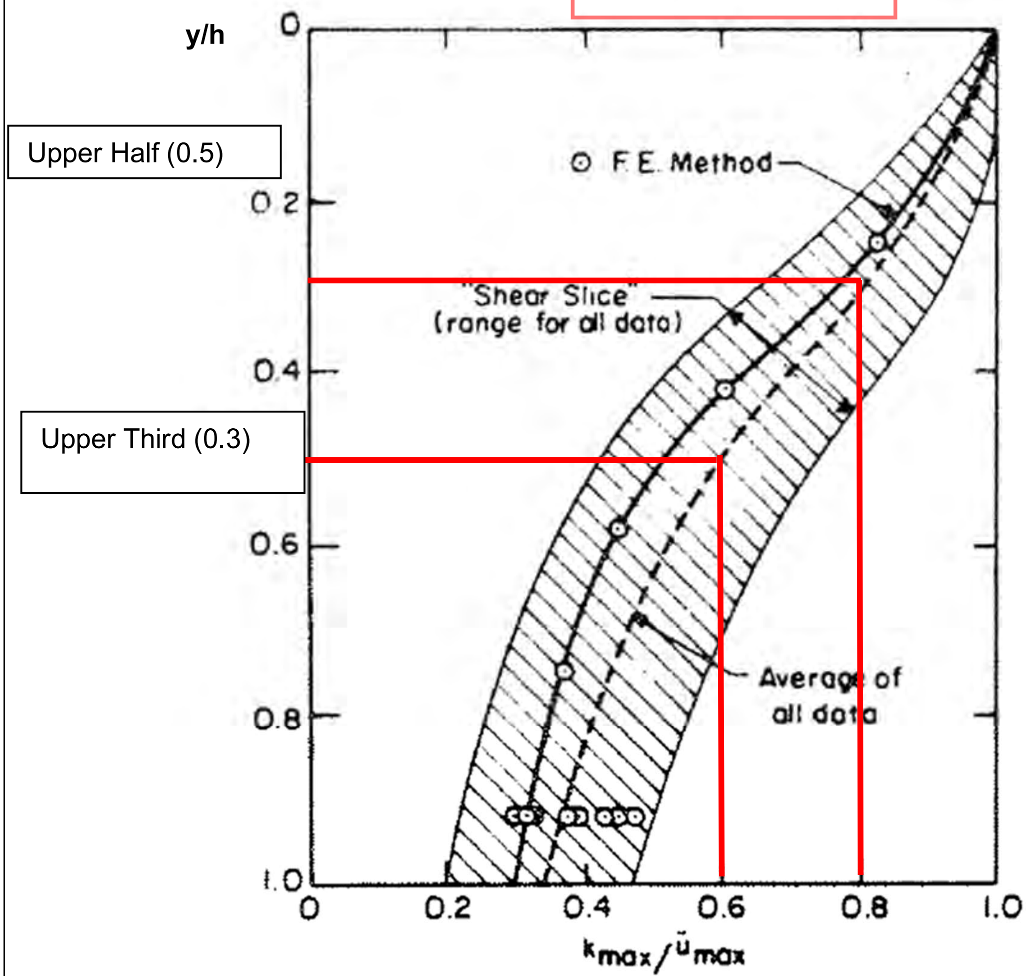


Figure 20: Variation of "Maximum Acceleration Ratio with Depth of Sliding Mass (Makdisi and Seed 1977)

Embankment Resistance to Crest Settlement

Table 34 indicates that total crest settlement in the order of 243 to 409 mm are estimated to occur as a result of high accelerations from the SEE and aftershock earthquake loading, giving normalised crest settlements in the order of 3.04% to 3.41% of the embankment height. Settlement of this magnitude at the embankment crest is less than the available freeboard of 1.2 m (excluding crest capping of 0.3 m) and no uncontrolled release of ponded water is expected due to crest settlement.

7.9.2 Longitudinal Cracking

Case histories of embankment dam response to earthquakes indicate longitudinal cracking along the dam crest and upper upstream face is common. The Wrights Road Storage Pond embankments are low height, resulting in being relatively stiff under earthquake dynamic loading. They have short natural periods, in the order of 0.1 second, resulting in resonance and high spectral accelerations at the embankment crest. Thus, it is considered longitudinal cracking will likely occur along the upper upstream face.

During the main shock (SEE) longitudinal cracking has been estimated according to Pells and Fell (2002) empirical method based on normalised crest settlement. These are reported in Table 34. These main shock crack widths are maximum, estimated, unsupported embankment separation. Cracks within the compacted sandy gravel embankments, Zone 4 material, would not be expected to hold an open crack for an extended period. An open crack in Zone 4 material will collapse with time and result in a depression on the surface.

For an aftershock scenario, it is necessary to assume the embankment could potentially deform further in tension. Given the further crest settlements, additional cracking was estimated. It is reported as ‘Main and Aftershock Separation Width’ in Table 34. It is likely the crack will deform and collapse at least partially between the SEE and aftershock so this refers to the separation distance.

Embankment Resistance to Longitudinal Cracking

Table 34 indicates that the maximum longitudinal cracks, in the order of 304 mm to 341mm, may occur under SEE and aftershock ground motions. If the geomembrane liner also fails, there is a risk of piping failure leading to an uncontrolled release of stored water.

The worst case is assumed, where the geomembrane and geotextile cracks in the same order as the earthquake induced longitudinal crack in the embankment, and that seepage into the embankment occurs. The rate of seepage occurring through the longitudinal crack was estimated using methods published by Giroud & Bonaparte (1989) and Giroud, et al (1989) and presented in Table 37. For this analysis it is assumed that the crack develops in the top third of the upstream embankment (i.e. maximum water depth over crack is approximately 3 m), the cracked area is 0.34 m (along the slope) x 1.0 m (transverse to the slope). The hydraulic conductivity of the Zone 4 embankment material is estimated to be in the order of 3 x 10^{-2} to 3 x 10^{-3} m/s based on empirical equations (e.g. Sherard et al, 1984) and is consistent with the general range of permeability for sandy gravels indicated in Fell, et al (2015). Results from Table 37 indicate that seepage through the crack is sensitive to the hydraulic conductivity of the Zone 4 material and in the order of 38 to 207 l/s/m.

A SEEP/W model was constructed to further investigate seepage resulting from longitudinal cracking. SEEP/W is a finite element software package developed by GEO-SLOPE International Limited for analysis of groundwater seepage. Four SEEP/W model scenarios were analysed as outlined in Table 38. Scenarios 1 and 2 considered homogenous permeability of the Zone 4 embankment ranging from 3 x 10^{-2} to 3 x 10^{-3} m/s. The same permeability was assumed for the sandy gravel foundation material. Scenarios 3 and 4 considered anisotropic permeability of the Zone 4 embankment and foundation material with a ratio of horizontal to vertical permeability (k_h/k_v) of 10 assumed (a conservative estimate for compacted embankment fill based on ranges published in USBR, 2014 (USBR, 2014)). For all scenarios it was conservatively assumed that two cracks develop in the top third of the upstream embankment (i.e. one crack at 1.5 m and another at 3 m below Full Supply Level), the cracked area is 0.34 m (along the slope) x 1.0 m (transverse to the slope). As outlined in Section 6.2, the groundwater table is approximately 20 m below ground level and this was applied as a boundary condition in the SEEP/W model.

Table 38 provides seepage estimates from the SEEP/W model immediately downstream of the crack in the upstream embankment slope, and the total seepage exiting on the downstream embankment slope and toe. This table shows that seepage in the order of 10 to 1000 l/s/m are estimated through the crack and are highly sensitive to assumptions regarding embankment permeability. At the downstream slope and toe of the dam seepage reduces to the order of 1 to 13 l/s/m as the SEEP/W model shows that seepage from the crack is drained near vertically towards the groundwater table.

Table 37: Estimated seepage through longitudinal crack using Giroud & Bonaparte (1989); Giroud, et al (1989) method

Scenario	Hydraulic conductivity of Zone 4 (k_h), (m/s)	Seepage through longitudinal crack on upstream slope (l/s/m)
1	3 x 10^{-3}	38
2	3 x 10^{-2}	207

Table 38: Estimated seepage due to longitudinal crack using SEEP/W

Scenario	Hydraulic conductivity of Zone 4 and foundation (k_h) (m/s)	Horizontal to vertical permeability ratio (k_h/k_v)	Seepage	
			Through longitudinal cracks on upstream slope (l/s/m)	Downstream embankment slope and toe (l/s/m)
1	$3x10^{-3}$	1	80.7	1.2
2	$3x10^{-2}$	1	807.1	12.7
3	$3x10^{-3}$	10	16.2	1.2
4	$3x10^{-2}$	10	162.2	12.4

Embankment resistance to leakage instability is predominantly a function of the downstream slope angle, mean particle diameter, and leakage discharge exiting the downstream slope and toe. Scandinavian researchers developed empirical methods for design of drainage buttresses to prevent toe unravelling based on large scale tests. Bartsch and Nilsson (Bartsch and Nilsson, 2007) provide an empirical relationship of mean rock particle size (D_{50}) to discharge flow based on these test results.

If leakage through cracks saturates the Wrights Road Storage Pond embankments, it could exit the downstream face with seepage rates indicated in Table 38. The allowable leakage discharge capacity of Zone 4 materials was estimated using Bartsch and Nilsson (2007) for the two cases as summarised in Table 39 The mean particle diameter, D_{50}, of Zone 4 embankment materials ranges between about 20 mm and 30 mm based on six gradation tests. We used a mean D_{50} of 23 mm to estimate leakage discharge capacity. The downstream slope angle is fixed by design and ranges between 1V:2H and 1V:2.5H for Cases 1 and 4 respectively.

Table 39 indicates that seepage in the order of 20 to 30 l/s/m are unlikely to lead to downstream slope unravelling of the Zone 4 embankment materials. Table 38 indicates that seepage rates through downstream face of the embankment are up to 13 l/s/m. Therefore, the Zone 4 material is expected to be stable under seepage from longitudinal cracks and no uncontrolled release of pond water is expected.

Table 39: Allowable seepage discharge for Zone 4 material

Parameter	Embankment Cross-Section (refer to Table 18)	
	Case 1	Case 4
Mean particle size $D_{50,\ mean}$ (m)	0.023	0.023
Slope angle parameter S_0 [1V:S_oH] (-)	1:2.0	1:2.5
Seepage discharge (l/s/m)	22.4	25.3

7.9.3 Transverse Cracking

Transverse cracks are especially hazardous to water-retaining embankments because they present an open pathway across the embankment that can potentially quickly erode and downcut, leading to breach. There is limited case history knowledge of transverse cracking on embankment dams, and very little applicable to enclosed embankment ponds. Figure 30 of Pells and Fell (2002) plots embankment dam cases for which both longitudinal and transverse cracks occurred, along with cases where only longitudinal cracks were reported. This figure is reproduced as Figure 21.

Most cases above M_w ~ 6.75 and PGA > 0.3g reported transverse cracking in addition to longitudinal cracking. Given the SEE ground motions having a moment magnitude (M_w) estimated at 7.0 and PGAs of 0.77g and 1.06g, the design ground motions indicate transverse cracking is possible.

Pells and Fell (2002) summarise embankment dam cracking influences in ‘Table 7, Influence of factors in the likelihood of cracking or hydraulic fracturing – feature giving low stress conditions’. This table is reproduced as Table 40.

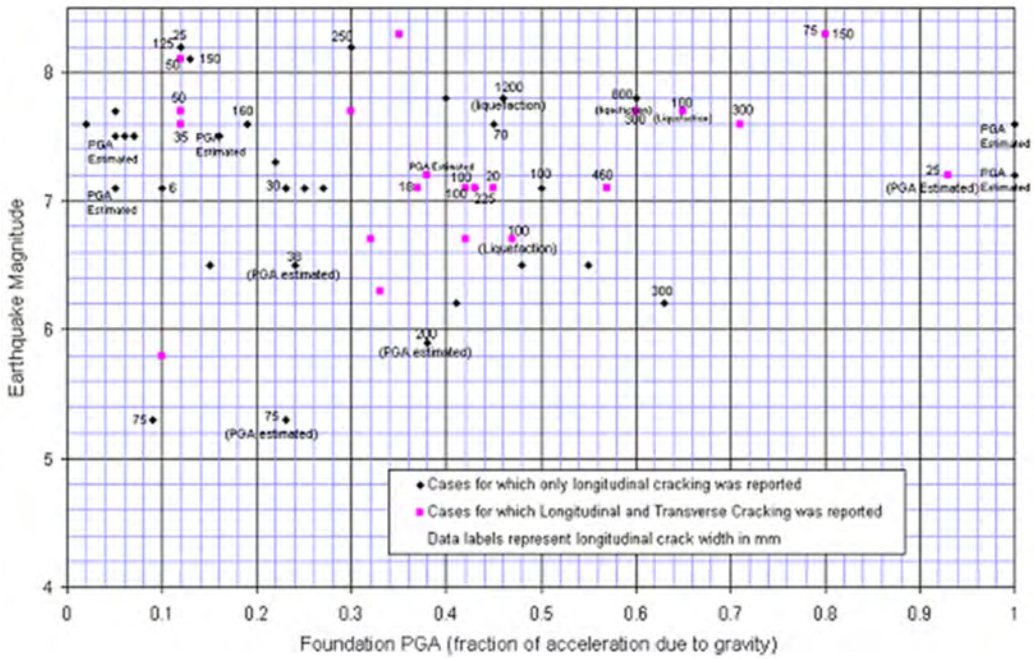


Figure 21: Foundation PGA vs. Earthquake Magnitude (from Pells & Fell, 2002)

Table 40: Influence of factors in the likelihood of cracking or hydraulic fracturing – features giving low stress conditions (from Foster and Fell, 2000)

Factor	Influence on likelihood of cracking or hydraulic fracture		
	More Likely	Neutral	Less Likely
Overall abutment profile	Deep and narrow valley. Abrupt changes in abutment profile, continuous across core. Near vertical abutment slopes	Reasonably uniform slopes and moderate steepness, eg. 0.25H:1V to 0.5H:1V	Uniform abutment profile, or large scale slope modification. Flat abutment slopes (>0.5H:1V)
Small scale irregularities in abutment profile	Steps, benches, depressions in rock foundation, particularly if continuous across width of core (examples: haul road, grouting platforms during construction, river channel)	Irregularities present, but not continuous across width of the core	Careful slope modification or smooth profile
Differential foundation settlement	Deep soil foundation adjacent to rock abutments. Variable depth of foundation soils. Variation in compressibility of foundation soils	Soil foundation, gradual variation in depth	Low compressibility soil foundation. No soil in foundation
Core characteristics	Narrow core, H/W>2, particularly core with vertical sides	Average core width, 2<H/W<1	Wide core H/W<1
	Core material less stiff than shell material	Core and shell materials equivalent stiffness	Core material stiffer than shell material
	Central core		Upstream sloping core
Closure section (during construction)	River diversion through closure section in dam, or new fill placed a long time after original construction		No closure section (river diversion through outlet conduit or tunnel)

Wrights Road Storage Pond embankments are dissimilar to typical embankment dams associated with most case histories because they are not situated in a river valley. Referring to the cracking influence factors in the table above, they do not have constraining abutments or irregularities in the abutment profile. Differential foundation settlement is expected to be minimal. They have an upstream impervious membrane resulting in unsaturated homogeneously stiff embankments; and there are no closure sections. Accordingly, any transverse crack is expected to result from shear, induced by differing amplitude of shaking at corners due to large differences in stiffness of the embankment longitudinally from the stiffness of the embankment sectionally. Further, transverse cracks resulting from shear rather than tension, as is the case in crack estimation which are not expected to form as an open crack, but rather a localised zone of sheared embankment material. Despite this reasoning, the following paragraphs explain transverse crack estimation based on experience with cross valley embankment dams that have been subjected to earthquake shaking. To provide a conservative design, the geomembrane is checked for its ability to span transverse cracks.

Fong and Bennett (1995) report transverse cracks are more prone to occur near embankment dam abutments. They particularly tend to occur where abutments are steeply sloping and stiffer than the embankment (Swaisgood, 1998). Wrights Road Storage Ponds do not have abutments like typical valley dams. Transverse cracking is also prone to occur where differential settlement across the foundation or rigid structure occurs. Negligible differential foundation settlement is expected. However, differential settlement at rigid structure – embankment interfaces is possible. At Wrights Road Storage Ponds this would be at the conduits.

The rectangular shape of the Wrights Road Storage Ponds suggest the sharp (e.g. 90°) corners could be prone to inertial tension within the embankment as earthquake induced settlement and spreading occurs. It is considered tensile separation resulting in transverse cracking could occur at either end of a corner radius. For this assessment transverse cracking by tensile separation is similar to embankment dam case histories for estimating crack geometry. The corners could also be prone to torsional rotation. Torsional rotation of the embankment at corners would likely result in narrower tensile cracks, but will be conservatively treated the same as tensile separation transverse cracks for simplicity.

Rigid structures within or adjacent to the embankment provide conditions prone to transverse cracking. Conduits and concrete spillways at Wrights Road Storage Ponds are such structures. Differential settlement between the rigid structures and the embankment is the transverse cracking mechanism at these locations.

Thus, transverse cracking cannot be ruled out for SEE ground motions at the Wrights Road Storage Ponds in two instances:

1. Embankment corners
2. Rigid structure – embankment interfaces.

Areas of transverse cracking risk at embankment corners of the Wrights Road Storage Ponds are anticipated within the curved section of the corner and over 100 m each side of this curved section. Tensile separation is considered most likely across the embankment at ends of radii. Considering possible torsion of the embankment corners, a distance of ½ the embankment base width beyond the torsional radius was judged safely beyond the zone of possible transverse cracking. The extents of embankment transverse cracking risk at each corner were determined based on this rationale.

Quantitative assessment of transverse cracking at embankment corners follows a sequential process:

1. Embankment crest settlements were estimated based on design ground motions as discussed in Section 7.9.1.
2. Transverse crack widths were estimated according to Fell et al. (2015) empirical method based on normalised crest settlement and “Damage Class” of 4.
3. Transverse crack depths are estimated to develop within the top third of the embankment. This corresponds to depths of 2.67 m for the Medium PIC embankment

and 4.0 m for the High PIC embankment. The estimated crack depths to widths are consistent to ratios of 15 to 20 provided by Pells and Fell (2002).

The depths of water in the transverse crack after the design earthquake (SEE plus aftershock) were determined. First, the post-earthquake freeboard is calculated by subtracting earthquake induced settlement from design freeboard, assuming the pond water level remains unchanged by the earthquake. Secondly, the post-earthquake freeboard is subtracted from the estimated transverse crack depth to estimate the post-earthquake water depth in transverse cracks.

1. The mean submerged crack width is then calculated assuming the crack is triangular from top to bottom.
2. Finally, the post-earthquake discharge through an open transverse crack is calculated, assuming a triangular shape, using the empirical relationship for a rough orifice:

$$Q = 0.6.(2gH)^{0.5}.W.L$$

where H is the mean head over the submerged part of the crack, W is the mean submerged crack width, and L is taken as the submerged height.

Determination of transverse crack geometry is summarised in Table 41. Discharge through transverse cracks is presented in Table 42.

Embankment Resistance to Transverse Cracking at Corners

Discharge through open transverse cracks at embankment corners has been assessed based on estimated crack depths and widths, post-earthquake crest settlements, and design full supply level (FSL). It was conservatively assumed that the FSL does not lower as a result of earthquake ground shaking. The results for Cases 1 and 4 are summarised in Table 40. Cases 2 and 3 are opposite slopes of the same embankment between Ponds 1 and 2. Case 3 results in a deeper and wider transverse crack than Case 2 and water from Pond 1 will flow by gravity to Pond 2. Thus, Case 2 is omitted from transverse crack discharge estimates.

Table 41 indicates that discharges in the order of 300 l/s to 900 l/s could occur through transverse cracks. As previously discussed, the Zone 4 materials are expected to be stable up to seepage rates of 20 to 30 l/s. Therefore, if the geomembrane liner cannot safely deform across transverse cracking without rupturing, the discharge through the transverse crack could erode, downcut and unravel the embankment.

Table 41: Estimated Transverse Crack Discharge

Parameter	Embankment Cross-Section (refer to Table 18 for section location)	
	Case 1	Case 4
Estimated post-SEE & aftershock water depth in crack (m)	1.90	3.39
Mean post SEE & aftershock head (m)	0.95	1.69
Estimated mean submerged crack width (mm)	62	74
Estimated discharge through open transverse crack (l/s)	307 *(Note 1)*	867 *(Note 1)*
Notes: *(1) Discharge calculated with orifice equation: Q = 0.6.[2gH].0.5.W.L; where L is taken as the submerged height.*		

Geomembrane Liner and Earthquake Induced Transverse Cracking at Embankment Corners

The ability of the liner to span the transverse crack is therefore the primary defence against embankment failure due to transverse cracking. The approach outlined in Giroud (Giroud J. , 2012)[3] was adopted to determine the behaviour of the geomembrane liner in the case of development of a crack in the subgrade due to an earthquake. This approach takes into account the tensile stress and strain in the geomembrane from the development of the crack, as well as the deflection of the geomembrane over the crack under the applied water pressure. Giroud (2012) indicates this recent approach supersedes his previous method (Giroud, 1990) to estimate geomembrane behaviour over a crack.

For calculation of the geomembrane behaviour over a crack, the parameters listed in Table 42 for enhanced HDPE geomembrane have been adopted. As outlined in Section 7.6 geomembrane with enhanced tensile and extension properties will be selected for placement in the pond corners. The yield tension and strain for the 1.5 mm thick HDPE geomembrane are listed in Table 42 for the enhanced HDPE geomembrane. On the pond embankments the HDPE geomembrane is to be placed over EcoTech 500 gsm non-woven, needle-punched geotextile. The EcoTech 500 gsm geotextile underneath the geomembrane has not been analysed for behaviour over a transverse crack. The geotextile is not relied on as a defence against transverse cracking.

The friction angle between a smooth HDPE geomembrane and a non-woven, needle-punched geotextile is published in Martin et al (Martin, 1984) as 8 degrees. An interface friction test was carried out, which gave a friction angle of 8.7 degrees. However, a more conservative friction of angle of 12 degrees has been adopted for design.

Table 42: HDPE Enhanced Liner Physical Properties

Parameter	Enhanced Value for Placement at Embankment Corners		Reference
	Machine Direction	Cross-Machine Direction	
Geomembrane type	HDPE, smooth, 1.5 mm thick		Refer Table 16
Yield tension	24 kN/m^2	26 kN/m^2	Refer Table 16
Yield strain	18 %	16 %	Refer Table 16
Friction angle between geomembrane and geotextile	12 degrees	12 degrees	Martin et al, 1984

Figure 22 plots the estimated crack width that 1.5 mm thick enhanced HDPE geomembrane can span with an adopted factor of safety of 2.0, as a function of water depth over the crack. Results

[3] The methodology to analyse geomembrane liner behaviour over a crack in the supporting soil was developed by Giroud (2012) for Genesis Energy Ltd as part of the Tekapo Canal Remediation Project. Rights to use the methodology summarised in Giroud (2012) for the present study has been received by both Genesis Energy Ltd and J.P. Giroud (J. Eldridge & J. P. Giroud, personal communication, Nov 2012). The main aspects of the methodology are summarised in Giroud, et al. (2013).

for geomembrane placed in either the machine or cross-machine direction are plotted in Figure 22. This figure indicates that 1.5 mm thick enhanced HDPE can span a crack of approximately 85 mm at a water depth of 1.7 m over the crack.

The 1.5 mm selected HDPE liner is therefore able to span greater than the estimated transverse crack widths of up to 74 mm at 1.7 m of head, with a factor of safety of 2 as demonstrated in Figure 22. This satisfies embankment stability criteria under earthquake induced transverse cracking.

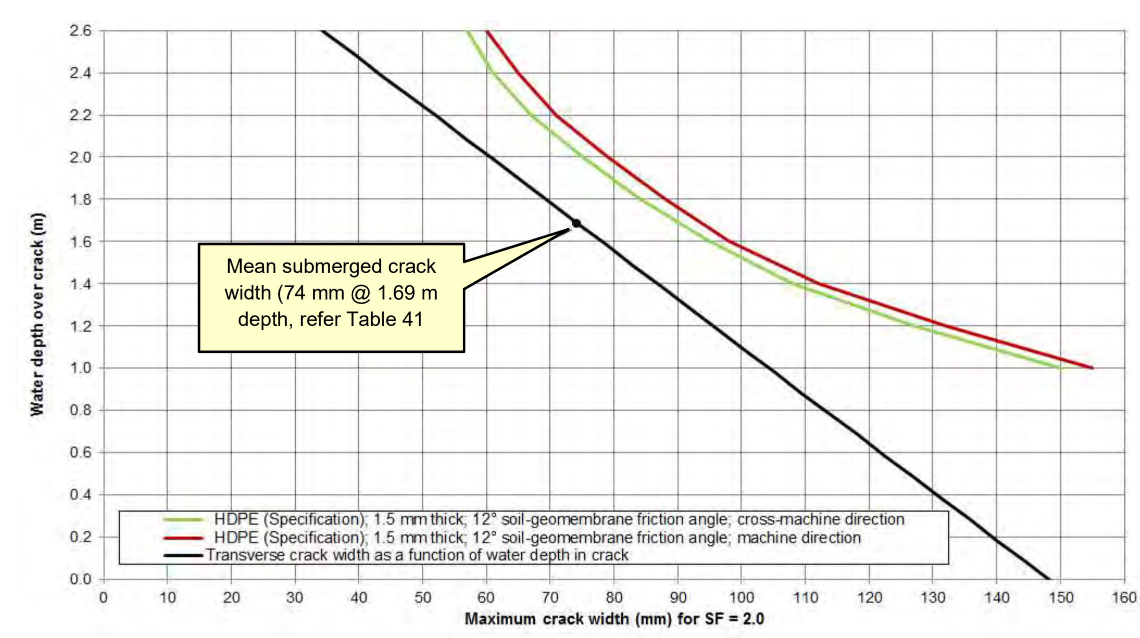


Figure 22: Crack width that can be spanned by 1.5 mm selected HDPE geomembrane at safety factor equal to 2 (SF= 2.0) as a function of water depth

Embankment Resistance to Transverse Cracking at Rigid Structure – Embankment Interfaces

Transverse cracking from differential settlement is common where rigid structures penetrate or abut embankments. Crest settlements estimated for design earthquake ground motions range between 3.04% and 3.41% of embankment height, resulting in total estimated settlements ranging between 243 mm and 409 mm for the two cases reported in Table 33. The conduit height represents approximately 15% to 20% of the embankment height. Thus, differential crest settlement at a conduit location should be no more than 80 mm; (20% x 409 mm).

If the crest transverse crack width was one-half the differential settlement amount, a conservative judgment, the maximum transverse crack width at rigid structure – embankment interfaces should not exceed 40 mm. It is assumed such a crack would remain open over the upper third of the embankment, that is extend 4 m below the crest in a triangular shape. Then at a depth of 2 m the crack width is 20 mm. Analysis of the 1.5 mm HPDE geomembrane liner indicates it will not rupture at depths of 2 m when subjected to a 20 mm crack (see Figure 22).

Therefore, the 1.5 mm HDPE material should not rupture where subjected to an earthquake induced, transverse crack at rigid structure – embankment interfaces. An under layer of heavy geotextile under the HDPE geomembrane has been included around structures which penetrate the geomembrane as a further mitigation measure.

A further precaution is added by extending compacted AP20 material screened from Zone 4 material which will have higher fines content and lower permeability than the surrounding Zone 4 material forming the embankment and foundation. This AP20 material is compatible with the surrounding Zone 4 material. Should the liner or concrete transition to culvert pipe junction be damaged, due to earthquake shaking, leakage will be restricted by the AP20 material adjacent to and supporting the concrete transition and the liner and filter fabric layer.

7.9.4 Summary of Analysis

In summary, assessment of earthquake induced embankment settlement and cracking indicates the embankment design without mitigating measures meets the design criteria in most areas. Transverse cracking at embankment corners and rigid structure – embankment interfaces require limited mitigating measures. Specifically:

- Crest deformation following the SEE and aftershock in the order of 243 to 409 mm are estimated, giving normalised crest settlements of the order of 3.04% and 3.41% of the embankment height. Such settlement magnitudes can be accommodated by the crest freeboard of 1.2 m and indicate suitable embankment stability under SEE loadings.

- Longitudinal cracks resulting in a separation width in the damaged geomembrane in the order of 304 to 341 mm wide are expected to occur under SEE ground motions. These cracks are expected to occur in the upper third of the embankment, and the worst case is assumed where the geomembrane liner also separates at the same length as the longitudinal crack. The seepage through a longitudinal crack was estimated using empirical equations of Giroud & Bonaparte (1989) and Giroud, et al (1989), as well as with a SEEP/W model. Seepage rates at the outer embankment toe in the order of 10 l/s/m are estimated and such a seepage rate is very unlikely to lead to instability in the embankment material.

- Transverse cracks could occur at embankment corners and rigid structure – embankment interfaces under SEE ground motions. Such cracks are especially hazardous to water-retaining embankments because they present an open pathway across the embankment that can potentially erode and downcut, causing breach.

- 1.5 mm thick HDPE geomembrane liner with “enhanced” properties is estimated to be able to span greater than the estimated transverse crack widths of up to 74 mm at 1.7 m of head, with a factor of safety of 2.

- The 1.5 mm HDPE geomembrane liner should not rupture where subjected to earthquake induced, transverse cracking at rigid structure – embankment interfaces. An underlayer of heavy geotextile is included around penetrations which will form a barrier should the liner be damaged at rigid structure – embankment interfaces.

- The above summary reflects the conservative approach adopted to address potential transverse cracks in the embankments resulting from extreme earthquake shaking on account of:
    - Estimated settlements are based on experience with zoned embankments with partially saturated low permeability core materials.
    - The drained gravel embankments forming the Wrights Road Storage Ponds will experience less settlement due to the more favourable drained granular embankment material.
    - Less settlement will result in lower potential for transverse cracking with smaller cracks.

7.10 Seepage Control for Pipe Penetrations

Placement of pipes through earth dam embankments are a common cause of piping failure and accidents. To prevent piping failure along pipe penetrations the following details have been included in the design:

- Haunching of all pipe penetrations with controlled low strength material (CLSM) (refer to Drawing WIL1125/30/135 for a typical example)
- Provision of a suitable drainage material around pipe penetrations which is compatible with underlying foundation gravels and facilitates vertical drainage, and
- Extending this suitable drainage material under and around pipe to transition structure connections to limit seepage should these connections be damaged by earthquake shaking.

7.10.1 Pipe Haunching

Pipe penetrations are supported on a low-strength concrete base (CLSM), which ensures that the pipes are well bedded and less susceptible to cracking from differential settlement. The CLSM is used as the bedding material, in accordance with AS/NZS 3725:2007, Appendix A.

It is noted that FEMA (FEMA, Technical Manual: Conduits through Embankment Dams. US Federal Emergency Management Agency., 2005) states that CLSM is not recommended for backfilling around conduits in high hazard dams due to its poor bond with the conduit and adjacent foundation and potential for interior cracks (shrinkage). However, it is considered that CLSM is appropriate as a bedding material for the Wrights Road Storage Ponds for the following reasons:

- FEMA (2005) is primarily addressing zoned, low permeability core embankments which are markedly different to the homogeneous, free draining and geomembrane lined embankments adopted for the Wrights Road Storage ponds.
- CLSM will have similar deformation characteristics as the adjacent foundation materials and will reduce the tendency for high stress areas on pipe penetrations resulting from earthquake shaking
- Each layer of CLSM shall be vibrated by means of an electric or pneumatic vibrator to ensure good contact with the pipe and surrounding foundation material.

7.10.2 Pipe Backfill

Pipe backfill consists of AP20 (Type 1) regraded foundation material. Filter compatibility between the AP20 backfill material and the underlying foundation material was assessed using the FEMA (FEMA, 2011) methodology and indicate that the AP20 backfill material meets particle retention (internal stability) and permeability requirements. The AP20 (Type 1) material is therefore suitable for seepage control at the pipe penetrations.

7.11 Instrumentation

7.11.1 Embankment Piezometers

Fifteen standpipe piezometers have been included extending into the gravels underlying the ponds to enable monitoring the groundwater level under the lining of the ponds. It is recommended that the standpipe piezometers are drilled into the embankments prior to commissioning.

Steel casing may be required during drilling and standpipe installation operations. The temporary steel casing will be withdrawn during piezometer installation. The piezometer standpipe will be 25 mm ID ABS tubing with external glued couplers. Port levels of the piezometer are to be below the base of the embankments. Each piezometer tip will consist of 40, 4 mm diameter holes evenly spaced over the bottom two metre section of the piezometer standpipe. The piezometers will be installed with ports surrounded with sand wrapped in geotextile.

7.11.2 Time Domain Reflectrometry (TDR)

A Time Domain Reflectrometry (TDR) cable will be installed along the entire crest length of all the embankments. It will be connected to the control system to enable automatic detection of embankment deformation. It will trigger a low-level alarm alert if the cable indicates a small movement and a high-level alarm alert if the cable has been sheared, which may be indicative of possible large embankment deformations.

When an alarm is triggered, a download of data will be carried out and the position of the strain or complete shear will be calculated.

It is expected during the operational phase that surveillance and maintenance staff will undertake an emergency inspection in the event of a TDR alarm.

7.11.3 Surface Survey Markers

Fifteen surface survey monuments will be installed along the crest, as shown on drawing WIL1125/30/21. The markers will be used to monitor embankment deformation and will complement the TDR system in monitoring the long-term dam safety of the embankments.

It is expected that the deformation markers will surveyed annually and/or after a strong earthquake (≥ MM6).

7.11.4 Access

Vehicular access to the pond embankment crest is provided at the south east end of the tub for maintenance and surveillance.

No access for vehicles into the ponds is provided as the geomembrane is not able to support loading by even light vehicles. Should vehicular access into the ponds become necessary in the future, the geomembrane and filter fabric underlay will be cut and pulled back so that vehicular access is over the underlying gravels at the location most suitable for the activity being undertaken. On completion the embankment surface, underlay and geomembrane will be reinstated.

8.0 Pump Stations and Pumps

The two proposed pump stations are the:

1. Buffer Pond Pump Station which is required to fill Pond 1; and
2. Pond 2 Pump Station which is required to lift water from the invert of Pond 2 into the main race MR4 via the Tub and G6.

Operation of both pump stations will be managed by the main control system. All the pumps are operated according to the control system instructions, which are based on the feedback received from level sensors in the Buffer Pond, Ponds 1 and 2 and the Tub, and either the required irrigation demand or availability of water for storage.

8.1 Buffer Pond Pump Station

Pond 1 is initially filled from the Buffer Pond by gravity until the water levels approach RL 223.00.

Depending on the availability of water in the Buffer Pond; Pond 1 is further filled by the Buffer Pond Pump Station. The three Buffer Pond pumps will be vertical shaft turbine pumps with 335 l/s capacity for each pump and are installed in partitioned wet wells, but hydraulically connected by bottom openings.

The pumps are fully automated and operation is managed by the main control system. Buffer Pond pumps are set to start at different water levels in Buffer Pond in the following sequence:

	Start	Stop
• Pump 1 will start when the Buffer Pond reaches RL	222.10	222.00;
• Pump 2 will start when the Buffer Pond reaches RL	222.30	222.20;
• Pump 3 will start when the Buffer Pond reaches RL	222.50	222.40.

Depending on availability of water in the Buffer Pond, the three pumps will pump water into Pond 1 up to its full supply level RL 226.50 m. The pumps are designed to operate continuously, and it will take 12-13 days to fill Pond 1 to the full supply level.

Staging the pumps start up sequence (as set out above) means that small amounts of excess water can be captured without excessive start/stop scenarios.

The choice of three pumps allows for one pump to be out of commission without a major impact on the filling operation.

The pumps and motors will be installed in the wet wells. A control structure will be provided on the embankment crest to house the electrical and control panels. Access with parking to the pumps and control houses will be provided.

8.1.1 Pump Selection

The pumps will operate within a narrow operating range resulting in a small variation in head and flow for each pump as detailed above. Three vertical shaft Macquarrie MC350 Axial Pumps

(+2º impeller) coupled to CMG 4 Pole 30 kW electrical motors have been selected to meet the required duty.

The operating range between the 3 pumps is 5.4 m – 5.6 m (static lift and friction losses) for a flow range of 325 l/s to 335 l/s (see Figure 23).

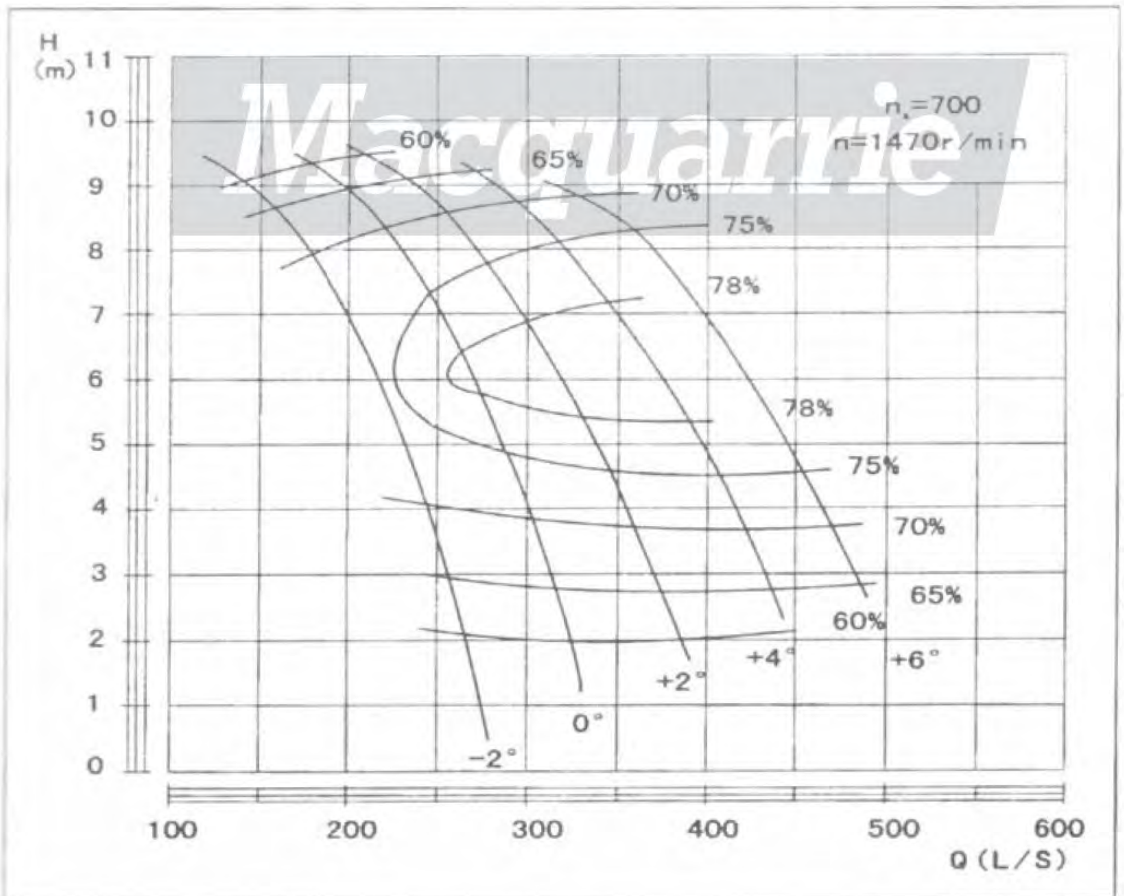


Figure 23: Macquarrie MC 350 axial pump performance curve

To limit the risk of cavitation occurring, the minimum water level in the wet well will be controlled 800 mm above the pump suction inlets. The maximum particle size the pumps can pass is 32 mm which determines the screen sizing on the intake. The intake and screen will be visually inspected from an observation platform and stairs leading down to the inlet to allow the screen to be raked manually.

8.1.2 Pipeline Design

A structural assessment of all pipelines was undertaken using NZS2566.1 Buried flexible pipeline (Part 1: Structural Design) and 1 mm of corrosion allowance was included in the assessment for steel pipelines. The structural assessment of the DN1050 RCRRJ Class 3 inlet pipeline was undertaken using the CPAA Pipe Class V2.0 software.

The friction head loss in the pipeline was calculated using the Darcy Weisbach equation with k=0.1 for steel pipes. Water hammer and fatigue are not considered a major issue due to:

1. Operating head of the pumps is low,
2. Length of pipeline (approximately 15 m) is short, and
3. An air valve to reduce transient pressures is included.

The discharge pipeline is DN450 SW steel pipe (OD = 457.2 mm & WT = 4.8 mm).

8.2 Pond 2 Pump Station

Two inclined pumps installed on the western bank of Pond 2, approximately 23 m from the Tub will lift water from below RL 220.84 in Pond 2 into the Tub. The inclined pumps are to have an individual maximum capacity of 1,280 l/s at the normal maximum pumping level of RL 220.84m. At minimum pump operating level with a static head of 7.2m the required minimum discharge of each pump is 1,000 l/s.

When the water demand from storage to MR4 is high (demand > 2x pumps capacity), both pumps will operate. When the demand from storage is moderate (between the capacity of one and two pumps), only one pump will operate. When the demand from storage is low (demand < 1x pump capacity), both pumps will be shut off and Pond 1 will release water into the Tub through G5.

Operation of the inclined pumps is managed by the control system, determined by demand required from storage to MR4 and also the water level inside the Tub. When the water level in the Tub exceeds RL 222.8 m the pumps are stopped (to avoid over filling the Tub).

The proposed pump operation is as described in Section 2.3 above. The logic for control is described below and shown in Figure 25 in Section 10.0 below.

8.2.1 Pump Selection

The pumps deliver to a fixed outlet above the maximum water level in the Tub. The static lift increases as the water level in Pond 2 decreases. Two inclined pumps have been adopted based on recommendations by a pump supplier to meet the required duty. For purposes of meeting pond design and operational requirements the model and make of pump assumed is the Ornell Flood-lifter FE575-750 single stage type, each coupled to a 132 kW 8 pole inverter motor (available from Fluid Engineering Pty, Griffith, Australia). The Specifications set out performance, functional, maintenance and service life requirements based on the Ornell Flood-lifter series, or an equivalent (or better) alternative.

The pumps operate between 4.5 m and 9.1 m head (static lift and friction losses) for a flow range of 1,000 l/s to 1,280 l/s (see Figure 24). They are required to be designed to operate at a static head of between 0 m and 7.2 m. There is no connecting pipework between the pumps or pipework as each pump will operate independently.

The pumps are specified to be capable of pumping to a minimum level of RL 215.8 m in Pond 2, without vortex entrainment or cavitation risk. Submergence characteristics adopted for reference design purposes have assumed a minimum of 1m of water above the pump intake.

This is to be verified and finalised by the pump supplier depending on the specific characteristics of the selected pump.

A strainer/screen is to be fitted at the suction of each pump for pump protection. The arrangement of screen and sizing of openings is to be specified by the pump manufacturer to suit the particular type of pump impellor/propellor. The screen will be sized to prevent risk of cavitation with half of the screen area blocked.

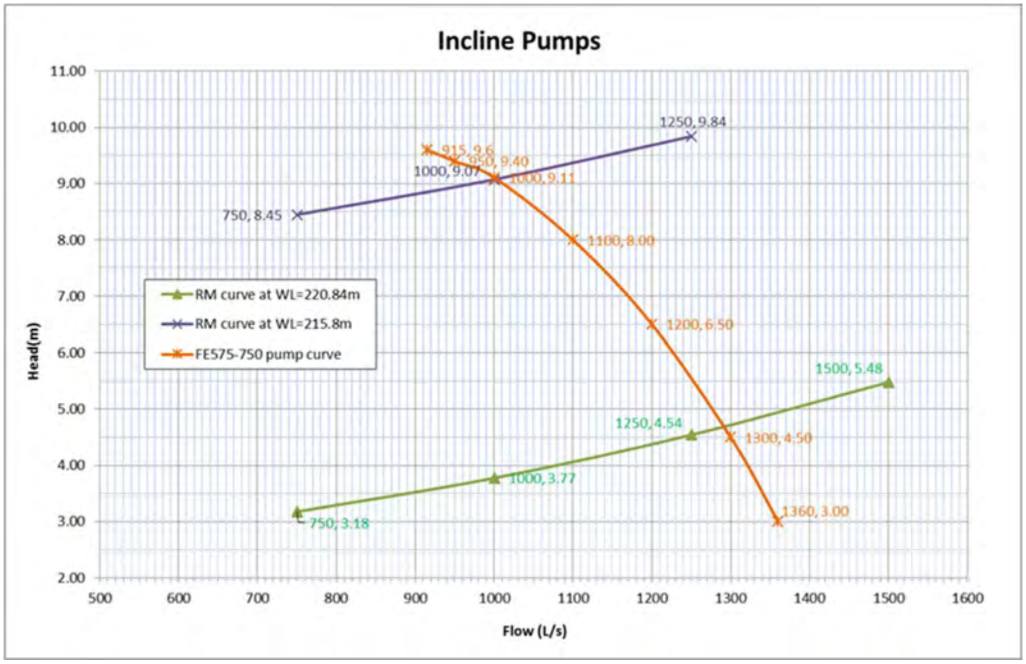


Figure 24: Performance curve for Pond 2 Pump Station

A concrete foundation will be constructed down Pond 2 embankment as shown in Drawing WIL1125/30/135. The reference design shows the inclined pump column placed in a concrete channel. Details of the pump column fixing to the foundation will be developed in collaboration with the selected pump supplier to suit the specific pump structural, operational and maintenance requirements. The connections will need to allow for thermal expansion/contraction in the order of 10mm (approx.) as well as having sufficient capacity to resist hydrodynamic forces during starting/stopping, and sufficient seismic capacity. The pumps do not have a dam safety critical function and will be designed for an OBE level event.

Major maintenance of the inclined pumps is expected to require lowering of the water level in Pond 2 so the pump column can be accessed and disconnected from the foundation for removal to the embankment crest. The main pump column will be required to be lifted out in sections to avoid the need for craneage during removal.

A concrete foundation will be constructed down Pond 2 embankment as shown in Drawing WIL1125/30/135. The pump and motor will be placed in the concrete channel and two short lengths of 150x150x10 equal angles will be bolted to the pump column flanges to locate the pump column.

Maintenance of the inclined pumps will require lowering of the water level in Pond 2 so the equal angles on the column flanges can be removed. The main pump column is then disconnected from the pump motor and a crane used to lift the entire pump column onto the top of the pond embankment. Access for a crane is provided from Dixon Road to the Pond 2 crest at the Pond 2 pump station site.

Pipeline Design

The outlet pipe from the Pond 2 pumps to the Tub is a DN750 steel pipe with a wall thickness of 8 mm. A structural assessment was undertaken using NZS2566.1 Buried flexible pipeline (Part 1: Structural Design) and 1 mm of deterioration was factored into the assessment for steel pipelines. This will be reviewed and verified as appropriate on receipt of design information (including transient pressures) specified to be provided by the Pump Supplier.

8.3 Pump Control and Protection

A control building will be provided on the embankment crest adjacent to each pump station to house the electrical and control panels. The required dimensions of the control buildings will be as recommended by the selected pump suppliers.

The electrical connections/controls for the pumps and motors will be designed as part of the "Design & Build" package in accordance with NZ Electricity Regulations and Codes of Practice. The electric motors will be controlled by a Soft Starter that will reduce the stresses on the motor and cables. The Soft Starters also provide additional electronic protection for a range of faults including:

- Loss of phase,
- Current overload protection,
- Phase imbalance,
- Temperature overheating.

Both the Buffer Pond and Pond 2 pump stations will each have a Programmable Logical Controller (PLC) installed and programmed to protect the pumps and motors. The PLC will 'communicate' with the control system designed by NIWA. The ability of the two systems to communicate is required to ensure the safe integration of the pumps into the overall control system and prevent overtopping the ponds.

A detailed Operation and Maintenance Manual (O&M) will be prepared by the Pump Supplier for each of the pump stations. The O&M manual will detail maintenance requirements along with a description of the control logic and safety features for each pump station.

8.4 Specifications for Pumps

Specifications for pumps are provided in the Technical Specification – Appendix F

8.5 Design & Construction Standards

The following Standards will be utilised during the design, manufacture of the pump station appurtenances and construction of both pump stations:

- AS/NZS 1170.5 Structural Design Actions Part 5: Earthquake actions – New Zealand
- AS/NZS 1594 Hot-rolled flat steel products
- AS/NZS 2566.1 Buried flexible pipelines Part 1: Structural Design
- AS/NZS 2566.2 Buried flexible pipelines Part 2: Installation
- NZS 3109 Concrete Construction
- NZS 3144 Specification for Concrete Finishes
- AS/NZS 4158 Thermal-bonded polymeric coatings on valves and fittings for water industry purposes
- NZS 4219 Seismic Performance of Engineering Systems in Buildings
- NZS 4442 Welded steel pipes and fitting s for water, sewage and medium pressure gas
- AS/NZS 4680 Hot-dipped galvanized (zinc) coating on fabricated ferrous articles

9.0 Control Gates

9.1 Operational Description

A total of seven flow control gates are required for the Wright's Road Pond Storage Scheme. Their locations are shown schematically on Figure 4 in Section 2.5 above

Gated inlet culvert G1 is a single barrel 1,800 mm diameter with concrete inlet and outlet transitions as shown on Drawing WIL1125/30/101. Gate G1 is a 1,800mm fully automated gate, managed by the control system to maintain the water level in the Buffer Pond. Once the Buffer Pond level approaches the Buffer Pond service spillway level Gate G1 is closed so that the Buffer Pond Pump Station can continue to fill Pond 1.

The two inter-pond gated culverts are G4, discharging water from Pond 2 to the Tub, and G5 discharging water from Pond 1 to the Tub. Both are single barrel gated culverts, each fitted with a 1,800mm control gate as shown on Drawings WIL1125/30/113 to 116. Gate G4 is located at the downstream end of its culvert in the Tub, and Gate G5 is located at the upstream end of its culvert in Pond 1.

The three pond outlet gated culverts, G2, G3 and G6, release irrigation discharges to the various races. Culvert G2 releases water from Pond 2 to Race R2 and is over-sized at 1,930mm diameter to provide for a possible future hydropower station, as shown on drawing WIL1125/30/103. Culvert G3 releases water from Pond 2 to Race R3 and comprises a single 1,600mm diameter culvert as shown on drawing WIL1125/30/110. Culvert G6 has twin 1,600mm diameter barrels discharging from the Tub to Race MR4 as shown on drawing WIL1125/30/117. Control gates to suit each culvert size are mounted at the upstream end of each culvert, with Culvert G6 having two gates (G6a and G6b) – one for each barrel.

9.2 Gate Design

The control gates are rectangular steel slide units operated with hydraulic rams and cylinders mounted on the gate frame.

Final detailed design of the control gates will be by a designated gate supply subcontractor under the Design and Construct Contract. A performance specification is included in Appendix F (Technical Specifications) and a reference design has been prepared as shown on Drawings WIL1125/30/201 to 206. The reference design drawings are to be considered as indicative to show the intended arrangement and form of the control gates.

The gate is formed with a steel skinplate stiffened by horizontal beams which are fixed to the skin plate with bolts in blind tapped holes. Water load on the skinplate is supported by the vertical edges bearing on UHMWPE slider blocks which extend over the full travel of the gate. A horizontal UHMWPE strip is used as a bearer and top seal for the closed gate. The bearers are bolted to a fabricated steel column frame with an overhead beam to support the lifting cylinder and ram connected to the gate. The frame is anchored to the intake concrete headwall with stainless steel sleeve anchors. The gate lifting hydraulic rams will operate submerged when the ponds have significant water in them.

The Hydraulic Power Units (HPU) for driving the hydraulic rams for opening and closing the gates will be housed in a services control building mounted on the pond embankment crest above each gate. Permanent access will be provided between the embankment crest and the gate for inspection and maintenance.

Gate operation is managed by the control system.

9.3 Design, Manufacture, and Installation Standards

Gate design will be carried out under the following (or approved equivalent) standards:

NZSOLD	NZSOLD (2015) Dam Safety Guidelines
DIN 19704	German Standard for Hydraulic Steel Structures, Parts 1, 2 and 3
EM 1110-2-2105, USACE	USACE Design of Hydraulic Steel Structures
AS 1111.1 and .2	ISO metric hexagon bolts and screws – Product grade C – (1) Bolts and (2) Screws
AS/NZS 1554.1	Structural Steel Welding Standard, Part 1: Welding of steel structures
AS/NAS 1627	Code of Practice for preparation and pre-treatment of Steel surfaces (Parts 1, 2 and 4)
AS/NZS 3678	Structural steel – Hot-rolled plates, floor-plates and slabs
AS/NZS 3679.1	Part 1: Hot-rolled bars and sections
NZS 3404	Steel Structures Standard

The control gates have a dam safety critical function for emergency dewatering purposes, as fully detailed in Section 3.0. Gate design is specified to comply with the NZDSG to satisfy the relevant dam safety critical function from Table 6 in Section 3.0 under all relevant load conditions including normal (static), flood and seismic. The design levels for each load condition will all be assumed to correspond to a High PIC classification, as follows:

- Flood condition: PMF event
- Seismic condition: Safety Evaluation Earthquake (SEE) = 1:10,000 AEP event

Those control gates that have an emergency dewatering function will be designed to be capable of opening following a SEE seismic event and any associated aftershocks.

The HPUs will be designed with fittings to allow connection of a backup portable hydraulic motor/pump.

9.4 Gate Flow Control

Each control gate is required to fully open or fully close with the reservoir at maximum operating level, and will be capable of being held partially open in any position for any length of time.

9.5 Gate Sealing

The gates are not required to seal drop-tight. Allowable leakage of any gate under its normal maximum pond water level is specified to be a maximum rate of 30 l/minute.

The reference design shows no elastomeric seals on the control gates, and sealing will be achieved by contact between the gate leaf and the UHMWPE slider block under water pressure against a closed gate. The performance specification requires the gate supplier to take suitable measures in design, manufacturing, and installation to ensure compliance with the maximum leakage requirements, including the following:

- Deflection of the gate limited to 1/600th of the gate horizontal span
- Machining of the gate lower edge and potentially the lower frame section
- Provide a means of accurately aligning the gate frame during installation

9.6 Gate Lifting/Lowering Forces

The control gates do not require to gravity close for any dam safety or operational reasons, and will not in fact be capable of gravity closure in normal operation.

In calculating gate lifting and lowering capacities and requirements for hoist design, the gate supplier is required to make the following allowances as a minimum for detailed design:

1. Hydrostatic pressure to be computed assuming the maximum upstream water level with no downstream backpressure, and with the gate in any position.
2. Friction factors for materials in sliding contact to be taken at the upper end of the range quoted by manufacturers and to allow for any long-term degradation that may occur over the design service life.
3. Hydraulic downpull forces may be calculated by any recognised means, or alternatively may be taken as a minimum of 20% of the total vertical lifting load (exclusive of safety factors).
4. Hydraulic downpull to be ignored for calculation of lowering forces.
5. Factor of safety of 1.5 to be applied to calculated vertical friction forces and hydraulic downpull forces.
6. Minimum factor of safety of 1.3 to be applied as a mechanical design margin

Seismic loads may be ignored for purposes of calculating lifting/lowering forces.

9.7 Hydraulic Hoist System

The control gate actuator hydraulic rams are mounted on top of the gate frames and will thus be variously submerged or exposed to weather depending on pond water levels. The hydraulic rams are therefore to be marine grade capable of operating submerged for extended periods of time, and shall comply with the 50-year design service life requirement for components that contain hydraulic oil or are submerged.

The HPUs will have electrically driven hydraulic pumps operating with single phase 220 volt power supply. The power packs will operate with a maximum 2000 psi with down pressure to 1500 psi. The HPUs will be mounted in weatherproof concrete service control buildings located on the embankments above flood water level.

Two types of installation are envisaged:

- Multiple gates and associated individual hydraulic rams in close proximity which may be operated with a common power pack (at outlet control G6); and
- Single gates with individual hydraulic ram operated with a single power pack (located at the inlet and each of the inter pond connections and outlet structures).

Hydraulic oil reservoirs will have a working capacity of at least 110% of the oil required for the gate cylinders. The oil reservoirs will be fully bunded to contain the total quantity of oil in the hydraulic system with at least 50mm of freeboard.

The HPU will be sized so as to produce a gate movement rate of not less than 900mm/minute for lifting or lowering.

9.8 Corrosion Protection Measures

All mild steel items will be painted for corrosion protection after fabrication. Gate and gate frame steelwork will be protected with a high-build epoxy paint system of 400 micron dry film thickness, such as “Interzone 954” by International Paints product or an equivalent approved product.

All bolts and nuts for securing the panels to the embedded frame will be stainless steel. Zinc-sprayed or galvanized fastenings will not generally be used for gate/frame components unless compliance with the design life requirements in the performance specifications can be demonstrated.

Galvanic corrosion between dissimilar metals will be prevented by the use of suitable electrolytic isolating components or materials.

10.0 Control System

The automation proposed will operate in tandem with the existing WIL control system (Ref Appendix G) and effectively form an extension of it. It will control inflow to the storage ponds and drawdown of the storage ponds by supply of irrigation water to the main race MR4, distribution races R3 and R2, including future hydro-power supply to generate electricity. Safety features are incorporated into the automation to prevent overfilling of the ponds. The control system will limit the rate of initial opening of the gated culverts if the ponds are empty in order to prevent rapid culvert outflow from eroding the lining ballast, particularly in the vicinity of the culvert outlet where flow is concentrated.

Alarms to indicate problems that need to be resolved quickly such as sensor failure or over-fill pond level will signal to mobile phones carried by the Operations and Surveillance staff. The basic operation logic for the control system is shown in Figure 25. Further detail of the control system scoping is included in Appendix G.

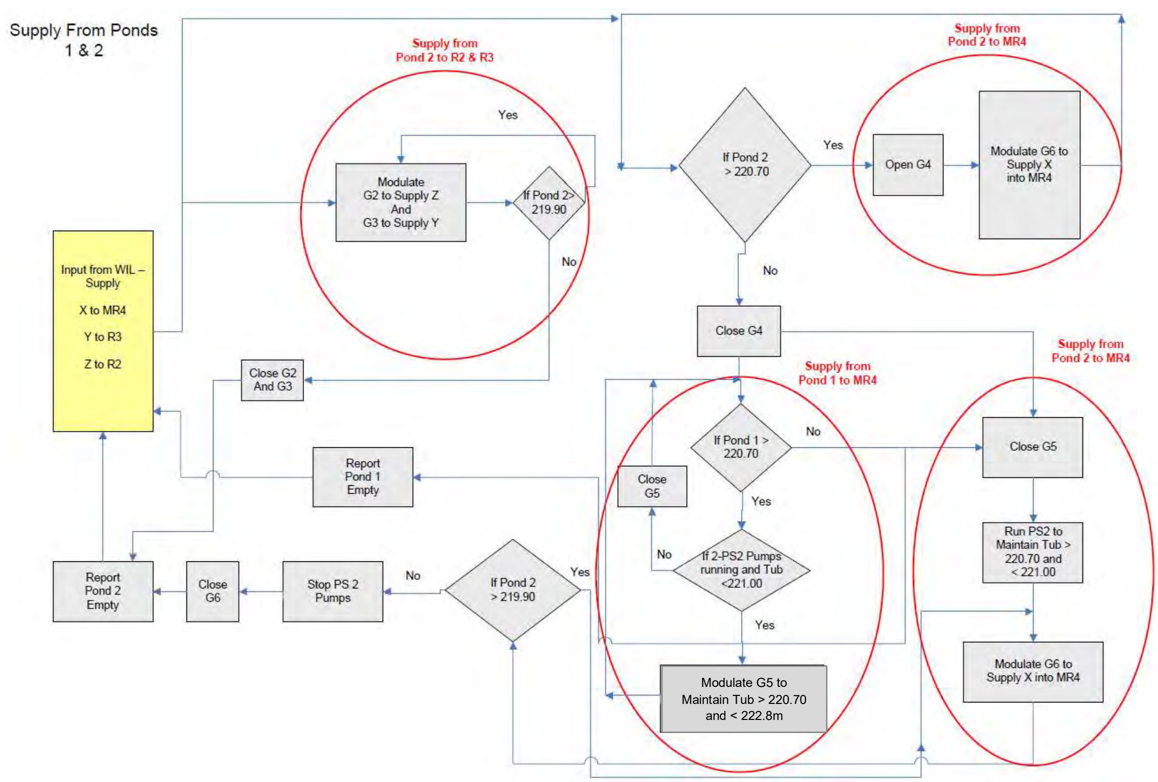


Figure 25: Wrights Road Storage Ponds Operation Control Logic

Superimposed on the above basic logic are:

- Limitation on the rate of inflow increase when ponds are near empty;

- Limitation on the rate of outflow increase when the canal downstream is empty or near empty;
- Initial controlled filling of Pond 1 by gravity until the water level reaches RL 222.60 m and maintaining the Buffer Pond water level at RL 222.60 m;
- Initiating the Buffer Pond pump station and terminating pumping into Pond 1 when its water level reaches RL 226.50 m;
- Maintaining the pre-defined optimal water level in the Tub;
- Open, control or closure of all the outlets in tandem with WIL's control system instructions on water requirements and availability in MR4, R3 and R2.

Alarms will be programmed into the control system to transmit an alarm on:

- Over filling of ponds;
- Gates do not move, when they are commanded to do so;
- Pumps are detected to be running when the lowest level switches are no longer submerged;
- WIL control system instruction to fill, when ponds are already fill;
- WIL control system instruction to supply, when ponds are empty;
- Power failure; and
- Signal failure.

10.1 Control System Design and Build

Conceptual design of the Control System is presented in Appendix G.

In tandem with pond embankment construction, detailed design of the Control System will be completed by NIWA as part of the Control System Design Build Subcontract.

11.0 Safety in Design

Safety in Design is the process of managing the health and safety risks throughout the lifecycle of the dam.

In accordance with the Health and Safety at Work Act 2015 (HSWA), a Safety in Design workshop was held during the detailed design stage, on 13 August 2021. Representatives from the Contractor (REL), the Designer (Damwatch) and the Developer (WIL) participated in the workshop. Hazards associated with the different stages of the project lifecycle were considered during workshop (from the Design to Decommissioning stage). As a group, the likelihoods, consequences and levels of risk were assessed and assigned to each of the hazards identified. Mitigation measures were then proposed to control each of the hazards identified. All measures proposed were approved by all parties to implement in each of their respective stages.

The developed register has been included as Appendix M and should be referred to and further developed as necessary, as the project evolves through its life cycle.

The following key items were added to the design, as a result of the Safety in Design workshop:

- Trash Boom: It was identified that the Pond 2 liner was vulnerable to debris damage. Debris could enter Pond 2 via the service spillway, which could tear the liner, and seepage through the liner could then erode and fail the embankment. This hazard was mitigated by designing a trash boom across the main canal, upstream of the service spillway, which would prevent floating debris from entering Pond 2.
- Pond Safety Ladders: It was identified that there was no means of exiting the ponds if someone was to fall in. Members of the public will be restricted from accessing the site and pond safety ladders have now been included around the perimeter of the ponds at 200m intervals. Pond ladders have also been provided adjacent to all concrete structures for maintenance access. Widening of the crests to align with the NZDSG 2015, also reduces the risk of personnel entering the pond.
- Handrailing – It was identified that there was a risk of potential injury from falling from height at and around concrete structures. Handrailing around all concrete structures accessible to the public and at high use areas within the perimeter fence have now been included.
- Crest widening: The crest was widened in line with the NZDSG to provide safe access along the crest during construction and operation.
- Permanent boundary fencing – on account of the vulnerability of the geomembrane to vandalism, access to the site will be strictly controlled by the provision of boundary fencing.

12.0 Quality Control

Safety of the ponds depends on both robust design and quality construction followed by an appropriate dam safety program to maintain the ponds in a safe condition.

This report outlines the design that has been adopted. An accompanying vital component of dam safety relative to the ponds is a comprehensive construction quality assurance programme.

The specification which forms Appendix F hereto specifies the quality control program, which the Contractor shall set up and operate. The quality control program includes material testing and sampling; compaction tests, soil aggregate gradings and geomembrane testing to establish correct physical properties and maintenance of these during construction. It further includes construction trials needed to establish that construction methods used produce construction consistent with the design described herein.

In accordance with NZSOLD (2015) for High PIC dams, the Designer shall be represented full time on site by an adequately qualified person. Specific preconstruction and construction control stages which will require the Designers attention are:

- Review of quality assurance method statements;
- All stages of fill placing trials of Zone 4 material;
- Foundation preparation and inspection for all earthworks and concrete works;
- Embankment construction;
- Placement, anchoring and jointing of the geomembrane and geotextile;
- Construction of crump weirs, service spillway and low level conduits (foundation preparation and inspection, reinforcement fixing, concrete placement and backfilling);
- Construction of the emergency spillways and fuseplug spillway;
- Installation of gates;
- Installation of automation control systems and water level stilling wells;
- Installation of instrumentation including piezometers, TDR cable and deformation survey markers; and
- Commissioning of the scheme.

All construction control stages will have hold points to ensure the intent of the design has been met. The Designer’s representative on site shall have authority to order additional work necessary for safety. It is proposed that the Designer’s representative and REL’s Site Engineer will sign off critical construction phases. If required, a REL engineer can be on site within two hours.

In summary, a Designer’s representative will be on site full time. Damwatch specialist staff will be available to advise on specific matters throughout the project and will be consulted during key milestone phases.

12.1 Design Quality Assurance

Damwatch are responsible for project design and review functions.

Damwatch is ISO 9000 certified which sets the quality assurance standards adopted for internal review. The “Design Review/Technical Review” as detailed in IPENZ Producer Statement 1 (PS1) has been completed within Damwatch as part of the Damwatch quality assurance in accordance with ISO 9000.

A three-man peer review panel of international dam engineering experts, Tony Pickford of Pickford Consulting Ltd., Trevor Matuschka of Engineering Geology Ltd., and Mike Sadlier of Geosynthetic Consultants Australia Pty Ltd., have undertaken an independent peer review. The scope of this Peer Review is consistent with IPENZ Producer Statement 2 (PS2) for “Review of Completed Work”.

12.2 Geomembrane Quality Assurance

Quality assurance of the geomembrane will be addressed as follows:

- During production in the factory; and
- During installation in the field.

Installation in the field will have the following quality control components:

- Inspection by designer of Zone 4 surface preparation prior to geomembrane placement;
- Full time observation by designer during placement, anchoring and jointing of the geomembrane;
- Removal of joint sample for laboratory testing; and
- Sampling at rate of one sample per 300 m of joint length.

The joint sample shall be divided into individual test specimens, five for shear and five for peel, tested in the laboratory to failure.

13.0 Construction Sequence

To limit the impact of extreme weather conditions during construction, it is recommended to start construction of the buffer pond emergency spillway and fuse plug spillway, before proceeding with either Pond 1 or Pond 2. Initially, the topsoil and silt layer immediately below the ground surface will be removed from the embankment footprint where Zone 4 bulk fill will be placed. This will be continued in the level areas of the pond invert. The topsoil will be stockpiled separately for subsequent placement as landscaping on the external embankment slopes. The silt will also be stockpiled separately for placement along the pond bottom under the liner.

The pond embankment will be placed as uniformly as practicable throughout its length. Excavation for installation of the low-level outlet concrete pipes will minimise the depth of excavation of a slot through the partially constructed embankment.

Access to the pond crests will be constructed as placement of Zone 4 bulk fill progresses.

Surface of the Zone 4 and pond base materials will be prepared for placement of the geomembrane by proof rolling with a 10 to 20 tonne vibratory steel roller or as determined during membrane placement trials.

Prior to commencement of geomembrane liner installation, layout of geomembrane panels will be planned and drawn with the panels sized to determine configuration and location of the seams. Each panel will be given an alpha-numeric identifier and will be fabricated to size in the factory. Geomembrane placement will be carried out as recommended by the manufacturer during suitable weather conditions; no extreme temperatures, no rain and low wind.

As-built drawings will record placement of the geomembrane panels along with location of joints in the geomembrane.

As required by Resource Consent, placement of topsoil and establishment of vegetation on completed batters and berms above operating water levels will be advanced as surfaces are completed and as seasonal conditions permit.

14.0 Commissioning

Commissioning of Wrights Road Storage Pond provides the first test for the design and construction and will be completed when the ponds are full and stable seepage conditions are established.

Experience has shown that inherent safety problems are often disclosed during commissioning and the initial year of full operation. Strategies which maximise the opportunity to monitor performance against expectations are therefore critically important during this early period from a dam safety viewpoint.

A commissioning plan will be developed during the construction phase. This is initially to test operation of the control system (dry testing) and, subsequently to initiate filling of the ponds. This will include a program for observation of performance by visual inspection, piezometer readings and observation of operation of the hydraulic structures. In accordance with the NZDSG, commissioning to achieve and maintain safety will involve the following as a minimum:

- Carrying out the commissioning of the dam and respective components in accordance with pre-planned written procedures after final construction inspections by the responsible Designer and respective Technical Specialists and recommendation by the Designer that commissioning may commence;
- Acquainting all those involved in executing and monitoring the commissioning of their roles and responsibilities including how to react in the case of measured performance not complying with expectations or safe limits;
- Continuing commissioning and initial performance surveillance as necessary until the responsible Designer is able to certify to the Owner that the dam can be considered fully operational within written operating and maintenance procedures;
- Completing commissioning with a suitably comprehensive commissioning report which documents all activities, performance and changes made to works if any, so that there is a permanent record for future evaluations; and
- Explicit review and monitoring of all seepages which must be interpreted against the sealing methods and foundations issues.

15.0 Operation, Maintenance & Surveillance

An Operations, Maintenance and Surveillance Manual shall be prepared by the Contractor during the construction period and prior to the commissioning phases. This shall comprehensively cover all operation and maintenance aspects of the scheme.

The Contractor shall submit the operation and maintenance section of the manual to the Designer at least two weeks prior to commencement of commissioning for review.

In accordance with NZDSG, an operations, maintenance and surveillance manual will cover the following principal areas:

- Summary description of dam with selected drawings and references to other documents (e.g. as-built drawings);
- Procedures for operating functional components;
- Owner's health and safety requirements;
- Routine equipment and structure maintenance procedures;
- Equipment overhaul requirements;
- Inspection and monitoring requirements;
- Evaluation procedures, definition of unacceptable behaviour and actions to be taken; and
- Emergency procedures including special safety assessment inspections required to be carried out after earthquakes.
- Minimum list of back-up replacements required to be maintained on site.

In addition, an annual mass balance check of water into and out of the ponds shall be conducted as part of Intermediate Dam Safety Review (IDSR). The purpose of this check is to annually detect for gross leakage from the ponds. This should be done by observing the pond water level, with the ponds close to full and no inflows or outflows over a period of 24 hours, then comparing the change in water level with allowance for precipitation and evaporation.

16.0 Dam Safety Monitoring

As required by the Resource Consent, a Dam Safety Management System will be prepared prior to commissioning of the pond. This will provide for safe operation in accordance with the NZ Dam Safety Guidelines and appropriate to a High PIC Dam.

The Dam Safety Management System will include the following components:

a) Governance and people
b) Pond and reservoir operation and maintenance; including monitoring of the performance of the HDPE liner
c) Surveillance
d) Appurtenant structures and valve systems
e) Intermediate dam safety reviews
f) Comprehensive dam safety reviews
g) Special Inspections and dam safety reviews
h) Emergency Preparedness
i) Identifying and managing dam safety issues
j) Information Management
k) Audits and reviews

The following subsections outline some of these procedures.

16.1 Pond Operation

Normal operation of the pond will be managed by Waimakiriri Irrigation Limited with assistance from an automated control system. Flows into the pond are from the Waimakariri River, via the main canal and Buffer Pond. There are automated controls which will reduce and then shut-off as the water level in the ponds approach their Full Supply Levels. Abstractions from the ponds into the races are also dependent on both the demand and the water levels in the pond. The water levels are recorded by water level bubbler sensors in each of the ponds. The pond water levels are then fed back to the automated control system which controls the inflows and outflows from the pond.

The control system will manage filling and emptying of the ponds with automatic control of hydraulic gates to prevent overfilling of the pond. Alarms to indicate problems that need to be resolved quickly, such as sensor failure or overfill pond levels will be conveyed to the scheme operators.

The control system will include a manual operation mode to enable on the spot management of the ponds under emergency or unusual conditions to allow the operator to control the scheme operation on-site. In emergency situations, the emergency spillways and fuse plug spillway are the secondary controls to prevent overtopping of the embankments.

16.2 Maintenance

Maintenance activities will primarily be control of vegetation on the embankment slopes and repair of any erosion by runoff from intense rainfall. It will also include maintenance and testing of the control system and gates. The pond water level sensors will have regular functional and calibration checks. The piezometers will also require periodic response testing.

Any puncturing or tearing of the HDPE lining identified during inspections will be repaired.

16.3 Surveillance

Surveillance will consist of periodic inspection and monitoring of the pond embankments, inlet and outlet locations, liner system condition and the emergency spillways. Irrigation operators involved in surveillance will receive training in surveillance procedures and the applicable potential failure modes.

Results of the surveillance will:

- Build an accurate history of dam safety observations both visual and instrumented;
- Allow the performance of the dam to be regularly assessed and reported;
- Enable the early detection and mitigation of potential deficiencies or adverse trends; and
- Fulfil legislative and regulatory requirements.

Instrument readings and visual inspection observations will be recorded by the operators on prepared forms and reported to the Dam Manager, who will manage the assessment of the ponds and appurtenant structures performance based on the data recorded. A monthly surveillance report assessing the performance, any alarm conditions and intervention actions taken will be prepared.

Surveillance instrumentation are:

- Standpipe piezometers located on the crest of the pond embankments;
- Survey marks; located on the crest of the pond embankments;
- Pond water level sensors;
- Time Domain Reflectometry (TDR) system; cabling located along the crest of the pond embankments.

Routine visual inspections will be undertaken on a monthly basis. In addition, intermediate inspections will be carried out annually and Comprehensive Dam Safety Inspections will be carried out 5-yearly. Inspection check lists will be prepared taking into account the potential failure modes and will include visual inspection for liner condition and seepage both through the embankment and adjacent to the intake and outlet structures.

Surveillance alarms will be set to alert the Dam Manager to the need for investigation and action to ensure dam safety. Additional inspections and monitoring will be carried out in the event of extreme rainfall and after earthquakes of felt intensity MM6 or greater at the site.

16.4 Emergency Planning

As required by the Resource Consent, an emergency action plan will be developed in accordance with the NZ Dam Safety Guidelines prior to commissioning. The objective of the Emergency Action Plan is to minimise the potential for dam failure through pre-planned or pre-conceived intervention actions should a dam safety emergency event arise and, in the event that a dam failure cannot be prevented, to limit the effects of a dam failure on people, property and the environment.

17.0 Limitations

This report has been prepared by Damwatch Engineering Limited solely for the benefit of Rooney Earthmoving Ltd., and Waimakariri Irrigation Ltd. No Liability is accepted by Damwatch or any director, employee, contractor or sub-consultant of Damwatch with respect to its use by any other person.

This disclaimer shall apply notwithstanding that the report may be available for other persons for an application for permission or approval or to fulfil a legal requirement.

In order to assess the suitability and quantity of materials at the proposed site of the ponds, site investigation and soils testing was carried out by REL in 2011 and by MWH in 2007. This site investigation comprised a total of 41 test pits. A selection of samples from these test pits were laboratory tested as set out herein, in Chapter 6. This investigation and testing programme is considered the minimum necessary to obtain information to design the ponds and associated embankments. The design has been prepared on the basis of this site investigation, results of the testing programme, and the evaluation of this information, as set out in this report.

18.0 Conclusions

The key conclusions from design of the Wrights Road Storage Ponds are summarised as follows:

- The Wrights Road Storage Ponds footprint is approximately a square of 1 km x 1 km and located about 12 km downstream of the Waimakariri intake at Browns Rock.
- The storage ponds are divided by a middle embankment forming Pond 1 and Pond 2 with full supply level of Pond 1 RL 226.50 m being 3.7m higher than that of Pond 2 RL 222.80 m.
- The storage ponds are designed for 8.2 Mm^3 storage capacity.
- A different Potential Impact Classification can be applied to discrete lengths of the pond embankments, because the downstream consequences of breach vary depending on the section of the embankment breached. The western embankment, and embankment north of the tub are classified as a MEDIUM PIC and all other embankments are classified as a HIGH PIC (refer to Figure 1, page v).
- The ponds have been designed to meet the requirements in the NZSOLD Dam Safety Guidelines 2015;
- The embankments are water retaining structures where the embankment material as well as the backfill over the associated culverts must be selected at the time of construction to meet the specification;
- Lining of the ponds is necessary to control leakage into the sandy gravel foundation and bulk embankment fill;
- The geomembrane lining adopted for this design must be supported on a suitably prepared surface to perform adequately in its function to safely retain the contents of the reservoir over the life of the ponds; and
- Construction quality control is most important to ensure the ponds and appurtenant structures are built in accordance with the design.

All the construction activities shall be subjected to the quality control measures stated in the Specification bound separately as Appendix F.

References

AECOM. (2017). *Mangorei Power Scheme, 2017 Comprehensive Dam Safety Review.*

Bartsch, M. &. (2007). *Function Analysis and Strengthening by Adding a d/s Berm. Intermediate Report of the European Working Group of ICOLD.*

Bradley, B. A. (2012). Empirical correlations between peak ground velocity and spectrum-based intensity measures. *Earthquake Spectra, 28(1)*, 17-35.

Bray, J. &. (2007). *Simplified procedure for estimating earthquake-induced deviatoric slope displacements. Journal of Geotechnical and Geoenvironmental Engineering, ASCE 113(4): 381-392.*

Campbell, F. J. (2010). *Mapping and resorting Late Cretaceous - Present potentially seismogenic structures beneath the northwestern Canterbury Plains (New Zealand) based on hig.*

D2487-11, A. S. (2011). *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). ASTM International.*

Engineers, U. C. (1984). *Shoreline Protection Manual. Dept Army, Waterways Experiment Station, Coastal Engineering Research Centre.*

EPRI. (1987). *Design Guidelines for Pressure Tunnels and Shaft (EPRI AP-5273).* Berkeley: Electric Power and Research Institute.

Fell, R. M. (2015). *Geotechnical Engineering of Dams, 2nd edition. Leiden: CRC Press/Balkema.*

FEMA. (2005). *Technical Manual: Conduits through Embankment Dams. US Federal Emergency Management Agency.*

FEMA. (2011). *Filters for Embankment Dams - Best Practices for Design and Construction. US Federal Emergency Management Agency.*

Finnemore, M. (2004). *The application of seismic reflection surveying to the characterisaiton of aquifer geometry and related active tectonic deformation, North Canterbury. PhD Thesis, University of Canterbury.*

Fong, F. &. (1995). *Transverse cracking in embankment dams due to earthquakes. ASDSO Western Regional Conference. .*

Foster, M. &. (2000). *Use of event trees to estimate the probability of failure of embankment dams by internal erosion and piping . ICOLD 20th Congress on Large Dams. Beijing.*

Giroud, J. (2012). *Tekapo Canal Remediation Project, Analysis of Geomembrane Liner Behaviour Over a Crack in the Supporting Soil. A report prepared by J. P. Giroud, Inc. for Genesis Energy Ltd, dated 03/03/2012.*

Giroud, J. P. (1989). *Leakage through Liners Constructed with Geomembranes - Part II. Composite Liners. Geotextiles and Geomembranes, Vol. 8, Elsevier Science Publications Limited.*

Giroud, J. P. (2013). *Hydropower canal geomembrane liner analytical techniques. Proc. 2013 NZSOLD/ANCOLD Conference, Rotorua, New Zealand.*

Giroud, J. P.-T. (1989). *Evaluation of Leakage through Composite Liners. Geotextiles and Geomembranes, Vol. 8., Elsevier Science Publishers Limited.*

Giroud, J. P.-T. (1989). *Evaluation of Leakage through Composite Liners. Geotextiles and Geomembranes, Vol. 8., Elsevier Science Publishers Limited.*

GNS. (2011). *Greendale Fault: Investigation of Surface Rupture Characteristics for Fault Avoidance Zonation. A report prepared by GNS Science, Ref: 2011/121.*

GNS. (2015). *Seismic Hazard Spectra for Mangamahoe Dam.* GNS Science Consultancy.

GRI. (2005). *Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions. Geosynthetic Institute: White Paper #6.*

ICOLD. (2010). *Geomembrane Sealing Systems for Dams. ICOLD Bulletin No. 135.*

Koerner, R. M. (2005). *Designing with Geosynthetics (5th Edition). Prentice Hall.*

Kulhawy, F. &. (1990). *Manual on Estimating Soil Properties for Foundation Design. Electric Power Research Institute.*

Makdisi, F. a. (1978). *Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. Journal of the Geotechnical Engineering Division, ASCE, Vo. 104, No. GT7, 849-867.*

McVerry, G. Z. (2006). *New Zealand acceleration response spectrum attenuation relations for crustal and subduction zone earthquakes. Bulletin of the New Zealand Society for Earthquake Engineering, 39(1): 1-58.*

Mosonyi, E. (1991). *Water Power Development.* Budapest: Akadémiai Kiadó.

MWH. (2007). *Waimkariri Irrigation Ltd, Storage Reservoir, Feasibility Study - Stage 1, 5 June 2007.*

NPCED. (1987). *Evaluation of the Safety of Mangamahoe Dam.* New Plymouth City Energy Department.

NZ Standards. (2004). *NZS1170 Part5: Earthquake Actions.* Standards New Zealand.

NZ Standards. (2006). *NZS 3101, Concrete Structures Standard, Amendment 3.* Standards New Zealand.

NZGS. (2005). *Field Description of Soil and Rock. New Zealand Geotechnical Society Inc.*

NZSEE. (1980). *Seismic Design of Bridges, Section 9, Earthquake Retaining Structures. Bulletin of the NZ Society of Earthquake Engineering, September 1980.*

NZSEE. (2017). *The Seismic Assessment of Existing Buildings - Technical Guidelines for Engineering Assessments.*

NZSOLD. (2015). *New Zealand Dam Safety Guidelines.* Wellington: New Zealand Society on Large Dams.

NZTA. (2016). *Bridge Manual, 3rd edition.*

Pells, S. &. (2002). *Damage and cracking of embankment dams by earthquakes, and the implications for internal erosion and piping. UNICIV Report R 406, The University of New South Wales.*

Pickford and Grilli. (2012). *Mangorei Hydroelectric Power Scheme, Dam Safety Review.*

Raphael, J. (1984). *Tensile Strength of Concrete.* ACI Journal.

Re-Generate. (2018). *Mangorei Forebay Flood and Seismic Assessment, MGR-0002-REP Mangorei Forebay Gate Report.* ReGenerateNZ Limited.

Riley. (2012). *Mangorei Hydro Scheme, Tunnels and Tailrace Inspection 2012.*

Riley. (2016). *Mangorei Hydro Electric Power Scheme, Mangamahoe Dams Slope Stability Review.*

Riley. (2020). *Mangorei Hydro Scheme, Liquefaction Effects on Dam Stability, Mangamahoe Main Dam.*

Rocscience Inc. (2002). RocLab Version 1.0 - Rock Mass Strength Analysis using the Generalized Hoek-Brown failure criterion. Toronto, Ontario, Canada: www.rocscience.com.

Sherard, J. L. (1984). *Basic properties of sand and gravel filters. Journal Geotech. Eng. (ASCE), Vol. 100, No.6, 684-700.*

Stirling, M. G. (2008). *Seismic hazard of the Canterbury region, New Zealand: New earthquake source model and methodology. Bulletin of the NZ Society for Earthquake Engine.*

Stirling, M. M. (2010). *The national seismic model for New Zealand: 2010 Update. Bulletin of the Seismological Society of America, Vol 102, No.4, pp 1515-1542. .*

Swaisgood, J. (1998). *Seismically-induced deformation of embankment dams. 6th US National Conference on Earthquake Engineering.*

T&T. (1987). *Evaluation of the Safety of Mangamahoe Dam.* Tonkin & Taylor Ltd. (prepared for New Plymouth City Energy Department).

Thomlinson, A. &. (1992). *Probable maximum precipitation in New Zealand. Electricity Corporation of New Zealand Ltd.*

Tokimatsu, K. &. (1984). *Simplified Procedures for the Evaluation of Settlements in Clean Sands. Report No. UCB/EERC-84/16, October: University of California, Berkeley.*

Trustpower. (2004). *Mangorei Scheme Databook.*

Trustpower. (2021). *Mangorei Hydroelectric Power Scheme, Mangamhoe Reservoir, Operations and Maintenance Manual.*

USACE. (1995). *Planning and Design of Hydroelectric Plant Structures, Engineer Manual EM 1110-2-3001.* Washington: US Army Corps of Engineers.

USACE. (1997). *Tunnels and Shafts in Rock, Engineer Manual EM 1110-2-2901.* Washington: US Army Corps of Engineers.

USBR. (1992). *Freeboard criteria and guidelines for computing freeboard allowance for storage dams. ACER Technical Memorandum 2, US Bureau of Reclamation.*

USBR. (2014). *Design Standards No. 13: Embankment Dams, Chapter 8: Seepage. U.S. Department of Interior Bureau of Reclamation.*

Wang, J.-N. (1993). *Seismic Design of Tunnels, A Simple State-of-the-Art Design Approach (1991 William Barclay Parsons Fellowship. Parsons Brinckerhoff. Monograph 7).* New York: Parsons Brinkerhoff Inc.

Wood, J. H. (2005). *Earthquake Design of Rectangular Underground Structures.* 2005 NZSEE Conference, New Zealand Society for Earthquake Engineering.

APPENDIX A Test Pit Data

APPENDIX B Soil Lab Test Results

APPENDIX C Site Seismicity

APPENDIX D Embankment Slope Stability Analysis

APPENDIX E Drawings

Drawings are bound separately

APPENDIX F Specifications

Technical Specification is bound separately

APPENDIX G Control System

APPENDIX H Dam-Break Analysis & Potential Impact Classification

APPENDIX I Additional pga Ground-shaking and Surface Deformation Hazard Studies for the Wrights Road Storage Ponds

APPENDIX J Producer Statements (PS1,PS2) and Peer Review Statements

APPENDIX K GNS Seismic Hazard Report

APPENDIX L GNS Ground Motion Recommendations Report

APPENDIX M Safety in Design Workshop Register (Detailed Design Phase)

APPENDIX N Log Boom Design Information