## Document Control

**Title:** Waimea Dam

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<th>Date</th>
<th>Version</th>
<th>Description</th>
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<td>11/06/18</td>
<td>DRAFT 1</td>
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Executive summary

This report summarises the detailed design undertaken for the proposed Waimea Dam, Tasman District. The dam’s purpose is water augmentation for irrigation and community water supply to provide drought security to the Waimea Plains. The dam is intended to supplement the Lee River’s natural flows to provide a constant residual flow as well as an irrigation flow. The Waimea Dam does not have a specific flood attenuation design function. This project is anticipated to provide significant regional benefits.

The dam proponent is Waimea Water, which is owned in part by Tasman District Council and Waimea Irrigators Ltd. Funders for the project include Waimea Irrigators Ltd, Tasman District Council, Crown Irrigation Fund, and Nelson City Council.

The Tonkin & Taylor Ltd (T+T) lead design team have been engaged by Tasman District Council for the permanent works design. The Stage 4 design included early contractor involvement (ECI) to inform the permanent works design, develop temporary works design arrangements, and determine the contract price for construction. The ECI Contractor was Fulton Hogan Taylors Joint Venture (FHTJV).

The dam design was been developed in a staged approach with feasibility design in 2009, and design to support the resource consent application (Stage 3) in 2012 and 2014. Peer reviews have been undertaken at each design stage. The detailed design presented in this report is Stage 4 and is intended to support the building consent application and be the basis for Waimea Water’s design documentation.

The proposed dam site is located on the Lee River approximately 40 minutes by car to the south of Nelson. The Lee River is a tributary of the Waimea River. The proposed Waimea Dam is a concrete face rockfill dam (CFRD) up to 53 m in height and 220 m long at crest level. The dam will impound a 13 Mm³ reservoir at normal top water level of 197.2 m RL. The reservoir may be drawn down to a minimum operating level of RL 166.5 m.

The dam is classified as a high PIC (Potential Impact Classification) dam in accordance with New Zealand Society on Large Dams New Zealand Dam Safety Guidelines (NZSOLD DSG 2015). The dam is therefore designed to the highest standards currently applicable in New Zealand for dams.

The dam design presented in this report includes the following components:

- A zoned rockfill embankment founded on rock with concrete facing on the upstream slope.
- 4 m high reinforced concrete parapet wall on the dam crest.
- A twin barrelled diversion culvert underneath the dam and located on the true right bank.
- A nominally 6.5 m high mass concrete starter dam across the valley at the upstream toe of the dam.
- A horizontal type reinforced concrete plinth founded on the starter dam in the river channel and on rock in the abutments.
- An ungated oggee weir controlled concrete lined spillway located on the true left abutment of the dam.
- Access roads to the dam from existing forestry road.
- Two access bridges for access to the dam crest and the toe berm area.
- Two outlet pipelines with submerged intake screen structures, isolation valves, and discharge valves at the toe of the dam. The outlet works control the usual flow released from the dam (residual, irrigation, flushing) and enable dewatering of the reservoir.
- An open channel fish pass located on the true right bank of the dam.
• Dam safety instrumentation to enable real time remove monitoring of the dam performance.

The permanent works design considers the construction diversion works proposed by the ECI Contractor (FHTJV). Key uncertainties and construction considerations are highlighted throughout this report to clarify the design intent and limitations.

Designer involvement during construction is essential especially for review of foundations, rockfill material and permanent cut batter slopes. In particular, a Foundation Committee is required to inspect and approve the foundations prior to placement of embankment material and structures. Localised over-excavation and backfill with concrete and/or specific foundation treatment is anticipated.

T+T emphasise that the design of the dam will not be complete until the dam is fully commissioned and functioning. This is normal for any dam project that must be assessed and if necessary adapted to the site conditions as they are encountered. The design and peer review team must therefore work closely with the Contractor, Engineer to the Contract, the regulator (Building Consent Authority (BCA) and the Owner to complete the dam in a safe and satisfactory manner.

The permanent works design also considers future provisional installation of a mini-hydropower station at the toe of the dam noting that some amendments will be required should the mini hydro station be constructed after the main dam.
Abbreviations

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<tr>
<td>AEP</td>
<td>Annual Exceedance Probability</td>
</tr>
<tr>
<td>ALARP</td>
<td>As Low as Reasonably Practicable</td>
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<tr>
<td>ARI</td>
<td>Average Recurrence Interval</td>
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<td>Cd</td>
<td>Coefficient of Discharge</td>
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<td>CDF</td>
<td>Construction Diversion Flood</td>
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<td>CFRD</td>
<td>Concrete Face Rockfill Dam</td>
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<td>CL</td>
<td>Centreline</td>
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<td>d/s</td>
<td>Downstream</td>
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<td>Dam Safety Emergency Plan</td>
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<td>Earthquake Severity Index</td>
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<td>GWh</td>
<td>Gigawatt hour</td>
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<td>Hs</td>
<td>Significant wave height</td>
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<td>Intergovernmental Panel on Climate Change</td>
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<td>LIDAR</td>
<td>Light Detection And Ranging</td>
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<td>MALF</td>
<td>Mean Annual Low Flow</td>
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<td>Inflow Design Flood</td>
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<td>MFE</td>
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<tr>
<td>m RL</td>
<td>metres Reduced Level</td>
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<td>NIWA</td>
<td>National Institute Of Water &amp; Atmospheric Research</td>
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<td>HIRDS</td>
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<td>New Zealand Society on Large Dams</td>
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<td>OBE</td>
<td>Operational Basis Earthquake</td>
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<td>PAR</td>
<td>Population at Risk</td>
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<td>Parsons Brinckerhoff/WSP</td>
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<td>Peak Ground Acceleration</td>
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<td>PMF</td>
<td>Probable Maximum Flood</td>
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<td>Probable Maximum Precipitation</td>
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<td>Roller Compacted Concrete</td>
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<td>Run up of significant wave</td>
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<td>Tonkin &amp; Taylor Ltd</td>
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<td>T_p</td>
<td>Wave period</td>
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<td>TPA</td>
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<tr>
<td>UDL</td>
<td>Uniformly Distributed Load</td>
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<tr>
<td>USACE</td>
<td>US Army Corps of Engineers</td>
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<tr>
<td>USBR</td>
<td>Reclamation (formally US Bureau of Reclamation)</td>
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<tr>
<td>WL</td>
<td>Water Level</td>
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<tr>
<td>WW</td>
<td>Waimea Water</td>
</tr>
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<td>WWAC</td>
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1 Introduction

1.1 General

This report summarises the detailed design undertaken for the proposed Waimea Dam, Tasman District. The dam’s purpose is water augmentation for irrigation and community water supply to provide drought security to the Waimea Plains. The dam is intended to supplement the Lee River’s natural flows to provide a constant residual flow as well as an irrigation flow. The Waimea Dam does not have a specific flood attenuation design function.

The proposed dam site is located on the Lee River approximately 40 minutes by car to the south of Nelson. The Lee River is a tributary of the Waimea River.

The proposed Waimea Dam is a concrete face rockfill dam (CFRD) up to 53 m in height. The dam is located at chainage (horizontal distance measured) 12,430 m upstream from the confluence between the Wairoa and Lee rivers as shown in Figure 1.1 and Figure 1.2 below. The Waimea Dam site is accessed by forestry roads off Lee Valley Road as shown on Figure 1.2.

![Figure 1.1: Regional location of the proposed dam.](image)

The approximate NZTM coordinates of the dam location are 1613437 mE, 5409020 mN.
1.2 Proposed Dam

The project comprises the construction of a concrete face rockfill dam (CFRD) which will impound a 13 Mm$^3$ reservoir at normal top water level, and is located in the upper Lee Valley approximately 200 m upstream of Anslow Creek. The Lee River is one of two major tributaries of the Wairoa River which drains the Richmond Range east of the Waimea Plains. The Wairoa River is then joined by the Wai-iti River, and together they form the Waimea River.

The dam is located at Chainage 12,430 m (measured upstream from the confluence of the Wairoa and Lee Rivers). The dam would be approximately 53 m high and 220 m long at crest level. The location and layout of the dam is shown in Figure 1.3 below.

The storage reservoir will have a top water level of RL 197.2 m and will extend approximately 3.7 km upstream of the dam. The arms of the reservoir will extend approximately 1 km into Waterfall Creek on the right bank, and 350 m into Flat Creek on the left bank. The reservoir may be drawn down to a minimum operating level of RL 166.5 m.

The CFRD will be constructed from approximately 430,000 m$^3$ of locally sourced rockfill. Appurtenant structures associated with the dam include a spillway and a diversion culvert, the latter to be used after construction to house the outlet works.

The dam is classified as a high PIC (Potential Impact Classification) dam in accordance with New Zealand Society on Large Dams New Zealand Dam Safety Guidelines (NZSOLD DSG 2015). The dam is therefore designed to the highest standards currently applicable in New Zealand for dams.
1.3 Project background and timeline

Waimea Water Augmentation Committee (WWAC) engaged T+T to undertake both pre-feasibility and feasibility designs of the storage dam. Feasibility was completed in December 2009. The feasibility study concluded that a CFRD at Chainage 12,430 m was the most appropriate location and dam type.

In December 2010, T+T was appointed by WWAC to undertake detailed design of the dam. The design was put on hold following completion of the Stage 3 Design in late 2012 at the Client’s instruction. This was such that funding and resource consent/s could be procured.

Following procurement of resource consent, the project developer became Waimea Water; a joint venture between Tasman District Council and Waimea Irrigators Ltd. T+T as permanent works designer was novated from WWAC to Tasman District Council in around 2014.

The detailed design recommenced in late 2017 (termed Stage 4), and was to be substantially completed by June 2018 as part of an Early Contractor Involvement (ECI) project phase which would culminate in the Contractor submitting a price for the project for consideration by Waimea Water. The current detailed design phase has not re-considered dam type, location or dam storage volume requirements since these key criteria were peer reviewed and endorsed by WWAC prior to completion of feasibility and subsequently reviewed by Beca in 2015.

Key engineering reports documenting the design development preceding the Stage 4 detailed design are listed in Table 1.1.

Figure 1.3: Location and layout of the Waimea Dam.
Table 1.1: Selected previous design reports and letters

<table>
<thead>
<tr>
<th>Title</th>
<th>Date</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lee Valley Dam Feasibility Investigations Geotechnical Investigations Report</td>
<td>December 2009</td>
<td>24727.204</td>
</tr>
<tr>
<td>Lee Valley Storage Dam Engineering Feasibility Report</td>
<td>December 2009</td>
<td>24727.303</td>
</tr>
<tr>
<td>Waimea Water Augmentation Phase 2 - Water Resource Investigations</td>
<td>December 2009 (Rev 1.0)</td>
<td>24727.100</td>
</tr>
<tr>
<td>Waimea Water Augmentation Phase 2 - Lee Valley Dam Feasibility Investigations - Summary Report</td>
<td>February 2010</td>
<td>24727.800</td>
</tr>
<tr>
<td>Stage 3 report (appendices include hydropower feasibility report; GNS 2011 report, PB Power M&amp;E preliminary design report)</td>
<td>October 2012 (reissued for Resource Consent in July 2014)</td>
<td>27425.100</td>
</tr>
<tr>
<td>Lee Valley Dam Response to Peer Review of Stage 3 Design</td>
<td>December 2013</td>
<td>27425.100</td>
</tr>
<tr>
<td>Waimea Community Dam: Cost/Risk and alternative options review for an affordable dam (Beca)</td>
<td>28 April 2015</td>
<td>Beca ref 3311210</td>
</tr>
<tr>
<td>Seismic Hazard Assessment for the Proposed Waimea Dam (GNS) (update of 2011 report)</td>
<td>September 2017</td>
<td>GNS ref 2017/150</td>
</tr>
<tr>
<td>Seismic Design of the Waimea Dam</td>
<td>30 November 2017</td>
<td>1002177</td>
</tr>
<tr>
<td>Factual Geotechnical Report</td>
<td>February 2018</td>
<td>27425.100</td>
</tr>
<tr>
<td>Embankment Trial Testing Factual Geotechnical Report</td>
<td>May 2018</td>
<td>27425.100</td>
</tr>
</tbody>
</table>

Extensive non-engineering investigations and reports were produced as part of the feasibility stage. These are summarised in the report titled "Waimea Water Augmentation Phase 2 - Lee Valley Dam Feasibility Investigations - Summary Report T+T Ref 24727.800". Pre-feasibility documents are not listed in Table 1.1 because they investigated regional solutions for water augmentation and are therefore not directly relevant to the current detailed design phase.

1.4 Project structure

1.4.1 Roles and responsibilities

The following organisations were involved in the detailed design of the Waimea Dam.

<table>
<thead>
<tr>
<th>Role</th>
<th>Organisation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principal</td>
<td>Waimea Water (formally Waimea Water Augmentation Committee, WWAC) – Project delivery vehicle comprising of Waimea Irrigators Ltd and Tasman District Council</td>
</tr>
<tr>
<td>Unitary (Territorial and Regional) Authority</td>
<td>Tasman District Council</td>
</tr>
<tr>
<td>Funders</td>
<td>Waimea Irrigators Ltd (WIL)</td>
</tr>
<tr>
<td></td>
<td>Tasman District Council</td>
</tr>
<tr>
<td></td>
<td>Nelson City Council (NCC)</td>
</tr>
<tr>
<td></td>
<td>Crown Irrigation Fund (CIF)</td>
</tr>
<tr>
<td>Designer</td>
<td>Tonkin &amp; Taylor Ltd (T+T)</td>
</tr>
<tr>
<td>Permanent Works Designer</td>
<td>Mott MacDonald (sub-consultant to T+T)</td>
</tr>
<tr>
<td>E&amp;M works designer</td>
<td>WSP</td>
</tr>
</tbody>
</table>
Specific roles within the design team organisations and the interaction pathways with the ECI Contractor and Principal are presented in Figure 1.4 below.

Figure 1.4: Design team organisation chart.
1.4.2 Design scope

The design scope for T+T has been developed from pre-feasibility through to detailed design. T+T’s (with its sub-consultants) specific scope for the Stage 4 detailed design is covered in the proposal “Waimea Dam Stage 4 – Dam Engineering Design and Construction Services” dated 7 December 2017 (T+T Job No. 27425.1000). In brief, this scope includes:

- Permanent dam works.
- M&E works (excluding intakes).
- Electrical site distribution.

T+T design scope does not extend design or design review of:

- Temporary works (including cut and batter slope design).
- Diversion work.
- Lee Valley access road and bridge design.
- Landowner access design.
- Spoil disposal design.
- Erosion sediment control.
- Consent management or compliance.
- Fish trap and transfer.
- Future hydro power station.
- Construction contract administration.
- Procurement or procurement management.
- Transmission line design.
- Construction cost estimation, cost review or quantity scheduling.
- Maintenance or operation of the dam.

This report does not describe design of temporary works which is the responsibility of the Contractor.

1.4.3 Changes in design team from Stage 3 to Stage 4

The dam design from Stages 1 to 3 was led by Phil Carter and reviewed by Len McDonald (both former NSW Public Works Department Dam Engineers), both as sub-consultants to T+T. Len McDonald and Phil Carter had retired in the period from when the then dam developer Waimea Water Augmentation Committee put the dam design on hold until the design was recommenced in late 2017, and therefore were no longer available to complete the design. Both Len McDonald and Phil Carter were informed that the design was proceeding with a revised design team and had no objections.

Mott MacDonald was approached by T+T to provide specialist CFRD and dam expertise to complete the existing design in Stage 4. T+T considered that this was necessary given the lack of specialist CFRD experience in New Zealand. A summary letter for Mott MacDonald outlining their design involvement to date is included in Appendix J.

The change in design team was communicated to Tasman District Council and Waimea Water prior to commencement of the Stage 4 design.
1.4.4 Peer review

The Principal has engaged a number of peer reviewers for the design of the Waimea Dam. WWAC initially engaged Montgomery Watson Harza (MWH) to independently review the Engineering Feasibility Report (T+T, 2009) (noting the feasibility design had already been peer reviewed by Engineering Geology Ltd). Subsequently Opus International Consultants Ltd (Opus) (now WSP) was appointed by WWAC as their peer reviewer for Stage 3 detailed design in mid May 2011. Ian Walsh (WSP|Opus) has remained as the peer reviewer for the Stage 4 detailed design.

Table 1.2 summarises the delivery of key reports and meetings with the Principal, the peer reviewers, the date the peer review was received and the date of any response (if relevant). This illustrates the involvement that the peer reviewers have had to date with the design. In addition to the key documents listed in Table 1.2, there have been occasional email and telephone discussions between T+T and WSP | Opus.
Table 1.2: Summary of key peer review since completion of feasibility

<table>
<thead>
<tr>
<th>Description</th>
<th>Type</th>
<th>Date issued</th>
<th>Peer reviewer</th>
<th>Review received</th>
<th>Responded</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feasibility</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lee Valley Storage Dam Engineering Feasibility Report</td>
<td>Report</td>
<td>Dec 2009</td>
<td>MWH</td>
<td>July 2009</td>
<td>6 Oct 2010</td>
<td>The feasibility report has already been reviewed by Engineering Geology Ltd.</td>
</tr>
<tr>
<td>Detailed design (Stages 1 to 3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lee River Dam Project Quality Plan</td>
<td>Report</td>
<td>Feb 2011</td>
<td>Opus</td>
<td>NA</td>
<td>NA</td>
<td>Issued for information only.</td>
</tr>
<tr>
<td>Peer Review Site Visit accompanied By Mark Foley of T+T, and associated discussions including Joseph Thomas.</td>
<td>Site visit</td>
<td>1 June 2012</td>
<td>Opus</td>
<td>3 June 2011</td>
<td>12 Sept 2011</td>
<td></td>
</tr>
<tr>
<td>Stage 1 Design Report</td>
<td>Report</td>
<td>Sept 2011</td>
<td>Opus</td>
<td>25 Oct 2011</td>
<td>Stage 3 Design Report</td>
<td>Informal discussions have been had with Opus on specific aspects (e.g. climate change assumptions).</td>
</tr>
<tr>
<td>Discussion Paper on Procurement and Delivery Options</td>
<td>Report</td>
<td>28 June 2011</td>
<td>Opus</td>
<td>NA</td>
<td>NA</td>
<td>The discussion paper was discussed at Contractual workshop on 27 March 2012.</td>
</tr>
<tr>
<td>Risk Workshop</td>
<td>Workshop</td>
<td>26 Oct 2011</td>
<td>Opus</td>
<td>NA</td>
<td>NA</td>
<td>Risk workshop was interactive rather than having a formal review output.</td>
</tr>
<tr>
<td>Description</td>
<td>Type</td>
<td>Date issued</td>
<td>Peer reviewer</td>
<td>Review received</td>
<td>Responded</td>
<td>Comment</td>
</tr>
<tr>
<td>--------------------------------------------------</td>
<td>---------------</td>
<td>-------------</td>
<td>---------------</td>
<td>----------------</td>
<td>-----------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Lee River Dam – Hydropower design and interfacing</td>
<td>Letter</td>
<td>18 Jan 2012</td>
<td>Opus</td>
<td>8 Feb 2012</td>
<td>NA</td>
<td>Minutes issued to all attendees. The Opus peer review was for WWAC’s benefit. i.e. The comment did not require a response from T+T.</td>
</tr>
<tr>
<td>Contractual procurement workshop</td>
<td>Workshop</td>
<td>27 March 2012</td>
<td>Opus</td>
<td>NA</td>
<td>NA</td>
<td>Risk workshop was interactive rather than having a formal review output. Minutes issued to all attendees.</td>
</tr>
<tr>
<td>HAZOP Workshop</td>
<td>Workshop</td>
<td>27 March 2012</td>
<td>Opus</td>
<td>NA</td>
<td>NA</td>
<td>Risk workshop was interactive rather than having a formal review output. Minutes issued to all attendees.</td>
</tr>
<tr>
<td>Progress reports</td>
<td>Report</td>
<td>Various (Jan 2011 to Sept 2012)</td>
<td>Opus</td>
<td>None</td>
<td>NA</td>
<td>The progress reports have reported significant design developments (e.g. change from two spillways to a single spillway) to keep WWAC and the peer review informed of key decisions.</td>
</tr>
<tr>
<td>Lee Valley Dam - Hydropower Preliminary Design</td>
<td>Report</td>
<td>7 Aug 2012</td>
<td>D. Inch/Opus</td>
<td>None</td>
<td>NA</td>
<td>We understand that WWAC has received feedback from D Inch on the report. This has not been communicated to T+T.</td>
</tr>
<tr>
<td>Peer review reports 6-9</td>
<td>Report</td>
<td>7 June 2013</td>
<td>Opus</td>
<td>7 June 2013</td>
<td>Dec 2013</td>
<td>These reports by Opus reviewed the Stage 3 design documentation. T+T responded in December 2013.</td>
</tr>
<tr>
<td>Cost review workshop</td>
<td>Workshop</td>
<td>2015</td>
<td>WSP</td>
<td>NA</td>
<td>NA</td>
<td>Ian Walsh attended cost review workshops run by Beca.</td>
</tr>
</tbody>
</table>

**Stage 4 Detailed design and ECI**

<p>| ECI RFP                                          | Workshop      | 2017        | WSP           | NA             | NA        | Ian Walsh attended procurement workshops to |</p>
<table>
<thead>
<tr>
<th>Description</th>
<th>Type</th>
<th>Date issued</th>
<th>Peer reviewer</th>
<th>Review received</th>
<th>Responded</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site visit with T+T and Mott McDonald</td>
<td>Site visit</td>
<td>Dec 2017</td>
<td>WSP</td>
<td>NA</td>
<td>NA</td>
<td>Site visit for design team following hiatus between Stage 3 and Stage 4 design.</td>
</tr>
<tr>
<td>Diversion temporary works workshop</td>
<td>Workshop</td>
<td>March 2018</td>
<td>WSP</td>
<td>NA</td>
<td>NA</td>
<td>Attendee for part of a workshop with FHTJV exploring alternative starter dam and conduit levels.</td>
</tr>
<tr>
<td>Other interactions</td>
<td>Meeting minutes, telecons and key communications (emails, memos etc)</td>
<td>2017-current</td>
<td>WSP/Damwatch</td>
<td>Various</td>
<td>Various</td>
<td>There has been regular interaction keeping WSP informed of key issues and progress.</td>
</tr>
</tbody>
</table>

1.5 Summary of key information

Table 1.3 below summarises key information related to the dam design.

**Table 1.3: Key design information summary**

<table>
<thead>
<tr>
<th>Embankment characteristics</th>
<th>Concrete Face Rockfill Dam (CFRD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment type</td>
<td></td>
</tr>
<tr>
<td>Embankment volume (approximate)</td>
<td>435,000 m³</td>
</tr>
<tr>
<td>Nominal crest elevation (excluding camber)</td>
<td>201.23 m RL</td>
</tr>
<tr>
<td>Top of parapet wall (excluding camber)</td>
<td>203.13 m RL</td>
</tr>
<tr>
<td>Design Camber</td>
<td>0.3 m</td>
</tr>
<tr>
<td>Maximum dam height (from riverbed to dam crest on CL)</td>
<td>53 m</td>
</tr>
<tr>
<td>Crest length (approximately)</td>
<td>220 m</td>
</tr>
<tr>
<td>Crest width (excluding abutment turning area)</td>
<td>6 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hydrology, reservoir and flood routing characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Catchment area</td>
<td>77.5 km²</td>
</tr>
<tr>
<td>Normal top water level (NTWL)</td>
<td>197.2 m RL</td>
</tr>
<tr>
<td>Reservoir storage at NTWL</td>
<td>13 Mm³</td>
</tr>
<tr>
<td>Reservoir area at NTWL</td>
<td>630,000 m²</td>
</tr>
<tr>
<td>Inflow design flood peak water level (IDFL)</td>
<td>202.53 m RL</td>
</tr>
<tr>
<td>Reservoir storage at IDFL</td>
<td>16.6 Mm³</td>
</tr>
<tr>
<td>--------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>200 year ARI flood level</td>
<td>200.48 m RL</td>
</tr>
<tr>
<td>Reservoir storage at 200 year ARI flood</td>
<td>15.2 Mm³</td>
</tr>
<tr>
<td>Reservoir storage at top of parapet wall (203.13 m RL)</td>
<td>16.8 Mm³</td>
</tr>
</tbody>
</table>

**Spillway characteristics**

<table>
<thead>
<tr>
<th>Primary spillway type</th>
<th>Ogee Weir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ogee weir effective length (on arc)</td>
<td>41.89 m</td>
</tr>
<tr>
<td>Peak outflow – Mean Annual Flood (MAF)</td>
<td>179 m³/s</td>
</tr>
<tr>
<td>Peak outflow – 200 year ARI flood</td>
<td>472 m³/s</td>
</tr>
<tr>
<td>Peak outflow – Inflow Design Flood (IDF) (PMF)</td>
<td>1060 m³/s</td>
</tr>
<tr>
<td>Dam crest flood outflow – Reservoir level at top of parapet wall</td>
<td>1152 m³/s</td>
</tr>
</tbody>
</table>

**Spillway and Energy dissipation characteristics**

| Chute length (plan – ogee crest to start of flip bucket) | 124 m |
| Chute width, narrow section | 20 m |
| Chute horizontal transition length | 71 m |
| Chute vertical curve length | 21 m |
| Chute minimum height of concrete lining | 3.0 m |
| Dissipation type | Flip Bucket |
| Flip bucket radius | 20 m |
| Bucket lip level | 156.6 m RL |

**Outlet characteristics**

| Outlet type | Sloping pipes on the upstream face with removable screens and valve control. |
| Number of outlets | 2 |
| Outlet level – Upper (elevation of top of bellmouth) | 181.5 m RL |
| Outlet level – Lower (elevation of top of bellmouth) | 163.0 m RL |
| Pipe diameter and material | DN1000 epoxy coated steel |
| Control type | Fixed Cone Discharge Valves (2x DN850 and 2x DN300 valves) |
| Butterfly isolation valves (2x DN1000) | Butterfly isolation valves (2x DN1000) |
| Maximum design discharge capacity | 20 m³/s (dewatering) |
| (No valve manufacturer velocity limits applied) | (dewatering) |
| Concrete conduit size under embankment (internal dimensions) | Twin 2.5 m wide x 4.0 m high |

**River tailwater characteristics (at confluence with spillway)**

| Tailwater level MAF | 150.85 m RL |
| Tailwater level 200 year ARI | 153.46 m RL |
| Tailwater level IDF (PMF) | 156.54 m RL |
Irrigation and environmental flow release

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak irrigation release at dam toe (at minimum operating level</td>
<td>2.23 m³/s</td>
</tr>
<tr>
<td>and from either intake)</td>
<td></td>
</tr>
<tr>
<td>Environmental residual flow (7 day Mean Annual Low Flow (MALF)</td>
<td>0.51 m³/s</td>
</tr>
<tr>
<td>at minimum operating level and from either intake)</td>
<td></td>
</tr>
<tr>
<td>Environmental flushing flow (at minimum operating level and</td>
<td>5.0 m³/s</td>
</tr>
<tr>
<td>from either intake)</td>
<td></td>
</tr>
</tbody>
</table>

(1) Note that actual valve sizes may differ upon procurement to suit manufacturers design.

(2) The design capacity of the outlet is the largest of the requirements of 5.0 m³/s and is not additive (i.e. It is not 2.23 + 5 + 0.51).

1.6 Storage elevation curve

Figure 1.5 shows the current design storage elevation curve developed for the Waimea Dam. This has been developed using contours derived from LiDAR supplied to T+T and accounts for the design finished surfaces. The curve does not account for any changes in the reservoir due to:

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

The storage elevation curve used in the previous flood routing (as presented in the Stage 3 report) did not account for the volume of the dam itself. This resulted in a very slight potential overestimation of the reservoir storage volume but is not considered significant in the context the overall accuracy of the curve and the flood routing assessment.
1.7 Site survey

The design for the Waimea Dam is based on the following supplied topographical information:

1. Light Detection and Ranging data (LiDAR) provided by New Zealand Aerial Mapping Ltd (NZAM), who also supplied ortho-corrected photos from digital imagery captured at the same time as the LiDAR on 18 May 2011.
2. Ground survey data provided by Staig & Smith Ltd, undertaken progressively between February 2011 and March 2012.
3. Additional ground survey data provided by Staig & Smith Ltd and undertaken in late 2017/early 2018.

All survey information used for design was supplied in NZTM coordinate projection and Nelson Vertical Datum 1955 (NVD1955).

Comparison and ground verification of the 2011 LiDAR using the ground survey was undertaken by Staig & Smith Ltd in 2012. This assessment was repeated in January 2018 and generally confirmed that the 2011 LiDAR was in close agreement (in the order of ±100 mm) with the ground survey in the areas that were cleared of vegetation/forestry before the LiDAR was flown.

However, in the areas that were not cleared before the LiDAR was flown, the reported difference between the LiDAR and ground survey was up to -100 mm to +1 m (i.e. the LiDAR surface is higher than the actual ground). The LiDAR survey was assessed by Staig & Smith Ltd to be sensitive to vegetation and post processing of the raw LiDAR has not accurately adjusted the finished surface levels in the heavily vegetated areas around the dam site.

This assessment by Staig & Smith suggests that the ground surface model (digital elevation model, DEM) used for the Waimea Dam design may overestimate the ground surface (i.e. ground surface appears higher than it actually is) in the locations that are currently heavily vegetated and were not covered by the terrestrial survey. Specific locations of interest are the true right abutment of the dam and the lower chute/flip bucket area, and less excavation may be necessary in these locations that currently shown in the design.

It is strongly recommended that the Contractor or the Engineer to the Contract undertakes another terrestrial survey of the entire dam site following full clearance and to enable the required design excavation volumes to be identified before commencement of excavation.

T+T recommends that Staig & Smith is engaged prior to construction to assist in positioning of long term monitoring site benchmarks/monoliths. These can be used both during construction and for post construction dam safety monitoring. These are not required to be installed until construction and in our opinion would likely be disturbed or destroyed if installed prior to the vegetation clearance.
2 Design criteria

2.1 Potential impact category

The proposed Waimea Dam has an assessed High Potential Impact Classification (PIC). This is the highest classification and requires the highest standards and criteria to be applied to the dam design.

The assigned PIC is based on the dam break assessment carried out during the feasibility study (T+T 2009). Further dam break assessment was undertaken in 2011 as part of the Stage 1 design (T+T, September 2011) to inform the construction diversion works rather than the completed dam PIC.

The PIC was reviewed against the New Zealand Dam Safety Guidelines 2015 as part of the Stage 4 design for completeness noting that a High PIC has been adopted (and therefore the adopted design criteria are at the highest level). A summary of this review and the 2009 dam break assessment are provided below.

Identification of the potential incremental consequences from hypothetical dam break failures are required for both determining the PIC and informing the Emergency Action Procedures (EAP). Therefore while a High PIC has been adopted for the purposes of design, review of the dam break assessment is relevant for review of the potential incremental consequences also.

Only a sunny day failure scenario was assessed as part of the 2009 dam break assessment. It was considered at that time unnecessary to also model a flood-induced failure scenario as a decision was taken in the feasibility design to provide sufficient spillway capacity at the dam to cope with the Probable Maximum Flood (the largest flood that could conceivably occur at that location) without relying on mechanically controlled spillway gates. The adopted spillway design is based on the proposed Tillegra Dam spillway (NSW, Australia) and has capacity to route the assessed dam crest flood also.

Potential failure of the dam due to overtopping following blockage of the spillway was considered as part of the March 2018 FMEA workshop (refer Section 2.2.1 and Appendix D). While this potential failure mode is considered to be credible and had the highest risk, it is also unlikely given the approximately 6 m of freeboard above NTWL, wide ungated spillway, and debris boom.

A flood induced overtopping type failure (i.e. a rainy day scenario) would require a very large flood, at which point the potentially inundated area downstream would already be flooded, and therefore the incremental consequences resulting from this hypothetical event are unlikely to be greater than the sunny day scenario modelled. Further, it is likely for such a large flood that emergency procedures will already have been actioned downstream in advance of a dam failure from overtopping.

The adopted dam breach parameters for the sunny day scenario were:

- Reservoir at NTWL (taken at 197 m RL rather than the updated 197.2 m RL, noting this will have a negligible effect).
- Breach side slopes of 1V:1H (true left) and 1V:1.3H (true right).
- Base width of 50 m (i.e. the width of the river channel).
- Three different breach formation times ($t_f$) based on empirical formulae of 0.9 hrs (“best” estimate), 0.5 hrs (upper bound) and 1.5 hrs (lower bound).

The breach discharge hydrographs was determined using the HEC-HMS modelling package and checked with published empirical methods (as documented in Wahl, 1998). The calculated peak discharges ($Q_p$) immediately downstream of the dam were 5000 m$^3$/s ($t_f = 1.5$ hrs) and ~13,500 m$^3$/s ($t_f = 0.5$ hrs), with the best estimate peak discharge of 8000 m$^3$/s ($t_f = 0.9$ hrs). Each of these
estimates was assessed to be in reasonable agreement with different recorded discharges from actual dam break event and empirical methods, represent the uncertainty in the peak discharge estimates.

The potential incremental consequences downstream were assessed using a coupled 1D-2D hydraulic model (Mike Flood). All three dam breach hydrographs were modelled.

The coupled hydraulic model was built from the Tasman DC supplied LiDAR data (including LiDAR derived cross sections) and is consider appropriate for the very large dam breach flood flows relative to the river geometry. The adopted hydraulic roughness values (Manning’s n) were river channel n = 0.03; flood plain n = 0.06; roads n = 0.016. A sensitivity check of the modelled flood extents (and therefore the zone of potential inundation) was undertaken for a lower flood plain hydraulic roughness (n = 0.033).

Calibration of the hydraulic model was not possible for the flow range of the dam break floods due to the limited availability of detailed information on observed water levels and flows. The modelled inundation extents were compared to the reported flood extents from the largest flood on record (peak flow of 1466 m$^3$/s at Wairoa Gorge flow gauging station in January 1986) as supplied by Tasman DC. This comparison suggested that the model would provide reasonable results of flood patterns.

The population at risk (PAR) was assessed based on the modelled inundation extent where the water depth exceeded 0.5 m (as per industry guidance at that time). The 2009 dam break assessment report presents a PAR of >100, and an assessed damage level of Major or Catastrophic. This assessment results in a High PIC as per the NZSOLD Guidelines 2000 used in the 2009 assessment and the updated NZSOLD Guidelines 2015.

It is noted that current industry practice (as per the NZSOLD Guidelines 2015) also includes consideration of the potential loss of life (PLL) in addition to PAR. PLL is often assessed in the context of inundation flow depth velocity parameters (typically a dv > 0.5 represents danger to life). Depth velocity parameters were also assessed as part of the 2009 dam break assessment and indicated a PLL of >10.

The 2009 dam break assessment has not been updated for the Stage 4 detailed design because of the High PIC adopted and noting that the dam type, size, location and reservoir remain unchanged. The 2009 dam break assessment was developed using established industry practices and is considered to be a comprehensive review consistent with the NZSOLD Guidelines 2015.

### 2.2 Summary of design criteria

A design criteria report was prepared for the project in October 2011 (T+T, 2011). Any criteria that have been considered subsequently necessary to be changed are identified in the current report. Key criteria are repeated in Table 2.1 below for ease of reference.

#### Table 2.1: Key design criteria

<table>
<thead>
<tr>
<th>Item</th>
<th>Value</th>
<th>Source</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Potential Impact Classification (PIC)</td>
<td>High</td>
<td>Assessment (T+T, 2009)</td>
<td>Based on dam break assessment.</td>
</tr>
<tr>
<td>Design life</td>
<td>Minimum of 50 years</td>
<td>Building Act</td>
<td>The Design Criteria Report did not specify a design life however did require 100 years for concrete durability.</td>
</tr>
<tr>
<td>Item</td>
<td>Value</td>
<td>Source</td>
<td>Notes</td>
</tr>
<tr>
<td>---------------------------------------------------------------------</td>
<td>---------------------</td>
<td>-----------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>Operational Basis Earthquake (OBE)</td>
<td>84\textsuperscript{th} percentile estimate of 150 year ARI design earthquake</td>
<td>NZSOLD DSG 2015</td>
<td>Based on PIC, with ground response based on site specific seismic assessment by GNS (2017).</td>
</tr>
<tr>
<td>Safety Evaluation Earthquake (SEE)</td>
<td>Mean 10,000 year ARI probabilistic estimate.</td>
<td>NZSOLD DSG 2015</td>
<td>Based on PIC, with ground response based on site specific seismic assessment by GNS (2017).</td>
</tr>
<tr>
<td>Aftershock earthquake</td>
<td>6.5M\textsubscript{w} event (one magnitude less than SEE)</td>
<td>NZSOLD DSG 2015</td>
<td>Based on site specific seismic assessment by GNS (2011 and 2017).</td>
</tr>
<tr>
<td>Seismic loading for non-critical structural elements</td>
<td>500 year ARI</td>
<td>NZSOLD DSG 2015 / NZ51170</td>
<td>Ground response based on a site specific seismic assessment.</td>
</tr>
<tr>
<td>Inflow Design Flood (IDF)</td>
<td>PMF</td>
<td>NZSOLD DSG 2015</td>
<td>Based on PIC.</td>
</tr>
<tr>
<td>Construction Diversion Flood (CDF)</td>
<td>Up to 1,000 year ARI (Refer to Stage 3 report)</td>
<td>NSW DSC</td>
<td>Refer discussion in this report.</td>
</tr>
<tr>
<td>Minimum freeboard above wind set up</td>
<td>0.5 m</td>
<td>-</td>
<td>Industry custom for rockfill dams.</td>
</tr>
<tr>
<td>and wave runup from significant wave caused by 100 year ARI design wind coincident with NTWL; or Tolerable mean overtopping discharge for same.</td>
<td>1\times10\textsuperscript{-6} m\textsuperscript{3}/s/m</td>
<td>Wallingford, (1999)</td>
<td>&quot;No damage to buildings&quot;.</td>
</tr>
<tr>
<td>Minimum freeboard above wind set up</td>
<td>0.5 m</td>
<td>-</td>
<td>Industry custom for rockfill dams.</td>
</tr>
<tr>
<td>and wave runup from significant wave caused by 10 year ARI design wind coincident with 200 year ARI design flood peak water level; or Tolerable mean overtopping discharge for same.</td>
<td>1\times10\textsuperscript{-6} m\textsuperscript{3}/s/m</td>
<td>Wallingford, (1999)</td>
<td>&quot;No damage to buildings&quot;.</td>
</tr>
<tr>
<td>Minimum freeboard above wind set up</td>
<td>0 m</td>
<td>-</td>
<td>Industry custom for rockfill dams.</td>
</tr>
<tr>
<td>and wave runup from significant wave caused by 10 year ARI design wind coincident with IDF peak water level; or Tolerable mean overtopping discharge for same.</td>
<td>0.002 m\textsuperscript{3}/s/m</td>
<td>Wallingford, (1999)</td>
<td>&quot;No damage to embankment seawalls&quot;.</td>
</tr>
</tbody>
</table>
2.2.1 Corrosion protection

Corrosion protection for exposed steel surfaces for the dam components have been specified in accordance with NZS 2312 and SNZ TS 3404. In general where embedded steel items are exposed to the elements or within the reservoir have been specified as Grade 316 Stainless steel.

The use of cathodic protection has been considered however this requires a knowledge of the water chemistry of the future reservoir. Given this will not be known until impoundment of the reservoir, we consider that a more pragmatic approach is to monitor performance (visually) of the steel structures and if early evidence of early corrosion occurs, then consider cathodic protection at that stage.

2.3 Failure modes effects and analysis

The NZSOLD Guidelines 2015 recommend that failure modes effects and analysis (FMEA) is completed during the design for new Medium or High PIC dams. A FMEA workshop was undertaken on 19 March 2018 as part of the Stage 4 detailed design. The outputs from this workshop are attached in Appendix D.

Twenty six (26) credible failure modes were identified and considered as part of the Waimea Dam detailed design. Of these credible failure modes only one was assessed as high risk and ten as moderate risk as summarised in Table 2.2 below. The high risk credible failure mode relates to debris blockage of the spillway and while the design includes a debris barrier, ongoing surveillance and maintenance of debris (especially during large floods) is essential.

Table 2.2: Assessed moderate and high risk credible failure modes (high risk in orange)

<table>
<thead>
<tr>
<th>Load case</th>
<th>Potential failure mode &amp; cause(s)</th>
<th>Surveillance requirements</th>
<th>Instrumentation requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood</td>
<td>Overtopping of the dam embankment due to flood and logs/debris blocking the spillway - assume no failure of wave wall but some erosion of downstream shoulder.</td>
<td>• Monitor condition of debris boom.</td>
<td>• Water level detector.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Manage catchment to reduce potential quantity of debris generated in flood events.</td>
<td>• Webcam/cameras at crest.</td>
</tr>
<tr>
<td>Normal</td>
<td>Major defect (i.e. construction design induced crack) in dam facing leading to sufficient flow through dam fill to cause internal erosion of embankment materials leading to dam failure.</td>
<td>• Continuous monitoring of toe seepage drains.</td>
<td>• Toe seepage drains flow monitoring system (electronic and telemetered).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Physical inspections of face when reservoir drawn down or by diver.</td>
<td>• Settlement markers on the parapet wall.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Monitoring during commissioning to check for issues on filling.</td>
<td></td>
</tr>
<tr>
<td>Normal</td>
<td>Severe leakage through fault in foundation rock leading to internal erosion of dam embankment materials leading to dam failure.</td>
<td>• Careful monitoring of toe drains during commissioning and operation.</td>
<td>• Toe seepage drains flow monitoring system (electronic and telemetered).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Inspection of downstream abutments and valley for new seepage (compare with pre construction)</td>
<td>• Consider having flow measuring in stream say 200 m downstream of dam and compare flows.</td>
</tr>
</tbody>
</table>

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Waimea Water

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<table>
<thead>
<tr>
<th>Load case</th>
<th>Potential failure mode &amp; cause(s)</th>
<th>Surveillance requirements</th>
<th>Instrumentation requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake</td>
<td>Deformation of downstream face during earthquake that leads to deformation of crest to below water level, overtopping, erosion of dam fill, and failure of dam.</td>
<td>inspections) during commissioning, and periodically as part of dam safety regime during operation.</td>
<td>at stream with flows through dam conduits / spillway.</td>
</tr>
<tr>
<td>Earthquake</td>
<td>Deformation of upstream face during earthquake that leads to cracking and displacement of face slab, deformation of crest to below water level, overtopping, erosion of dam fill, and failure of dam.</td>
<td>• Inspections and survey and analysis after earthquake events.</td>
<td>• Seismographs at dam crest and toe.</td>
</tr>
<tr>
<td>Earthquake</td>
<td>Overtopping of dam from earthquake generated seiche wave (landslide into reservoir and/or fault displacement within reservoir). Assume wave wall fails, subsequent erosion of dam fill and complete failure.</td>
<td>• Post earthquake inspections, survey and analysis of landslides.</td>
<td>• Settlement monitors.</td>
</tr>
<tr>
<td>Earthquake</td>
<td>Displacement of dam foundation or abutment rock during earthquake leading to major seepage path through foundation or abutment and uncontrolled release of reservoir</td>
<td>• Post earthquake inspections to include downstream abutments and valley for new or increased seepage (compare with normal operation inspections).</td>
<td>• Possible horizontal profilometer along crest.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Monitoring of flow in toe drains.</td>
<td>• Toe seepage drain monitoring system.</td>
</tr>
<tr>
<td>Flood</td>
<td>Overtopping of the dam embankment due to flood combined with failure of the rock slope above the left side of the spillway approach resulting in obstruction of the spillway entrance - assume no failure of wave wall.</td>
<td>• Inspections of the rock cutting above the spillway after floods and earthquakes.</td>
<td>• Lake water level detector.</td>
</tr>
<tr>
<td>Flood</td>
<td>Structural failure of spillway weir block from hydraulic loads leading to damaged spillway and uncontrolled release of some of the reservoir contents (limited by</td>
<td>• Regular and post flood inspections of weir block. Look for distress in concrete, upstream erosion, movement in joints etc.</td>
<td>• Consider instruments to monitor flow from ogee block underdrains.</td>
</tr>
<tr>
<td>Load case</td>
<td>Potential failure mode &amp; cause(s)</td>
<td>Surveillance requirements</td>
<td>Instrumentation requirements</td>
</tr>
<tr>
<td>-----------</td>
<td>----------------------------------</td>
<td>---------------------------</td>
<td>------------------------------</td>
</tr>
<tr>
<td></td>
<td>depth of erosion in underlying</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>rock)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Flood     | Structural failure of true right wall of spillway from hydraulic loads leading to damaged spillway extending into dam embankment fill beside spillway resulting enlarge scour hole in downstream toe of dam and potential instability of downstream face. | • Regular spillway inspections for cracking or movement at joints.  
• Monitor flow from spillway underdrains. | • Consider instruments to monitor flow from spillway underdrains. |
| Earthquake| Complete failure of concrete culvert at upstream end causing collapse of rockfill and settlement of dam and break in concrete face leading to loss of contents of reservoir through the collapsed culvert. | • Conduit inspections during commissioning, regular inspections during operation, and special inspection post earthquakes. | • Seismographs - dam crest and toe. Settlement monitors. |

The following three key recommendations were made from the FMEAs workshop:

1. Robust QA inspection and test plans to be specified prior to construction.
2. Surveillance manual to be completed in parallel with detailed design to ensure that instruments / facilities are included to facilitate all recommended monitoring and surveillance.
3. Design of instrumentation (e.g. seepage monitoring) and communication systems to enable critical data (toe drain flow) to be remotely monitored and include alert and alarm systems to provide immediate notification.

2.4 Safety in Design

A Safety in Design workshop was held with participants from the Constructor (FHTJV); the Temporary works designer (GHD) the designer (T+T) and the dam owner (Waimea Water). The workshop worked through the different stages of the project lifecycle and made recommendations.

The SiD workshop was effectively a follow up to the HAZOP workshop that occurred during Stage 3 Design. Whilst the name was different the purpose and outcomes were same (i.e. to consider the design of the project and improved aspects from a H&S perspective of users and constructors).

The developed register is included as an appendix to this report and should be referred to and further developed as necessary by the Constructor and Waimea Water as the project evolves through the project lifecycle.

Of note items that were considered and the design amended as a result include:

- Inclusion of harness anchor locations for future access;
- Inclusion of access platforms on the upstream dam face
- Ventilation was emphasised for access (having been identified in Stage 3)

Extensive workshopping has been had in respect of use of divers for inspections and maintenance of the dam and whether alternatives are practical. The attendees were of the opinion that the use of divers is usual for dam construction and maintenance. Commercial divers are trained and are experienced in performing the types of inspections and maintenance expected during the project lifecycle.
T+T recommended to Waimea Water to obtain advice on maintenance of the pipework and intake from a commercial diver company. Waimea Water (refer to meeting minutes 30 July 2018) chose not to pursue this recommendation but did request the inclusion of hand holds at joint locations to assist divers. The inclusion of these was considered however WSP concluded that without the specialist input from divers the positioning of any such handhold would not be appropriate but that diver’s clamps can be positioned to suit their needs.

Alternative arrangements were considered and could be proposed for the dam (e.g. an intake tower rather than inclined intakes); however this arrangement would likely prevent the project from continuing (due to capital cost) and will still require diver inspections and future maintenance (but just different activities and components to be maintained). Furthermore, other alternatives considered (e.g. the use of wheels on the intake rather than sliding connections) would introduce different risks and complexities.

2.5 Seismic design

2.5.1 Standards and references

For High PIC dams such as the Waimea Dam, the New Zealand Dam Safety Guidelines 2015 state the following seismic design standards:

- Operating basis earthquake (OBE) of at least 150 year ARI (noting could be up to 500 year ARI if adopted for improved serviceability performance).
- Safety evaluation earthquake (SEE) of the 84th percentile level of the controlling maximum earthquake (CME) (assessed via a deterministic method) or up to the mean 10,000 year ARI event (assessed via a probabilistic approach).
- Aftershock earthquake of at least one aftershock event at one magnitude lower than the CME.

Design standards for the appurtenant structures (e.g. bridges, spillway, parapet wall) are assessed individually based on the potential consequences noting the SLS (OBE) and ULS (SEE) events adopted as often as per NZS1170 for the selected Importance Level(s). A ULS event of 500/475 year ARI is stated in the Stage 3 design report for non-critical structural elements.

The peer reviewer (Ian Walsh, Opus) suggested during the Stage 3 review that an OBE standard of 500 year ARI may be more suitable. This suggestion has been considered, and based on the stability analysis undertaken, adoption of a lower probability event for the OBE such as the 500 year ARI does not appear to offer significant advantages as explained in the paragraphs below.

Selection of an OBE standard is predominantly an economic decision based on consideration of the consequences of repair works and service interruptions due to damage resulting from an OBE event. Waimea Water has emphasised the need to optimise the design and has communicated to T+T that it is willing to accept a higher operational cost if it reduces capital costs.

In the case of the Waimea Dam, the embankment and parapet wall can accommodate a wide range of deformation before this results in service interruption. Applying a higher OBE standard would therefore increase the parapet wall costs without a corresponding improvement in serviceability.

The plinth and concrete face are designed on the basis of precedent and therefore the performance of the primary seepage control components are not affected by the adopted OBE. The design of the diversion culvert and starter dams are driven by the SEE and increasing the OBE standard likewise does not appear to offer benefits in terms of serviceability.

The methodology adopted by GNS for deriving the horizontal seismic accelerations was peer reviewed by Engineering Geology Ltd acknowledging that WSP had excluded this review from their
scope. The methodology for derivation of vertical accelerations and for selecting site specific time histories (accelerograms) has also been reviewed by Engineering Geology Ltd.

### 2.5.2 Horizontal seismic design criteria summary

The adopted horizontal seismic design criteria for the Waimea Dam Stage 4 detailed design are summarised in Table 2.3 below. Vertical actions were determined from the horizontal actions, and the resulting load combinations are described in Section 2.5.3 below.

#### Table 2.3: Horizontal seismic design criteria for Stage 4 (based on 2017 GNS report)

<table>
<thead>
<tr>
<th>Design element</th>
<th>Horizontal peak ground acceleration</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load cases</td>
<td>OBE&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>SEE&lt;sup&gt;(2)&lt;/sup&gt;</td>
</tr>
<tr>
<td>Embankment</td>
<td>C(0) = 0.17g</td>
<td>C(0) = 0.64g</td>
</tr>
<tr>
<td>Parapet wall</td>
<td>C(0) = 0.55g</td>
<td>C(0) = 1.90g</td>
</tr>
<tr>
<td>Bridge and pier wall</td>
<td>C(T) = 0.60g</td>
<td>C(0.15) = 1.58g</td>
</tr>
<tr>
<td>Ogee weir</td>
<td>-</td>
<td>0.64g</td>
</tr>
<tr>
<td>Flip bucket</td>
<td>-</td>
<td>0.64g</td>
</tr>
<tr>
<td>Spillway walls</td>
<td>-</td>
<td>0.64g</td>
</tr>
<tr>
<td>Conduit</td>
<td>0.17g</td>
<td>0.64g</td>
</tr>
<tr>
<td>Winch landing area</td>
<td>-</td>
<td>C(0) =0.85g</td>
</tr>
<tr>
<td>Concrete face</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Starter dam</td>
<td>C(0) = 0.17g</td>
<td>C(0) = 0.64g</td>
</tr>
</tbody>
</table>

(1) OBE is 84<sup>th</sup> percentile unweighted 150 year ARI event and taken at T(0) for the dam components, and 84<sup>th</sup> percentile magnitude weighted 500 year ARI event for bridges and winch landing area.

(2) SEE is mean 10,000 year ARI event (unweighted for stability, weighted for structural design).

(3) Aftershock event is the deterministic 84<sup>th</sup> percentile 6.8M<sub>W</sub> spectrum. The alternative value in italics is as suggested by EGL based on the M6.5 spectra presented by GNS.

(4) Refer Section 16.

### 2.5.3 Seismic design loading cases

The design seismic load cases considered for the design of specific structures include:

1. Horizontal seismic actions only.
2. Horizontal seismic actions with 30% coincident vertical action.
3. Horizontal seismic actions with 50% coincident vertical action.
4. Vertical seismic actions with 30% coincident horizontal action.
For pseudostatic analyses, the vertical actions were calculated from the horizontal actions as per NZS1170.5 (2016, Amendment 1) which gives a ratio of $\text{PGA}_V = 0.9 \times \text{PGA}_H$. ICOLD Bulletin 148 “Selecting seismic parameters for large dams” states a commonly adopted ratio for determining vertical actions from horizontal actions of $\text{PGA}_V = 2/3 \times \text{PGA}_H$ (i.e. 0.66H). Vertical response spectra were derived as described in Section 2.5.4 below.

The adopted 30% coincident vertical/horizontal actions are based on the NZS1170.5 Commentary (2016, Amendment 1). The 50% coincident vertical action case is included as a sensitivity check for selected structures such as the parapet wall as per the Stage 3 design.

2.5.4 Response spectra

The adopted horizontal design response spectra are as per GNS (2017) “Seismic Hazard Assessment for the Proposed Waimea Dam” (attached in Appendix F below).

Vertical response spectra were derived from the horizontal spectra using the method of Bozorgnia & Campbell (2004) which accounts for rock type, controlling earthquake magnitude and proximity. We have reviewed a number of methods and have chosen this method as it was the most widely used and has been adopted in NZS1170.5 (2016, Amendment 1). The derived vertical spectra are summarised in Table 2.4 below.

The Bozorgnia & Campbell (2004) method estimates a ratio between vertical and horizontal spectra by considering the distance from the source and the site subsoil class. The vertical to horizontal ratio ($V/H$) for $T < 0.1$ sec was estimated as $V/H = 0.9$ because the Waimea Dam site is founded on rock and the nearest fault source is less than 20 km away.

At $T = 0.075$ secs, the vertical spectra is greater than the horizontal spectra. This is because the Bozorgnia & Campbell (2004) method uses a vertical design spectrum has both a flat and a decaying portion. The amplitude of the flat portion is equal to an estimate of the vertical spectral acceleration
at $T = 0.1$ sec. As such it does not show the same drop in spectra that the GNS (2017) horizontal spectra shows for periods less than $T = 0.1$ sec.

### 2.5.5 Ground motion time histories for dynamic analysis

Four ground motion time histories/accelerograms were selected for use in the dynamic time history analyses as part of the Stage 4 detailed design. The selected records are as per GNS (2011) seismic hazard assessment report, and were reviewed to confirm their suitability with the updated design response spectra from the GNS (2017) seismic hazard assessment report.

Industry practice is to identify and use at least three different accelerograms for dynamic time history analyses (also as per the NZSOLD Guidelines 2015). In this case four suitable accelerograms (El Centro, Abbar Iran, Izmit Turkey and Tabas Iran) have been identified (with some limitations as described below). Scaling factors for the adopted time histories were determined using the method outlined in NZS1170.5 (2016, Amendment 1).

Other accelerograms are now available (e.g. the recent Cook Strait 2013, Lake Grassmere 2013, and Kaikoura 2016 events) that may also be applicable. However, given the four records advised by GNS in 2011 remain valid, these have been used and there is no requirement to search for additional records.

The suitability of the selected accelerograms is limited to the following:

- The El Centro N90W and Izmit Turkey records are appropriate for the OBE design event. The Abbar Iran N68W record is only appropriate when considering periods of interest equal to 1.0 seconds or above. The El Centro S00E record is appropriate when considering periods of interest less than or equal to 0.5 seconds or greater than or equal to 1.5 seconds. The Abbar Iran S22W and Tabas Iran records are not appropriate for the OBE design event.
- The El Centro, Abbar Iran, Izmit Turkey and Tabas Iran records are appropriate for the SEE design event.
- The El Centro, Abbar Iran, Izmit Turkey and Tabas Iran N16W records appropriate for the aftershock design event. The Tabas Iran N74E record is not appropriate when considering periods of interest above 1.0 seconds.

The following assumptions were made during the review process for the selected time histories:

- The Waimea Dam site has an assumed average shear wave velocity over the top 30 metres ($V_s30$) equal to 800 m/s, as specified in the 2017 GNS report. In terms of NZS1170.5, this makes the site Class B – Rock.
- The four selected ground motions provide an appropriate reflection of magnitude, source-to-site distance, faulting mechanism and site conditions for an SEE event (i.e. 10,000 year ARI).
- The four selected motions cover a magnitude range of 7.0 to 7.4 M$_w$, source-to-site distances of 1 to 13 km, faulting mechanisms including strike-slip (El Centro, Abbar Iran and Izmit Turkey) and thrust (Tabas Iran) and site conditions consisting of rock.
- The 2017 GNS report provides deaggregation of seismic hazard for the 10,000 year ARI event (which corresponds to the SEE event), and suggests a deterministic hazard scenario be used to characterise aftershock motions (6.8 M$_w$ rupture on southern Waimea Fault segment). Deaggregation of the seismic hazard at 150 year ARI events is not discussed. The fundamental period ($T$) of the embankment dam is estimated to be between 0.4 seconds to 0.5 seconds. This assumption is based on an embankment height ($H$) of 53 m and a shear wave velocity ($V_s$) range estimate for the embankment of 420 m/s to 530 m/s. The fundamental period was estimated using $T = 4H/V_s$. 

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**Tonkin & Taylor Ltd**

**Waimea Dam - Stage 4 Detailed Design Report**

**Waimea Water**

**January 2019**

**Job No: 27425.100.vIssue 4**
3  Geology

3.1  Site geology

The Waimea Dam site geology is summarised briefly below. Further details of the site geology are covered in the Geotechnical Investigation Reports from feasibility (T+T, 2009) and Stage 3 (T+T, 2012/2014).

The Waimea Dam site geology consists of typically less than 1 m to 10 m thick weathered greywacke derived soils (solifluction deposits) over Rai formation bedrock. The Rai formation bedrock consists of moderately strong to strong jointed greywacke (well indurated fine sandstone) and argillite (well indurated siltstone and mudstone) that is commonly fissile. At the dam site, bedrock consists of a sequence of greywacke sandstone and argillite beds generally dipping at between 30 to 60° towards the north-west.

The identified rock mass defects are:

- **Bedding** is the prominent defect set and is generally spaced at 100 mm but may range from less than 10 mm to more than 1 m (uncommon). Bedding dips beneath the dam generally dips to the north-west at between 30 and 60°, noting near the toe of the dam the dip is locally subvertical.
- There are four major prominent and persistent joint sets in the rock. These sets are orientated north-east, south-west, east and south-east and dip between 40 and 70°. Joint spacing varies from 1 to 5 m.
- A number of sheared zones (SZ) were mapped or inferred in the vicinity of the dam site.

Landslide deposits, derived from bedrock or soil slide or flow are present but not widespread within the immediate vicinity of the dam site, or within the margins of the reservoir. A number of landslides that are of interest to the dam have been identified with the reservoir margin.

Alluvial gravels form a thin veneer over rock in the bed of the Lee River, underlie low (2 - 4 m above the river) terraces beside the river and are mapped in isolated terrace remnants on the valley sides at heights of up to 60 m above the river. The lower level terraces alluvial gravels are anticipated to be suitable for use in the dam.

3.2  Geological and geotechnical investigations

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

The investigations undertaken to inform the design are summarised below:

- Trench and trial pit excavations were carried out between February 2011 and August 2012 using 20 and 34 tonne excavators.
- Ripping trials in test pits. Moderately to highly weathered rock was found to be easily excavated using the 34 tonne digger.
- Seismic refraction surveys were carried out on the left abutment, SL1 was located down the line of the proposed spillway, SL2 along the left abutment plinth line.
- Drilling and in-situ rock permeability testing. Twelve drill holes were completed, seven vertical and five inclined drill holes into the abutments. Casagrande type piezometers were installed in drill holes.
- Field compaction trials. Two small trial rockfill embankments were constructed from excavated moderate to slightly weathered rock. The rock fill was compacted following each lift
by eight to ten passes of a 7.5 tonne vibrating smooth drum roller, and tested for in situ density, permeability, and plate bearing modulus.

- A reconnaissance level stability review of slopes around the reservoir margin. Twelve areas were identified in 2012 as having potential for slope instability following reservoir impoundment.

Further details of the 2012 investigations are provided in the Lee Valley Dam Detailed Design Geotechnical Investigation Report (T+T, 2012/updated 2014).

Additional (pre ECI phase) investigations consisting of additional drilling and seismic refraction survey were undertaken from November 2017 to late January 2018, and are covered in the February 2018 “Factual Geotechnical Report” (T+T, 2018). Further investigations were also undertaken as part of the ECI phase to improve the understanding and characterisation of the excavation profiles, anticipated rockfill quality, and compaction methodology. The ECI phase investigations are summarised in the May 2018 “Embankment Trial Testing Factual Geotechnical Report” (T+T, 2018).

The testing results from the site investigations were considered in the development of the excavation design profiles and the earthworks sections of the Specification (refer Specification attached in Appendix B below).
4 Flood hydrology

4.1 Design flood hydrographs

4.1.1 Summary

Design flood hydrographs were presented in the Engineering Feasibility Report (T+T, 2009) along with a discussion of how they were developed. Further details can be found in that report.

Design inflow hydrographs were developed to inform the spillway design and flood routing assessments. The design reservoir levels and spillway discharges during floods are based on these inflow hydrographs.

The design inflow hydrographs were developed using the flood volume frequency analysis method for the Wairoa River at Gorge/Irvines site record as translated to the Waimea Dam site. These estimates where then adjusted for potential future climate change. A calibrated rainfall runoff model (refer below) was used to derive the inflow hydrograph for the Probable Maximum Precipitation Probable Maximum Flood (PMP PMF) (which was adopted as the IDF).

The routed outflow hydrographs were derived using a spreadsheet model from the inflow hydrographs, reservoir storage elevation curve, and the design spillway rating curve.

A review of the updated flow records (i.e. additional records since 2009) and potential effects on the design flood hydrology was undertaken in February 2018 as part of the Stage 4 design. This review confirmed that the design flood hydrology developed during the Stage 3 design remains appropriate.

4.1.2 Hydrologic records

The feasibility study for the Waimea Community Dam was finalised in December 2009 (T+T Ref. 24727.100) and hydrological design parameters, for both flood passage design and operational simulations, relied on flow records up to April 2008.

Tasman District Council (TDC) installed a recording station on the Lee River upstream of the Waterfall Creek confluence (Site 57536, Lee at Waterfall Creek) and commenced flow monitoring on 20 April 2007. The catchment area above the recording station is 65.3 km², while the catchment area above the dam site, situated about 1.2 km downstream of the recorder, is 18.7% larger at 77.5 km².

Recorded flows in the Lee River were found to closely mirror the flows in the Wairoa River at the Irvines/Gorge recorder (Site 57521/Site 57502), which has a well-maintained continuous flow record from November 1957.

The correlation between the Lee River and Wairoa River flows provided confidence that the limited data available at the time of the feasibility study (one full year of Lee River flows from April 2007 to April 2008) was adequate to reasonably define the catchment water balance and assess the long term mean flow at the dam site.

The rainfall-runoff catchment model was developed and calibrated using event rainfall and flood data recorded at the Lee at Waterfall Creek site.

4.1.3 Rainfall runoff model

A catchment rainfall-runoff model was developed for the Waimea Dam site during the feasibility study stage (T+T, 2009). This spreadsheet model was calibrated using a number of recorded storm rainfall and flood hydrograph events for the Lee River and wider Wairoa River catchments. HEC-HMS (Hydrologic Modelling System developed by the US Army Corp of Engineers) was also used to check the spreadsheet model.
In the feasibility study, three storm events were used to calibrate the model: 23 May 2007, 22 January 2008 and 24 November 2008. The calibration results gave a reasonably good fit between the predicted flows and the actual flows recorded at the Lee above Waterfall Creek recorder (refer Figure 4.1 and Figure 4.2 below).

4.1.4 Stage 3 validation of calibrated rainfall runoff model

A significant flood event occurred on 19 January 2011, after the completion of the feasibility study in 2009. This flood event peaked at 208 m³/s, which is only about 12% lower than the largest calibration event used previously, viz. the 24 November 2008 flood which peaked at 236 m³/s and represented an approximately 14 year ARI event.

This flood event was selected as an appropriate independent validation event. The event rainfall was run through the original models and the parameters were adjusted to improve simulation of the hydrograph peaks for the both the events assessed in the original calibration process, and the recent validation event. The modelled peak flow for each event assessed in the calibration process is in...
close agreement with recorded peak flows, and the recorded hydrographs have very similar flood volumes to those produced by the model.

A large flood occurred in February 2016 and this damaged the upstream bridge at Waterfall Creek. The recorded flood flows from this event at the Lee River at Waterfall creek recorder site were not suitable for calibration as the recorded hydrograph was incomplete and the actual flood peak uncertain.

**4.1.5 Stage 4 validation check**

A flood validation check was undertaken as part of the Stage 4 design to determine if previous hydrological analyses, including correlations, remain valid given the seven to nine years of additional flow data now available compared with the feasibility and detailed design assessments in 2009 and 2011.

The following additional data was received from Tasman District Council in February 2018:

- Wairoa at Irivines (site no. 57521) from beginning of record to 15 Feb 2018.
- Lee at Waterfall Creek for entire record from 20 Apr 2007 to 15 Feb 2018. Gap of 1.57 years from 17 Feb 2016 to 13 Sep 2017 (coincident with February 2016 flood). Recent data from Sept 2017 onwards do not appear to be reliable.
- Rainfall for Lee at Waterfall Creek from 20 Apr 2007 to 18 Feb 2016.

The flow correlation between the Lee River at the dam site and the long duration record from the Wairoa River at Irivines (as used to develop a synthetic record for the dam site) was rechecked. Based on review of the flow data received to February 2018, the estimated long term mean flow ratios were confirmed (within 0.3%) along with the slight seasonal dependency. Therefore the flow correlation with the long term flow record was confirmed by the additional seven years of data (compared with one year of data at feasibility).

The confirmed flow correlation means that the flood frequency estimates based on the synthetic records are also unchanged from Stage 3.

**4.2 Climate change adjustment**

**4.2.1 Methodology**

The Engineering Feasibility Report (T+T, 2009) was peer reviewed by MWH. That review provided a suggestion that climate change should be considered for flood hydrology during the detailed design stage.

Climate change adjustments were developed for the design floods with a finite return period (excluding the PMF). The climate adjusted design floods are used for the design of permanent works (e.g. spillway) only.

The design floods for construction diversion have not been adjusted for climate change as river diversion works can be expected to be short duration and should be undertaken in the short-term. If there is a significant delay between design and scheme construction, there may need to be a subsequent review of flood hydrology for construction diversion. Construction diversion design (including related design flood) has been undertaken by Gutteridge Haskins Davey (GHD) for the Stage 4 Detailed Design phase and the diversion design is temporary works and thus is not covered specifically in this permanent works design report.

Potential future changes to temperature and the corresponding design rainfall depths were assessed by following the approach set out in the New Zealand Ministry for the Environment (MfE) publication "Tools for Estimating the Effects of Climate Change on Flood Flow: A Guidance Manual"
for Local Government in New Zealand, May 2010”. This method gives a 2.0 °C temperature change in the year 2090 which translates to a predicted 16% increase in rainfall depth.

The climate adjusted design flood hydrographs were computed via rainfall runoff modelling using the climate-adjusted design storms (derived from rainfall with the 16% increase for future climate change). These hydrographs are presented in the sections which follow.

### 4.2.2 Climate change adjusted inflow flood estimates

The design flood hydrographs, with and without climate change adjustment, are shown in Figure 4.3 and Figure 4.4. Table 4.1 summarises the peak inflows from both cases. Peak inflows increase by between 20% for the 10,000 year ARI event and 25% for the mean annual flood with climate adjustments. The climate change adjusted inflows have been used in the design of the dam spillway.

#### Table 4.1: Peak inflow at the dam site, with and without climate change adjustments

<table>
<thead>
<tr>
<th>Flood Return Period (ARI see note)</th>
<th>Peak Inflow (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No Climate Adjustment</td>
</tr>
<tr>
<td>2.33 years (mean annual flood)</td>
<td>168</td>
</tr>
<tr>
<td>5 years</td>
<td>216</td>
</tr>
<tr>
<td>10 years</td>
<td>255</td>
</tr>
<tr>
<td>20 years</td>
<td>292</td>
</tr>
<tr>
<td>50 years</td>
<td>339</td>
</tr>
<tr>
<td>100 years</td>
<td>375</td>
</tr>
<tr>
<td>200 years</td>
<td>412</td>
</tr>
<tr>
<td>1000 years</td>
<td>496</td>
</tr>
<tr>
<td>10,000 years</td>
<td>616</td>
</tr>
<tr>
<td>PMF</td>
<td>1094</td>
</tr>
</tbody>
</table>

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

#### Figure 4.3: Synthetic inflow hydrographs without climate change adjustments.
4.2.3 Uncertainties

Accurate prediction of future climate change is not possible and introduces uncertainty into potential increases in flood flows due to future climate change. A description of the uncertainties in the adopted climate change adjustment method is provided below.

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

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

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

In the absence of assigned relative likelihood of these scenarios, the 2010 MfE publication takes account of all six illustrative marker scenarios while focusing on a mid-range scenario A1B. The earlier more generalised MfE publication (2008) concentrates almost exclusively on the A1B scenario, providing predictions for this scenario only. For the same reason, the current assessment also focused on the A1B scenario. Further, the adopted temperature change is the average of the predictions from 12 general circulation models (GCM), which is regarded as the best estimate (MfE, 2008). Actual changes in climate in the future could vary from those predicted using the A1B scenario to an unknown extent.

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

4.3 Inflow design flood (IDF) hydrograph

The design for the Waimea Dam considers the safety of the structures up to and including the inflow design flood (IDF). For High PIC dams the IDF is the Probable Maximum Flood (PMF).

The PMF hydrographs were developed from estimated Probable Maximum Precipitation (PMP) depths for a range of storm durations using the HEC-HMS rainfall runoff model with the calibrated model parameters and initial loss set to zero to simulate a saturated catchment. The critical storm duration was identified as 48 hrs.

The Probable Maximum Precipitation (PMP) was determined during the feasibility study (T+T, 2009) for the catchment at the dam site using the 1995 NIWA method “A Guide to Maximum Precipitation in New Zealand” (Thompson and Tomlinson).

During the Stage 3 design, the PMP hyetographs were reviewed and adjusted for size of catchment and effective height of barrier impeding the flow of moisture into the catchment, factors which were not considered in the feasibility study (T+T, 2009). The resulting hydrograph peaks and volumes are very similar to the PMF hydrographs from the original analysis and therefore the feasibility inflow hydrograph was retained.

The IDF inflow (48 hr PMP PMF) and routing outflow hydrographs are presented in Figure 4.5 below.

![48 hr PMP PMF (IDF) design hydrographs](image)

*Figure 4.5: IDF inflow and routed outflow hydrographs.*
5 Wave environment

5.1 General

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

Waves can be generated by wind action across the reservoir, landslides into the reservoir, or seismic action and reservoir response. The following sections provide estimates of the wind speeds, wind generated wave run-up heights, landslide generated waves and seiche. The resulting wave heights were adopted to inform the selection of the dam crest height and design of relevant structures (e.g. parapet wall).

5.2 Design basis

5.2.1 Standards and references

The wave environment has been assessed in accordance with the following standards and references:


5.2.2 Design wind speeds

In the absence of site specific wind data, estimates of extreme wind speeds were obtained from the NZS1170 and converted to mean 1 hour wind speeds via empirical methods (USACE, 2011).

The maximum straight line fetch to the dam is 1,100 m from the south. The effective fetch is limited by the surrounding land and the irregular shoreline. An effective fetch of 700 m was calculated using the method developed by Saville, McClendon, & Cochran (1962). The most significant fetch is from the south and the mean 1 hour wind speeds for the 1 in 10 and 1 in 100 year return periods in are presented in Table 5.1.
Table 5.1: Mean one hour wind speeds

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Mean Wind Speed (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>30.6</td>
</tr>
<tr>
<td>100</td>
<td>36.9</td>
</tr>
</tbody>
</table>

5.2.3 Wind generated waves

The wave climate was assessed using theory developed by Young & Verhagen (1996). The extreme fully developed significant wave heights and hydrodynamics were calculated for the dam site assuming depth and fetch limited conditions and wind speeds as evaluated in Table 5.1.

The main processes that have potential to affect the dam face are wind generated waves and wave run-up. Rock armour is typically used to protect the face of an earth dam and can serve to absorb some of the wave energy. The Waimea Dam has been designed with a concrete facing (with a slope of 1V:1.5H), thus it will absorb less wave energy and result in more reflection and run-up than a rock armoured face.

Run-up is the vertical height of waves above the still water line. Run-up is calculated for different wave height probabilities (i.e. exceeded by x% of the incoming waves). Run-up was calculated using the methods developed by Delft Hydraulics and reported by van der Meer (1992) and incorporated in the method used by the USACE (2011). The method was developed from long crested wave data impinging head on to an impermeable slope.

The run-up is dependent on the significant wave height, wave properties and the slope of the dam. Significant wave heights (Hs), Peak Period (Tp) and wave run-up above still water level at the dam face for the significant wave height and the highest 2% and 0.1% of waves (Rs, R2% and R0.1%) are presented in Table 5.2. Figure 5.1 provides an illustration of the wave climate.

Table 5.2: Design wave climate at dam face

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Hs (m)</th>
<th>Tp (s)</th>
<th>Rs (m)</th>
<th>R2% (m)</th>
<th>R0.1% (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.28</td>
<td>2.07</td>
<td>0.38</td>
<td>0.56</td>
<td>0.72</td>
</tr>
<tr>
<td>100</td>
<td>0.34</td>
<td>2.26</td>
<td>0.46</td>
<td>0.68</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Figure 5.1: Wave climate (sourced from Eurotop).
The Waimea Dam wave run-up is not particularly high due to the relatively short fetch.

5.3 Wind induced wave overtopping

5.3.1 Methodology

It is common practice in CFRDs to include a parapet wall at the crest providing an economic means of increasing freeboard where the alternative is to provide additional embankment height. The Waimea Dam also includes a parapet wall (refer Section 16 for details) and its effect on wave run-up and preventing overtopping is described below.

Limited wave overtopping during extreme low probability events can be acceptable for CFRD’s provided this does not result in unacceptable consequences. No specific design guidance exists for tolerable wave overtopping flows over CFRD’s, noting that the downstream shoulder is comprised of selected large rockfill that is likely to withstand relatively significant flows without compromising the integrity of the dam.

International seawall design standards for crown walls do include tolerable overtopping design criteria based on the potential consequences for damage to the wall, damage to adjacent buildings and life safety to vehicles and pedestrians. These standards have been adopted for this overtopping assessment. It is noted that the dam crest road may require vehicle and/or person access during significant flood events and therefore limited wave overtopping would be tolerable.

The run-up equations referenced in Section 5.2 are typically not applicable for vertical walls (due to the surf familiarity parameter being high). Instead, empirical data is used to estimate the overtopping discharge per metre of wall in a given wave climate. This approach is applicable for the larger floods (e.g. IDF) where the waves run up the concrete face and impact the parapet wall (base of parapet wall is at 199.13 m RL).

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

Selection of the parapet wall height of 203.13 m RL (without precamber) included consideration of the assessed overtopping discharge (per metre of wall). The allowable overtopping rates are summarised below in Table 5.3 and are taken from Wallingford (1999) (from Simm, 1991) noting these were not developed for rockfill dams and are considered to be conservative overtopping allowances.

<table>
<thead>
<tr>
<th>Design event</th>
<th>Allowable overtopping mean discharge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind induced wave overtopping from significant wave height (H_s) caused by 100 year ARI design wind coincident with NTWL.</td>
<td>(1 \times 10^6 , m^3/s/m) (no damage threshold for buildings).</td>
</tr>
<tr>
<td>Wind induced wave overtopping from significant wave height (H_s) caused by 10 year ARI design wind coincident with 200 year ARI design flood peak water level.</td>
<td>(1 \times 10^6 , m^3/s/m) (no damage threshold for buildings).</td>
</tr>
<tr>
<td>Wind induced wave overtopping from significant wave height (H_s) caused by 10 year ARI design wind coincident with IDF peak water level.</td>
<td>(0.002 , m^3/s/m) (no damage threshold for embankment seawalls).</td>
</tr>
</tbody>
</table>
5.3.2 Overtopping results


The assessed freeboard and overtopping for the parapet wall (with crest at 203.13 m RL excluding precamber) due to wind induced wave run-up and wind set up are:

- At the NTWL (197.2 m RL) coincident with the wave run up induced by the design 100 year ARI wind ($R_{0.1\%}$ of 0.87 m as per Table 5.2), the runup does not reach the base of the parapet wall (at 199.13 m RL) and the remaining freeboard is approximately 5.0 m. Therefore no overtopping discharge is anticipated.

- At the 200 year ARI design flood water level (200.48 m RL) coincident with the design 10 year ARI wind generated wave climate, the remaining freeboard above the significant wave height ($H_s$ of 0.28 m as per Table 5.2) is approximately 2.37 m and no overtopping discharge is anticipated (Eurotop 2016 gives mean discharge of $9.5 \times 10^{-17} \text{m}^3/\text{s/m}$).

- At the IDF water level (202.53 m RL) coincident with the design 10 year ARI wind generated wave climate, the remaining freeboard above the significant wave height ($H_s$ of 0.28 m as per Table 5.2) is approximately 0.3 m and the mean overtopping discharge estimates range from $2.2 \times 10^{-5}$ to $0.0013 \text{m}^3/\text{s/m}$.

The assessed overtopping rates are less than (i.e. within) the design criteria.

The wave overtopping assessment summarised above also shows that wind induced waves are the critical design case for the parapet wall height selection. The landslide generated wave scenarios below consider the effects of single waves.

5.3.3 Freeboard

The NZSOLD Guidelines 2015 were published following the Stage 3 design (2012) and included further specific recommendations on minimum freeboard allowances for embankment dams. These recommendations (which are understood to be partially based on rules of thumb from the US) are to adopt the largest freeboard from the following:

- Wind set up and wave runup from highest 10% of wave caused by 100 year ARI design wind coincident with NTWL.

- At IDF peak water level the greater of:
  - 0.9 m, or
  - Wind set up and wave runup from highest 10% of wave caused by 10 year ARI design wind.

- Combinations of intermediate flood elevations coincident with wind set up and wave runup from highest 10% of waves.

Based on the wind induced wave runup and wave overtopping assessments outlined above, these freeboard criteria do not appear to result in greater freeboard requirements with the exception of the 0.9 m freeboard above the IDF requirement. We do not consider the application of a set value (based on a historic rule of thumb) for earth embankment dams to be appropriate for CFRD, especially where a risk based assessment has been undertaken to inform the design.
5.4 Landslide generated waves

5.4.1 General

Geological investigations (T+T, 2012) for the dam identified a number of potential slope instability or landslide features around the reservoir perimeter. These are shown on the reservoir landslide map presented in the Drawings (Dwg 27425-RES-101). Waves generated by a landslide into or within the reservoir may have the potential to overtop the dam crest and cause damage.

To understand and manage the risk associated with reservoir landslide events, the potential landslides identified were prioritised with guidance from methods described in ICOLD Bulletin 124 (2000). The two landslides considered to pose the most significant risk to the dam were then selected for hydrodynamic modelling to more accurately predict their impact on the dam.

Modelling assumptions, results and conclusions are presented below. For detailed assessment of the landslides refer to the 2012 Geotechnical investigations report (T+T, 2012/reissued 2014).

5.4.2 Modelling and assumptions

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

- The movement of each of the landslides into the reservoir was modelled using a time varying bathymetry (vertical displacement of the bed).
- No water was entering or exiting the reservoir during the wave’s propagation around the reservoir, to simplify the modelling process.
- A landslide velocity of 19 m/s was chosen based on information presented in “Review of natural terrain landslide debris-resisting barrier design - Geo Report No. 104” (Lo, D.O.K. (2000)), and noting the modelling approach does not use velocity as an input.

Two landslides were selected for detailed hydrodynamic modelling as follows:

- Scenario 1: landslide located at approximately Chainage 1400 m upstream of the dam (labelled as landslides 6 and 7 on Drawing 27425-RES-101, being the likely critical landslide to occur under the 200 year ARI flood conditions (triggered by extreme rainfall) with an approximate volume of 84,000 m$^3$.
- Scenario 2: landslide located at approximately Chainage 600 - 800 m upstream of the dam (labelled as landslide 3a on Drawing 27425-RES-101 being the critical landslide to occur under OBE and NTWL conditions (triggered by seismic event) with an approximate volume of 80,000 m$^3$.

5.4.3 Hydrodynamic modelling results

Figure 5.2 and Figure 5.3 show the modelled waves propagating through the reservoir to the dam face for Scenario 1 and 2 respectively. Concurrent modelling of Scenario 1 and 2 is not considered necessary given that Scenario 1 is related to extreme rainfall and Scenario 2 is related to seismic events.

Scenario 1:

A maximum water level of 201.91 m RL was calculated at the right abutment of the dam. This equates to a wave height of 1.43 m above the 200 year ARI design flood water level (200.48 m RL), and 1.22 m below the top of the parapet wall. The wave period is approximately 56 seconds.
Under this scenario some overtopping over the parapet wall is likely. The wave is expected to pass safely under the spillway upper bridge deck.

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

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

**Scenario 2:**

A maximum water level of 201.91 m RL was calculated at the right abutment of the dam. This equates to a wave height of 4.71 m above the NTWL, and 1.22 m below the top of the parapet wall. The wave period is approximately 8 seconds. Note that although the maximum water levels in the two scenarios are the same, this is a coincidence.

The resultant wave heights were also calculated using empirical methods described by Pugh and Hubert (ICOLD, 2000) as a check on the model results. The wave is expected to pass safely under the spillway upper bridge deck based on modelled water levels.

Under this scenario some splashing type overtopping over the parapet wall is likely.

Given that the wave period equates to approximately 8 seconds, forces from a dynamic impact wave on the parapet wall were calculated using the approach outlined in the USACE (1984) Shore Protection Manual Volume II. The calculated loading that may be applied to the wall as a result of the wave has been allowed for in the design of the parapet wall (refer Section 16).

The landslide displacement wave was also routed through the spillway (on the true left) to check the resulting outflow hydrograph. The modelled peak discharge via the spillway was 68 m$^3$/s (based on modelled overtopping water level of 199.3 m RL) which is approximately half the mean annual inflow flood. This means that the life safety risk is very low to itinerant persons who might be in the affected areas immediately downstream of the spillway and river channel.
The landslide displacement wave heights determined from the Mike21 modelling are not considered to be sensitive to landslide velocity given the modelling approach of changing the storage volume over a time step to approximate a landslide. The modelled wave results presented in this report are therefore influenced by the landslide volume rather than velocity. Upper bound estimates for landslide volumes have been used and therefore the analyses presented are conservative.

5.5 Reservoir seiching

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

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

The magnitude of the standing wave generated by an earthquake is dependent on two primary factors:
• The magnitude of energy that a potential earthquake can impart to the water body; dependent on the magnitude of the earthquake, the distance of the lake from the source and the ability of the ground to transmit the energy to the water.

• The natural frequency of oscillation of the lake, which is dependent on the geometry of the lake. The larger the lake the greater the difference between the natural frequency of the lake and the frequency of the earthquake. There is a reduction in the magnitude of the wave generated as the two frequencies diverge, all other factors being equal.

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

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

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

Each of these presume that energy is transmitted from the ground to the water by a notional sliding mass down slope (landslide) with little or no physical movement of the rest of the slope in general. In line with this approach, the assessment of seiching at the Waimea Dam is based on landslide generated wave modelling, discussed in Section 5.4 above.

Tilting of the reservoir body and/or the ground beneath as a result of earthquakes is a further mechanism that can generate seiches in lakes, as in the case of Hebgen Dam, USA.

The Waimea Dam reservoir has the active Waimea – Flaxmore transcurrent (strike-slip) fault 8.5 km to the north-west and the Wairau segment of the transcurrent Alpine fault 20 km to the south east. Both faults are expected to have horizontal to vertical movements approximately in the ratio of 1V to 10H. We would expect a vertical movement of no more than 1 metre per event.

Given the distance of the fault trace from the site (closest is 8.5 km) any potential resulting tilt is likely to be negligible (particularly given that the normal freeboard is relatively high at 5.6 m). No further assessment has been made of the effects of such small scale tilting on the reservoir.

T+T has not been able to identify any further methods (since Stage 3) for quantification of seiche wave heights. We have however undertaken further review of information and noted that Trevor Matuschka on behalf of NZSOLD (https://nzsold.org.nz/2017/08/31/dams-in-the-kaikoura-earthquake/) has published learnings from the 2016 Kaikoura Earthquake. This concluded that while a number of farm dams had flows over the spillway and a saddle dam (with 1.2 m freeboard above NTWL) was overtopped, but that no damage was recorded.
6 Design description

The Waimea Dam is a multipurpose on river storage concrete face rockfill dam (CFRD). The dam is 53 m high, 220 m long at the crest, and impounds approximately 13 Mm³ of storage at NTWL. The dam has an assigned High potential impact category (PIC) as per the NZSOLD Guidelines 2015.

The dam features the following components (note the dimensions are approximate and for descriptive purposes only. The Drawings should be referred to where minor inconsistencies occur):

- A 165 m long twin barrelled diversion culvert located on the true right bank of the river and running from the upstream of the starter dam to the downstream toe of the dam. The internal dimensions of the barrels are 4 m high by 2 m wide (refer Sections 7 and 9).
- A nominally 6.5 m high 65 m long mass concrete starter dam across the valley at the upstream toe of the dam to facilitate construction of the plinth (refer Section 10).
- A horizontal type reinforced concrete plinth founded on the starter dam in the river channel and on rock in the abutments. The plinth supports the concrete face slab at the toe of the dam. The plinth is anchored to the rock to resist uplift during grouting (refer Section 12). The plinth geometry and width vary to suit the location (refer Section 11).
- Upstream curtain and blanket grouting under the plinth, starter dam, spillway ogee weir and abutment parapet walls (refer Section 12).
- A 51.5 m high 220 m long zoned rockfill embankment (refer Sections 13 and 14) with a 6 m wide crest and 300 mm thick concrete facing on the upstream slope (refer Section 15). The dam is founded on rock (refer Section 8).
- 190 m of 4 m high and 4.55 m wide ‘L’ shaped reinforced concrete parapet wall on the dam crest, and partially buried under the dam crest road (refer Section 16).
- 29 m of 5.33 m to 4 m high and 6 m wide ‘U’ shaped crest ramp structure located on the true left abutment to provide a level transition from the upper bridge/mass concrete block to the dam crest (refer Section 16).
- An ungated ogee weir controlled spillway located on the true left abutment of the dam (refer Section 17). The spillway features:
  - An unlined inlet forebay cut into rock.
  - A 40 m wide 2.5 m high curved concrete ogee weir.
  - A 130 m long (plan) concrete lined chute from 40 m to 20 m wide with vertical and horizontal curves, inclined walls, and a maximum grade of 1V:2H.
  - A double curvature reinforced concrete flip bucket (vertical radius of 20 m).
  - A 35 m long (plan) concrete lined downstream impact area apron.
  - An 80 m long (plan) unlined plunge pool excavated into rock (10 m wide at the base with battered slopes).
- Two access bridges of composite steel beam and concrete deck design with 4.6 m wide decks and 26.4 m long total spans. The upper bridge provides access to the dam crest and has two spans with a central support pier (integrated with the ogee weir). The lower bridge has a single span and provides access to the outlet works, control building, fish pass inlet and toe berm area (refer Section 18).
- A 330 m long debris barrier in the reservoir upstream of the spillway inlet (refer Section 19).
- Two outlet pipelines (1,000 mm diameter steel pipes) with submerged intake screen structures, isolation valves, and discharge valves at the toe of the dam. The outlet works control the usual flow released from the dam (residual, irrigation, flushing) and enable dewatering of the reservoir. The pipes are inclined on the upstream face of the dam before
entering the diversion culverts and terminating at the end of the culverts. An access chamber with associated landings and stairs is located at the end of the culverts (refer Section 20).

• Access roads to the dam from existing forestry road, including permanent roads to be formed over the temporary construction haul roads (refer Section 21).

• A 170 m long 1.5 m wide open channel fish pass located on the true right bank of the dam. The fish pass channel is reinforced concrete with cast in rocks to provide upstream passage only for the target climbing fish species. The inlet to the fish pass features a low upstream barrier weir and inlet sump. Water is pumped from a gallery in the channel upstream of the barrier weir, via a pump in a wet well and pressure main up the downstream face of the dam to a flushing box located on the dam crest. The flushing box releases flow into the channel and the slotted outlet pipe on the upstream face of the dam (refer Section 22).

• Dam safety instrumentation including reservoir water level probes, a rain gauge, settlement instruments for the embankment and structures, a downstream dam seepage collection and monitoring system, two seismographs (crest and toe), a spillway underdrainage collection and monitoring system, and outlet works flow and pressure instruments. The electronic instruments record, store and transmit the collected data for controls and remote surveillance off site (refer Sections 23 and 24).

Further details on the specific design arrangements, and the design basis are summarised by component in the following sections.
7 River diversion

7.1 General

The Stage 4 diversion works for the Waimea Dam have been designed by GHD, with integration into the permanent works design remaining with T+T. While the diversion works are designed and reported separately (GHD, 2018), this key component of the temporary works for the dam interfaces completely with the permanent works and is therefore also summarised in this report.

T+T has not undertaken a formal peer review of the diversion works design prepared by GHD, and we strongly recommend to Waimea Water that peer review of this key component is undertaken in parallel with the permanent works design peer review.

The Waimea Dam is an on river storage dam and therefore temporary diversion of the Lee River is required as part of the dam construction. The proposed diversion strategy has been developed in conjunction with the permanent work design as part of previous design stages.

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

1. **Safety** - To adequately protect public safety during construction (in accordance with the NZSOLD Guidelines 2015 and other applicable international standards and precedents).
2. **Cost** - To balance the cost of providing diversion capacity and the probable costs of losses incurred if that capacity is exceeded.

Selection of the diversion arrangements and capacity to meet the safety objectives should be conservative and sets a minimum standard. Additional measures and design features may be incorporated above this minimum standard to manage construction risk and cost.

The Stage 3 specimen diversion strategy was developed by T+T in conjunction with the permanent works design. River diversion is a key aspect of the temporary works for the dam, and it is essential that the temporary and permanent works designs are coordinated and integrated. The selected diversion concept in Stage 3 was a large twin barrelled reinforced concrete culvert placed in the river bed with a mass concrete starter dam, upstream coffer dam, diversion walls and reinforced rockfill on the downstream toe of the main embankment. Alternative arrangements were extensively explored and considered as summarised in Section 7.3.2 below.

The Stage 3 design arrangements were intended to provide a workable basis for river diversion noting that refinement to specific details would likely be required to suit the specific methodology developed by the dam constructor (Contractor).

A rigorous assessment process was followed in developing the diversion works arrangements as presented in the Stage 3 documentation. Further details of the concept development and assessment process are covered in the Stage 3 design report (T+T, 2012/2014). The previous work on hydrological and population-at-risk assessments resulting from construction diversion floods is covered in the Stage 1 design report (T+T, September 2011).

The Stage 4 diversion strategy and design by GHD for the ECI Contractor (FHTJV) ultimately adopted a similar concept to the Stage 3 concept, albeit with some detail and sequencing changes, as discussed in Section 7.4 below.

7.2 Design criteria

The key design criteria adopted for the Stage 3 diversion works design was a diversion works capacity to safety pass up to and including the 1000 year ARI design flood (~500 m³/s). This diversion works standard was adopted based on the NSW Dams Safety Committee (Demonstration of Safety
for Dams – DSC2D Section 6.17) advice that they will accept a “flood capacity, during those phases of construction with public safety at risk, in the range of the AEP 1 in 500 to 1 in 1,000 flood discharge on the basis of world practice provided the risks are as low as reasonably practicable (ALARP)”.

The 1000 year ARI design diversion flood capacity is consistent with current international practice as outlined in ICOLD Bulletins 108A and 144. The NZSOLD Guidelines 2015 do not specify flood diversion standards and states that “There is no universally accepted standard for selecting an appropriate flood for the sizing of diversion works during construction and the choice is generally based on the dam site, the dam type, the construction cost and the consequences if the diversion capacity is exceeded”.

The NZSOLD Guidelines 2015 recommends a risk based approach to inform the selection and sizing of diversion works, and the following performance criteria for new dams:

- The risk of loss of life during construction, as far as practicable, should be no greater than that over the life of the dam.
- The design of any temporary works should include consideration of the PIC for any necessary cofferdams, and the design criteria for the coffer dams should be consistent with their PIC.

The arrangements selected during Stage 1 (T+T, 2011) and reconfirmed in Stage 3 (T+T, 2012) (refer Section 7.3 below) are consistent with the NZSOLD Guidelines 2015 and current international practice.

### 7.3 Summary of Stage 3 arrangements

#### 7.3.1 Description

The river diversion arrangements developed and presented in the Stage 3 documentation comprise of the following main components:

- A twin barrel concrete culvert in the river bed with two rectangular barrels, each 2.5 m wide by 4 m high and approximately 165 m long.
- A low height upstream coffer dam with a crest at 154.6 m RL and a diversion wall able to retain flood water to the same elevation. The crest level of 154.6 m RL was selected based on an assessment of tolerable overtopping frequency with regard to construction nuisance.
- A starter dam, comprising mass concrete, in the upstream shoulder of the permanent rockfill embankment with a crest level of 154.6 m RL.
- A main coffer dam with a crest at 173.4 m RL, 6 m wide, located in the downstream shoulder of the permanent rockfill embankment. This main coffer dam is described as the “downstream stage” and comprises of reinforced rockfill (also described as “meshing”) designed to enable large floods to flow over and through the embankment without failure.

A reinforced rockfill downstream stage is a commonly adopted arrangement for CFRD and is often considered more cost effective to reinforce the downstream stage to withstand overtopping rather than enlarging the culvert. Subject to appropriate detailing, the reinforcement improves the durability of the rockfill to withstand overtopping without significant damage. The extent of reinforcement was determined in Stage 3 as follows:

- Downstream meshing only continued up to an elevation of 173.4 m RL, at which level a flood between the 100 year ARI and 200 year ARI event can be passed entirely through the culverts without any overtopping. This capacity was deemed suitable for the dam height and the assessed potential incremental consequences.
- The Stage 3 arrangements adopted a central “quick rise” berm to enable an embankment crest level of 180.4 m RL to be built quicker than conventional staging (i.e. and therefore pass
the 1000 year ARI design flood entirely through the culverts without overtopping the embankment).

These levels give a reinforced rockfill downstream stage of approximate 26 m high and a quick rise berm of 7 m high. Stability of the downstream stage and quick rise berm under flow through and overtopping conditions was also assessed during Stage 3.

7.3.2 Alternatives considered

A range of different culvert sizes and corresponding starter dam, upstream coffer dam and downstream stage heights were considered as part of the Stage 3 assessment before the preferred arrangements were adopted.

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

Alternative materials were also considered for construction of the starter dam such as roller compacted concrete (RCC) or rockfill (with a conventional concrete plinth). These alternative material options were not progressed further in Stage 3 on the basis that:

- Removing the diversion wall from the rockfill starter dam after it had fulfilled its purpose would be impractical and it would be unacceptable to leave the diversion wall embedded in the rockfill starter dam permanently because of the potential for cracking of the concrete face due to differential settlement.
- The working area available for starter dam construction was considered too tight for the widths required by RCC plant and to fit in the two operations required (e.g. bulk RCC placement and placement of grout enriched RCC/conventional concrete on the upstream face).
- Materials testing for RCC mix design can often take in the order of half a year, which would introduce an additional constraint for programming.
- The cost of RCC was unlikely to be competitive compared with mass concrete.

7.3.3 Flood hydrology

The flood hydrology is described in detail in the Stage 3 design report and a summary is provided below.

Design synthetic inflow hydrographs were developed for the at the dam site without climate change (since negligible climate change will have occurred at the expected time of construction) as presented in the Stage 1 Design Report (T+T, September 2011) and reproduced in Figure 7.1 below. Based on the Stage 3 review of flood flows from a 52 year record, it was assessed that the flood estimates are not affected by seasonality on long term average and therefore annual events were adopted rather than deriving seasonal estimates.
7.3.4 Potential incremental consequences assessment

An assessment of the potential incremental consequences of a hypothetical dam break failure during construction was undertaken to inform the diversion works arrangements. This assessment is described in detail in the Stage 1 Design Report (T+T, September 2011).

The assessment used a hydraulic model of the Lee and Wairoa/Waimea River systems, which extended from the toe of the dam to the coast, to map inundation extents from hypothetical flood induced dam breaches during construction. Population at risk (PAR) was initially used to inform selection of coffer dam heights and culvert size. The estimation of the population at risk (PAR) was based on the inundation extents and depths and available aerial photography and census data.

Additional dam break analysis was also completed to determine the dam height at which a hypothetical breach would begin to have implications for public safety. The additional dam break analysis incorporated the proposed coffer dam height of 15.46 m RL and the finalised diversion culvert size and configuration (rather than the range of embankment heights and culvert sizes considered in the initial PAR focused assessment).

7.3.5 Design considerations for Stage 4

The following diversion works design considerations were noted in the Stage 3 design report (T+T, 2012) for consideration/design by the temporary works designer (subsequently confirmed as GHD):

- **Debris screening** at the culvert intake.
- **Temporary stoplogs** for upstream plugging of the conduits.
- **Upstream coffer dam** including review of the crest level with regard to construction nuisance in terms of frequency of overtopping. A preliminary upstream coffer dam and diversion wall height has been specified in this document based on analysis of overtopping frequency as described in the previous sections, and this has been checked to ensure there are no negative
implications for public safety. However, if the contractor determines that there is benefit in considering a higher coffer dam, then the contractor will need to assess public safety implications relating to coffer dam break.

- **Diversion wall** height and stability.
- **Starter dam height** the starter dam height also has implications for the number of times construction works are inundated and the cost associated with this construction nuisance. The starter dam height does not have implications for public safety since it comprises concrete and can be overtopped without unravelling. The permanent works design for the starter dam specifically considers stability during overtopping up to a 200 year ARI flood event (based on 2 m of overtopping depth) and this requires confirmation for the adopted Stage 4 diversion strategy.
- **Downstream stage reinforcing** including consideration of restricting the width of overtopping over the downstream stage by maintaining a channel with reinforced sides at a slightly lower lift height compared to the remainder of the embankment as rockfill placement progresses. T+T recommended specific details in the Stage 3 design report including use of heavier than standard mesh, and detailing regarding order in which downslope bars and mesh strands are laid on the rockfill relative to the transverse horizontal bars.
- **Quick rise berm** to be placed above the downstream stage reinforcing level. This must be designed to ensure it can retain flood water without failure.
- **Development of a Dam Safety Emergency Plan (DSEP)/ Construction Emergency Management Plan (CEAP)**, which details measures to protect the partly completed dam, and measures to warn the population at risk.

### 7.4 Summary of Stage 4 arrangements

#### 7.4.1 Description

The Stage 4 detailed design diversion works arrangements have been developed by GHD as reported in their draft “Waimea Community Dam Diversion Design Report” dated June 2018. These arrangements are a similar concept to the Stage 3 design by T+T noting the following changes/details:

- Upstream river channel profiling works and concrete apron in front of diversion culvert.
- Channel profiling on the true left bank and two coffer dams parallel to the diversion culvert and around the outlet area (Stage 1).
- Twin barrel culvert with geometry as per Stage 3 with upstream end located lightly north east and approximately 2 m above the Stage 3 location. We understand this arrangement was selected to facilitate construction by moving the culvert further into the abutment rock. The design level requires mass concrete backfill underneath the culvert. The outlet included an anchor slab detail. The diversion culvert forms part of the permanent works and is designed by T+T to the internal dimensions, levels and location provided by GHD and the ECI Contractor.
- An inclined bar trashrack fastened to the diversion culvert inlet.
- Upstream and downstream coffer dams at the diversion culvert inlet and outlet with reinforced concrete diversion walls (Stage 2).
- Starter dam height increased to 155.1 m RL to match the diversion culvert crown level. This decision was made in conjunction with T+T to facilitate a single level plinth at the starter dam. The starter dam forms part of the permanent works and is designed by T+T with inputs from GHD and the ECI Contractor.
• Revised reinforced rockfill detailing including increased crest level to 176.4 m RL, revised material zones and anchor bar details, and toe slab detail.

7.5 Key interfaces with permanent works

The diversion works arrangements described in the supplied GHD documentation have the following key interfaces:

• Temporary cut batters/profiles around the diversion culvert and the upstream and downstream channels interfacing with permanent cut profiles and the rockfill embankment around the diversion culvert. Placement and compaction of the rockfill around the diversion culvert requires sufficient space for access by the require machinery.
• Starter dam interface with diversion culvert (e.g. vertical waterstops at contraction joints) and embankment fill.
• Temporary works bypass line on true left on culvert (to enable diversion during culvert closure) interface with diversion culvert and starter dam (assumed to be cast into the starter dam mass concrete).
• Reinforced rockfill shoulder (downstream coffer dam) interfaces with Zones 3B, 3D, 3E and 4 (material compatibility).
• Reinforced rockfill concrete toe slab interface with seepage control bund and geomembrane connection point.
• Upstream and downstream reinforced concrete inlet/outlet aprons and retaining walls.

7.6 Construction considerations for the Contractor

7.6.1 Staging

Staging of the diversion works requires careful consideration by the Contractor and we recommend that the Contractor prepares a detailed methodology outlining the diversion sequencing and closure prior to construction commencing such that this can be adequately reviewed and amended if necessary.

A specific methodology for achieving closure of the diversion culverts is a key aspect of the construction sequencing.

7.6.2 Debris management

Significant quantities of felled timber are present on steep slopes in the catchment. The possibility that the timber could mobilise during a construction flood event and need to be passed down the downstream face of the dam without damaging the mesh was considered at Stage 3 and requires further consideration.

Logs could potentially be mobilised by the following:

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

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

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

Effective reservoir clearing and debris management is therefore an essential consideration for construction dam safety. The volume and frequency of mobilised floating debris could be significantly reduced by effective management procedures. For example, standing trees and felled
logs that will be inundated by the final reservoir are expected to be removed for water quality purposes as part of the reservoir clearing works. The slopes immediately surrounding the reservoir should also be inspected for potential zones of instability and subsequent removal of logs in these zones.

7.6.3 Diversion risks

While management of construction risk remains the Contractor’s responsibility, we note some of the risks as follows for consideration by the Contractor:

- Debris fouling of culvert resulting in overtopping of embankment, excessive scour and potential loss of embankment.
- Debris damage to the reinforced rockfill downstream stage resulting in excessive scour to this material and potential loss of embankment.
- Construction of the diversion culvert in the river bed and associated control of water and construction safety.
- Temporary slope stability.
- Height of diversion wall and upstream coffer dam to be limited to avoid significant increase in hazard downstream.
- Staging of diversion with embankment construction as height of embankment increases to protect works.

A Failure Modes and Effects Analysis workshop specifically for the temporary works may be beneficial.
8 Foundation excavation and treatment

8.1 General

The excavations have been designed based on geological mapping of exposed bedrock at specific locations, interpretation from geophysical surveys, 14 drillholes and testing of samples from drillholes and exposures. Subsurface conditions away from test locations are inferred. Specific permanent slope protection details have been provided but will require onsite confirmation. A risk assessment document has been provided to the Contractor (delivered by hand to Wayne Newton, FHTJV on 17 May 2018) describing possible specific slope issues and treatments that may be required during Construction. The Contractor should include and allow for these issues in their risk contingency.

Subsurface conditions have been inferred based on the above investigations. It is expected that there will be some variability between actual and inferred conditions. We therefore recommend that the Contractor make risk allowance in their costing for such variability.

The existing ground levels are based on LiDAR. The T+T Geotechnical Factual Report (February 2018) included survey verification that identified variability exists between topographic survey and the LiDAR surface in steep areas covered by vegetation. We therefore recommend the Contractor makes risk allowance for quantities due to base survey error. Furthermore we recommend that the dam footprint is resurveyed once vegetation clearance has been undertaken to identify and quantity differences prior to excavation commencing.

The excavation levels have been set based on required rock quality for the foundations of structures, which are different for the spillway, plinth, and dam embankment structures. The spillway foundation was set for unweathered to slightly weathered rock. The plinth foundation was set for slightly weathered to moderately weathered rock depending on location transitioning up to crest level at the abutments. The embankment foundation level is set to moderately to highly weathered rock. All soil and loose highly weathered rock shall be stripped to dump stockpile(s).

The excavation profile model supplied allows for stripping of overburden and unsuitable weathered rock from the existing ground surface model to the anticipated suitable rock profile within the dam footprint. The actual extent of stripping/foundation excavation required underneath the dam, starter dam, and plinth to achieve a suitable foundation is highly uncertain and subject to confirmation during construction. We have recommended that the Contractor allows a suitable contingency for additional excavation.

Further general considerations follow:

- It is anticipated that rock batters on the right abutment will require permanent slope instability mitigation.
- The excavation depths, batters and slope protection requirements require confirmation during construction to suit the encountered rock quality.
- Additional rock anchors and or dental concrete may be required and will need specific assessment and design during construction.

8.2 Embankment foundation

Site investigations (T+T, 2012) indicate that moderately weathered to fresh rock (e.g. Class 1, 2 and 3 rock) are all likely to form a suitable general foundation for the dam embankment. This will require removal of soils that are locally up to 12 m deep, consisting of slope derived silt and sand and alluvial gravel that overlie bedrock on the left abutment. On the right abutment, scree and colluvium that is generally less than 2 m thick, but is locally up to 5 m thick, will need to be removed.
Gravel may be left in place but any significant sand deposits need to be removed. While sand is unlikely to be a problem, should it be encountered, it needs to be checked for susceptibility to liquefaction and stability under seismic loading conditions and given the small quantities expected to be involved, removal is preferable to reduce uncertainty in the foundation condition and performance.

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

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

8.3 Plinth foundation

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

- Excavation of soft material from joint and shears to a depth at least equal to twice the width (i.e. H = 2D).
- Clean-up with air and high pressure water.
- Backfilling of cracks, joints, cavities etc. with dental concrete or mortar.

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

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

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

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

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

Site investigation to date has shown little in the way of major foundation defects with only one significant sheared zone (SZ8) located on the right abutment in drill hole 10. Where the plinth does not provide an adequate hydraulic gradient for foundation defects, the clean-up is extended further downstream and a reinforced shotcrete slab or slab extension is provided.
If the defect infill or sheared zone material is erodible, it will be excavated and backfilled with concrete underneath the plinth and downstream. A reverse filter may also be provided over the shotcrete in case the shotcrete cracks where directed on site by the Foundation Committee (as below). Where required, the reverse filter may also be extended for a distance downstream of the shotcrete to allow seepage to emerge in a controlled manner and prevent the migration of fines into the embankment rockfill.

Foundation treatment will include curtain and blanket grouting to reduce seepage caused by foundation disturbance during excavation, and reduce seepage along defects. This is discussed further in Section 12.

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

The foundation committee will be confirmed prior to construction but is likely to include:

- Mark Foley (Engineering Geologist).
- Philippe Cazalis de Fondouce or Eric Guilleminot (CFRD Specialist).
- Ian Walsh (Peer Reviewer).
- Engineer’s Representative.

### 8.4 Starter Dam

The starter dam is to be founded on a prepared, hard, non-erodible, groutable fresh to slightly weathered rock. The foundation level shown on the Drawings is based on the geotechnical investigations undertaken to date and is indicative only. Further excavation may be necessary to achieve a suitable rock foundation for the starter dam. Where localised defects are identified, these should be treated as per the plinth. Careful planning and management of the river diversion control and dewatering will be essential during excavation for the starter dam foundation.

### 8.5 Diversion culvert

The foundation for the diversion culvert shall be on prepared, hard, non-erodible, fresh to slightly weathered rock. Site concrete or mass concrete backfill may be required where significant overexcavation and/or overbreak occurs below the design excavation profile under the culvert.

The temporary excavation profile around the culvert shall include a minimum bench width of 1.5 m either side of each wall to facilitate placement and compaction of the embankment rockfill. We understand FHTV have adopted a temporary cut batter of 0.8H:1V as per the Stage 3 design. The Contractor will need to make provision for temporary support of the cut face to protect worker and works area safety.

### 8.6 Spillway

The spillway cut batters will require localised mitigation to suit the encountered rock quality. The provision of roads and benches above the chute wall will provide a degree of mitigation to potential instability arising on the upslope batters.

We highlight temporary slope risks associated with the Contractor’s crane pads that the Contractor should address from a health and safety perspective.

The true right spillway cut batter results in a relatively steep and thin area of rock to support the spillway wall between Chainage 1070 and 1100 m. There is a low probability of rock defects being
present that could result in wedge failures and loss of support to the spillway chute wall. Failure of the spillway chute wall could result in flow on the dam (i.e. an extreme consequence).

In addition to carefully planned and constructed excavations in this area, specific treatment measures are also specified including vertical dowel bars in the spillway chute, and provisional inclined dowel bars in the rock face. These dowel bars are nominally 6 m long at 2 m centres, as adjusted to suit the mapped cut surfaces (as mapped from the spillway cut). The spillway chute wall in this area has also been designed a free cantilever and does not rely on the rock batter for support. It is desirable however for the rock profile to be consistent with the spillway wall profile to offer support. Provisional mass concrete backfill is also allowed for in the design.
9 Diversion culvert

9.1 General

The diversion culvert is a key feature of the temporary construction diversion works that is also integrated into the permanent works. The diversion culvert is located at the base of the dam and is buried under the embankment. The outlet works are housed in the diversion culvert which acts as a conduit following culvert closure.

As the diversion culvert is to be a permanent feature of the dam, it has been designed by T+T. The internal geometry, location and levels for the culvert have been set by FHTJV as these aspects are integral to the temporary works diversion strategy.

9.2 Design basis

9.2.1 Standards and references

The following standards and references have been used for the design of the diversion culvert:

- NZS3101 “Concrete structures”.

9.2.2 Geometry

The diversion culvert geometry was considered in detail as part of the Stage 3 design. A twin barrelled culvert with internal barrel dimensions of 4 m height and 2.5 m width, providing a total full flow area of 20 m². The internal culvert dimensions are set to suit the temporary works diversion flood routing requirements. These dimensions are as per the Stage 3 design and as confirmed by FHTJV and GHD for the Stage 4 temporary works.

Of note that there would be operational advantages in having larger internal dimensions for space requirements; however Waimea Water has emphasised that it would prefer to sacrifice some operational advantages if capital costs can be kept to a minimum. We therefore note that the culvert dimensions may result in greater costs in the future if significant repairs/maintenance are required to the outlet works in the conduit.

9.3 Description

The final adopted diversion culvert is a 166 m long rectangular shaped twin barrelled reinforced concrete structure. The internal dimensions of each barrel are 4 m high and 2.5 m wide and provide a total full flow area of 20 m².

The diversion culvert is included to route the river flows up to and including the design construction diversion flood (1,000 year ARI flood, refer Section 7). Twin barrels are included to facilitate closure, where by flow can be diverted into one barrel while the outlet pipe work is installed in the adjacent barrel (refer Section 7).

The culvert is located on the true right bank of the Lee River and is excavated into sound moderately to slightly weathered rock. The temporary excavation profile around the culvert includes a minimum...
bench width of 1.5 m either side of each wall and a cut batter of 0.8H:1V. The slopes and bench width may be varied if preferred by the Contractor. The Contractor will need to make provision for temporary support of the cut face to protect work and works area safety. It is important that the excavation profile around the culvert facilitates placement and compaction of the surrounding rockfill and gravel fill (i.e. Zone 4 on the downstream end) to reduce potential deformations in this area under the imposed loads.

The upstream end of the culvert extends just beyond the starter dam and includes a concrete apron, flared entry walls and a curved roof to improve the hydraulic efficiency (e.g. smoother transition with lower entry losses). Knockout panels are included in the roof for the outlet works pipework (lobsterback bend), and the sidewall for the provisional temporary diversion pipe.

The upstream and downstream inverts of the culvert have been set by FHTJV at 150.3 m RL and 148.4 m RL respectively. This gives an internal grade of -1.75% for the 166 m long culvert.

The wall, roof and floor thicknesses were determined based on detailed structural analyses (refer Section 9.5 below), and are different for the upstream and downstream sections. The upstream section is thicker due to the higher stresses imposed by the embankment and reservoir.

The upstream section has 800 thick external walls, roof and floor. For the downstream section of the culvert, the concrete thickness was reduced slightly to 650 mm for the external walls, roof and floor due to the lower imposed loads. The internal dividing wall thickness of 450 mm is the same for both culvert sections.

The culvert is heavily reinforced due to the imposed loads. Reinforcement has been designed with consideration of the structural loads, temperature and shrinkage in accordance with NZS3101.

Alternative culvert shapes were considered (including a curved arch roof) for structural performance reasons, but were not assessed to be beneficial over the rectangular shape selected.

Vertical and horizontal construction joints are necessary at regular intervals to suit the Contractor’s construction methodology. The construction joints shall be Type B construction joints as per NZS3109, with continuous reinforcement. Swellable waterstop strip is specified at each joint to control seepage into the culvert. This is an important consideration for operation of the outlet works (including access through the culverts to the upstream valves).

The downstream end of the diversion culvert extend beyond the toe of the reinforced rockfill and features an access chamber for the outlet works. The chamber wall is 8.2 m high with the top of the walls set at the IDF tailwater elevation of 156.6 m RL. The chamber walls are designed for hydrostatic, static and seismic loads (including loads from the adjacent toe berm rockfill).

The access arrangements and outlet works to be housed within the diversion culvert are described in Section 20.

Penetrations for the hydro will be required to be cut through the conduit walls if the mini-hydro were to be constructed at a later dated.

9.4 Geotechnical analysis

9.4.1 Methodology

Estimates of the vertical and horizontal loads applied to the diversion culvert from the rockfill embankment were estimated as part of the Stage 3 design with two models implemented in the finite difference package FLAC/2D. The two models represent a cross section along the dam crest, and a cross section at the half embankment height on the downstream shoulder.
As part of the Stage 4 design, the loads were reviewed using Plaxis with the final culvert geometry and the revised seismic loads (as per GNS, 2017). The Plaxis derived loads were slightly lower than those considered in Stage 3 and therefore the Stage 3 design loads were retained for the culvert design.

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

The end chamber has been designed as a series of two way bending retaining walls. The rockfill loads applied to the chamber walls were determined in accordance standard earth retaining wall design methods (e.g. as per Craigs Soil Mechanics with seismic loads as per MBIE Module 6 Seismic Design of Retaining Walls.)

9.4.2 Static rockfill loads

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

- Shear strength function: Mohr Coulomb with Internal friction angle ($\phi$) = 40 to 47 degrees and apparent cohesion ($c$) = 0 kPa.
- Stiffness ($E$) = 13 – 40 MPa.
- Rockfill density ($\gamma$) =2250 - 2500 kg/m$^3$.
- Interface coefficient = 0.45.

The mohr coulomb constitutive model was used to represent the rockfill in layers. Mohr coulomb strengths were selected to represent the design strength obtained from Barton & Kjaernsli (1981) (refer Section 14 for rockfill strength parameters).

The interface friction between the rockfill and the external concrete faces affects the load transfer from the embankment.

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

9.4.3 Seismic deformation

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

For the analyses, the stiffness parameters controlling the behaviour of the rockfill represent estimates of the small strain stiffness. The small strain stiffness parameters have been estimated using the method of Makdisi & Seed (1978) and a $k_{2,\text{max}}$ of 120. This yields a maximum small strain stiffness of 200 MPa. The adopted maximum small strain stiffness has been degraded to account for the estimated strain level within the embankment based on the degradation curve presented by Makdisi & Seed (1978).
Pseudostatic analysis has been used to assess the potential embankment displacements at the location of the top and bottom of the culvert box. The adopted peak ground accelerations for the OBE and SEE events are as per the design criteria (Table 2.1).

The horizontal acceleration has been assumed to act parallel to the dam crest (the worst case scenario for culvert racking). Owing to the asymmetry of the cross section and the off centre location of the culvert, analyses consider horizontal acceleration applied from both directions.

Table 9.1 below summarise the modelling results for displacements and racking. The Stage 4 modelling results were consistent with the Stage 3 results. These results have been adopted for the structural design of the culvert, described in Section 9.5 below.

Table 9.1: Estimated culvert racking under the maximum embankment height (upstream section)

<table>
<thead>
<tr>
<th>Seismic load case</th>
<th>Maximum estimated horizontal displacement at culvert base (mm)</th>
<th>Maximum estimated horizontal displacement at culvert top (mm)</th>
<th>Maximum estimated racking (top of culvert relative to base) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OBE (0.17g)</td>
<td>2</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>SEE (0.64g)</td>
<td>14</td>
<td>57</td>
<td>43</td>
</tr>
</tbody>
</table>

9.5 Structural analysis and design

The results of the geotechnical analyses described in Section 9.4 have been used as inputs into a linear elastic model using the software package Microstran V9.0. The model analysed is a simple 'stick' model with the following key assumptions:

- The concrete walls and slabs have been analysed using estimates of their cracked section properties. The cracked section properties (Table 9.2) have been estimated using guidance in the commentary of NZS 3101.
- The conduit base slab supports have been modelled as a series of springs. The spring stiffnesses have been derived using a subgrade reaction modulus of rock of 240 MN/m³ (derived from fresh unweathered rock). Sensitivity analyses have also been carried out if the conduit is founded on slightly weathered rock. The analysis assuming fresh rock foundation results in conservative forces and moments.
- The wall/slab joints (joint block regions) have been modelled using rigid off-sets.
- The concrete structure has been designed with a nominal ductility (µ = 1.25).
- Maximum bending moments at corners have been re-distributed (reduced) by up to the code allowance of 30%. The mid-span bending moments have been increased by an equal amount. Shear forces have not been re-distributed because they are a brittle failure mode.
- Hydrostatic water pressures have been allowed for in the downstream culvert section to account for the seepage collection bund.
- All wall/slab joints are modelled and designed to be continuous (i.e. carry moment).
- For the seismic design case the maximum estimated rack displacement has been applied to the Microstran model as a horizontal displacement at the top of the conduit.

The resulting bending moments are similar to those derived during the feasibility studies (T+T, 2009) and the Stage 3 design. An envelope for un-redistributed bending moments is shown in Figure 9.1.

The Stage 3 Design included a load case where full hydrostatic water pressure was added in addition to dam rockfill loads. The Stage 4 design review identified that this load case was overly conservative.
because the maximum rockfill load that the culvert was exposed to already included for reservoir loads.

Steel reinforcing has been determined using spreadsheet based calculations at the ultimate limit state. Seismic combinations have been designed using over strength factors (refer to Table 9.2 below for a list of design parameters). Sample calculations have been checked using the design software package spColumn v4.60. The derived longitudinal reinforcing requirements have been confirmed using both approaches.

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

The culvert has not been designed as a water retaining structure (i.e. crack widths have not been assessed for criteria in NZS3106). This is because under normal operating conditions most of the conduit will not be retaining water as it is behind the concrete face. The downstream section of the culvert will retain up to 2 m of water height due to the seepage collection system (which maintains the water surface at least to 150.5 m RL, refer Section 24). The adopted design approach is considered appropriate given the relatively low water pressures, the heavily reinforced structure, and the presence of sump pumps at the downstream end of the chamber.

It is expected that given the number of joints in the conduits that there may be some seepage into the conduit during operation. It is usual for tunnel projects (acknowledging that the conduit is a

Figure 9.1: Example bending moment diagram for the maximum height embankment (Plaxis model review Stage 4 output).
buried structure rather than a tunnel) for leakage to be acceptable depending on its function (Haack A 1991). The sump pumps will consider Haack tightness rating 5 (Figure 9.2) for sizing of the sump pump. Given the M&E items will be IP68 rated there is no dam safety issue if leakage occurs.

In the event that greater leakage occurs the conduit and any joints or cracks may need epoxy or grout repair which is common for underground structures.

The use of temporary pumps can also be used in the case that larger inflows occur. Of note the downstream end of the conduit will be exposed to rainfall and water will therefore periodically fill until the sump pumps have cleared the rainfall.

Table 7: Summary of Haack Rating Tunnelling Watertightness – Permissible Daily Inflows

<table>
<thead>
<tr>
<th>Haack Tightness Rating</th>
<th>Moisture Characteristics</th>
<th>Intended Use</th>
<th>Watertightness Descriptive Definition</th>
<th>Permissible Daily Leakage Quantity (litres/m² for a reference length of 100 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Completely dry</td>
<td>Storerooms and workrooms, restrooms.</td>
<td>The wall of the tunnel lining must be so tight, that no moist patches are detectable on the inside.</td>
<td>0.01</td>
</tr>
<tr>
<td>2</td>
<td>Substantially dry</td>
<td>Frost-endangered sections of traffic tunnels; station tunnels.</td>
<td>The wall of the tunnel lining must be so tight, that only slight, isolated patches of moisture can be detected on the inside (observed as discoloration). After touching such slightly moist patches with a dry hand, no traces of water should be detectable on it. If a piece of blotting paper or newspaper is placed upon a patch, it must on no account become discoloured as a result of moisture absorption.</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>Capillary wetting</td>
<td>Route sections of traffic tunnels for which Tightness 2 is not required</td>
<td>The patches of moisture reveal that the wall all of the lining must be so tight that only isolated, locally restricted patches of moisture occur. Restricted patches of moisture reveal that the wall is wet, leading to a discoloration of a piece of blotting paper or newspaper if placed upon it – but no trickling water is evident.</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>Weak trickling water</td>
<td>Utility tunnels</td>
<td>Trickling water is permitted at isolated spots and locally.</td>
<td>0.2</td>
</tr>
<tr>
<td>5</td>
<td>Trickling water</td>
<td>Sewage tunnels</td>
<td></td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 9.2: Concrete design properties

<table>
<thead>
<tr>
<th>Description</th>
<th>Adopted property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compressive strength (28 days) $f'_c$</td>
<td>40 MPa</td>
</tr>
<tr>
<td>Longitudinal reinforcing yield strength $f_y$</td>
<td>500 MPa</td>
</tr>
<tr>
<td>Concrete cover</td>
<td>40 mm internal, 50 mm external (assumes shuttered formwork or the use of site concrete)</td>
</tr>
<tr>
<td>Concrete ductility (nominal) $\mu$</td>
<td>1.25</td>
</tr>
<tr>
<td>Modelled wall stiffness</td>
<td>$I_w = 0.25I_g$</td>
</tr>
<tr>
<td>Modelled slab stiffness</td>
<td>$I_s = 0.4I_g$</td>
</tr>
<tr>
<td>Strength reduction factor ($\phi$)</td>
<td>0.75 Shear</td>
</tr>
<tr>
<td></td>
<td>0.85 Bending</td>
</tr>
</tbody>
</table>

Figure 9.2: Haack Watertightness Rating (Haack, 1991).
10 Starter dam

10.1 General

The starter dam is a nominal 6 m high mass concrete gravity dam located at the upstream toe of the dam. The starter dam is included to facilitate construction of the plinth above the river level and to simplify the concrete face, plinth, and diversion culvert arrangements. The starter dam is to be founded on slightly weathered to unweathered rock in the base of the river channel.

10.2 Design basis

10.2.1 Standards and references

The following standards and references have been used for the starter dam design:

- NZSOLD Guidelines 2015.
- NZS3101 “Concrete structures”.

10.2.2 Geotechnical loads

The geotechnical rockfill loads (including surcharge effect of water pressure on the embankment concrete face) and load resultants on the downstream face of the dam have been determined using PLAXIS and Quake/W software for the design average rockfill parameters from the embankment (refer Section 14). Seismic rockfill loads were determined for the SEE and OBE design cases assuming the starter dam does not move.

10.2.3 Hydrostatic and seismic loads

Hydrostatic, hydrodynamic and seismic inertial loads have been developed in accordance with USACE (1995) Gravity Dams. The seismic inertial loads are based on the design seismic peak ground accelerations (Refer Section 2) at the dam foundation for the OBE and SEE with coincident horizontal and vertical actions considered.

10.2.4 Stability criteria

The stability criteria for the starter dam are as summarised in the NZSOLD Guidelines 2015. Stability was assessed considering cracked base analysis as per USACE (1995) Gravity Dams.

10.3 Description

The starter dam is a mass concrete gravity structure up to approximately 6.5 m high and 60 m long, and has a vertical upstream face, 3 m wide crest and an average downstream face slope of 1V:1.5H (formed as a series of steps 600 mm high). The crest of the starter dam is 155.1 m RL and is set to coincide with the top of the diversion culvert.

The downstream face is stepped to facilitate construction. The first step is set a minimum horizontal distance of 600 mm behind the plinth to enable placement and compaction of the Zone 2A material.

Each gravity block is approximately 10 m – 13 m long with a vertical contraction joint and shear key. Each horizontal lift is finished with a Type B construction joint with a specified provisional mortar layer should the horizontal lift joint require additional cohesion (for example if the joint surface is deemed insufficiently rough). The exact spacing of vertical contraction joints and horizontal lift
heights shall be confirmed to suit the Contractor’s concrete mix design and construction methodology, noting temperature and shrinkage control of the concrete is essential.

The vertical contraction joints include cast in PVC waterbar at the upstream face of the starter dam. The waterbar is cast into a concrete anchor trench excavated into the foundation rock and extends 500 mm along the dam crest to control seepage through the contraction joints.

The foundation of the starter dam is treated with a vertical upstream grout curtain and blanket grouting (an upstream and downstream blanket row either side of the deeper curtain grout row). Grouting will be undertaken following placement of the mass concrete by drilling holes from the crest of the dam. Refer Section 12 and the Drawings for further details on the starter dam grouting.

The design 28 day concrete strength for the starter dam is 30 MPa which is relatively high for mass concrete. The high concrete strength was adopted for durability reasons. Shrinkage and temperature reinforcement is provided for the exposed faces only (i.e. upstream face and crest of the starter dam).

10.4 Stability analysis

10.4.1 Summary

Table 10.1 presents a summary of the starter dam stability analysis results and compares these against the stability criteria outlined in the NZSOLD Guidelines 2015. The critical loading conditions that govern the starter dam stability are the construction flood (overtopping by up to 2 m during the 50 year ARI design flood), and the SEE (100% horizontal upstream and 30% vertical).

The stability analysis show that the starter dam is highly stable once the CFRD embankment is constructed behind it due to the stabilising effect of the fill. The starter dam is also stable in the construction overtopping case considered.

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Description</th>
<th>Mode</th>
<th>Calculated results</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>Static with 2 m overtopping during 50 year ARI flood</td>
<td>Overturning</td>
<td>Resultant in middle third</td>
<td>Resultant in middle half</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>1.9 (Friction) 16 (Cohesion)</td>
<td>FOS ≥1.3 (Friction only) FOS ≥2.0 (Cohesion not well defined)</td>
</tr>
<tr>
<td>Normal</td>
<td>Static NTWL (197.2 m RL)</td>
<td>Overturning</td>
<td>Resultant in middle third</td>
<td>Resultant in middle third</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>FOS &gt;&gt; 1.5</td>
<td>FOS ≥1.5 (Friction only) FOS ≥3.0 (Cohesion not well defined)</td>
</tr>
<tr>
<td>Unusual</td>
<td>Pseudostatic NTWL (197.2 m RL) +OBE</td>
<td>Overturning</td>
<td>Not assessed</td>
<td>Resultant in middle half</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>Not assessed</td>
<td>FOS ≥1.3 (Friction only) FOS ≥2.0 (Cohesion not well defined)</td>
</tr>
<tr>
<td>Extreme-flood</td>
<td>IDF (202.53 m RL)</td>
<td>Overturning</td>
<td>Resultant in middle third</td>
<td>Resultant within base</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>FOS &gt;&gt; 1.5</td>
<td>FOS ≥1.1 (Friction only) FOS ≥1.5 (Cohesion not well defined)</td>
</tr>
<tr>
<td>Extreme-earthquake</td>
<td>Pseudostatic NTWL (197.2 m RL) +SEE</td>
<td>Overturning</td>
<td>Resultant within base</td>
<td>Resultant within base</td>
</tr>
<tr>
<td>Loading condition</td>
<td>Description</td>
<td>Mode</td>
<td>Calculated results</td>
<td>Criteria</td>
</tr>
<tr>
<td>-------------------</td>
<td>------------------------------</td>
<td>---------------</td>
<td>--------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>1.6 (Friction)</td>
<td>FOS ≥1.1 (Friction only)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.3 (Cohesion)</td>
<td>FOS ≥1.5 (Cohesion not well defined)</td>
</tr>
<tr>
<td>Post earthquake</td>
<td>Static NTWL (197.2 m R)</td>
<td>Overturning</td>
<td>Resultant in middle third</td>
<td>Resultant within base</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sliding</td>
<td>FOS &gt;&gt; 1.5</td>
<td>FOS ≥1.2 (Cohesion well defined)</td>
</tr>
</tbody>
</table>

While typical gravity design does allow for some movement during extreme events, this is not acceptable for the starter dam as any movement could result in damage to the perimetric joint above and leakage into the rockfill embankment.

### 10.4.2 Design loads

The imposed geotechnical loads used in the starter dam stability analyses including the effect of reservoir surcharge pressure on the upstream face of the embankment, and embankment response during the design seismic events.

The Plaxis model was used for static pressures and the Quake/W model used for seismic pressures acting on the downstream face of the dam. Lower bound estimates were adopted for stabilising forces based on the combination of rockfill parameters that gave lower face pressures. Upper bound estimates were adopted for destabilising forces (i.e. pushing the starter dam upstream) based on the combination of rockfill parameters that gave higher face pressures. The geotechnical loads used in the stability analysis area presented in Table 10.2 below.

#### Table 10.2: Imposed geotechnical loads summary

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Estimated embankment loads on starter dam downstream face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal</td>
</tr>
<tr>
<td>Static (NTWL)</td>
<td>-2,800 kN/m (upstream)</td>
</tr>
<tr>
<td>Static (IDF)</td>
<td>-3,100 kN/m (upstream)</td>
</tr>
<tr>
<td>Seismic (OBE)</td>
<td>Not considered</td>
</tr>
<tr>
<td>Seismic (SEE) (100%H + 30% V)</td>
<td>-3,400 kN/m (upstream)</td>
</tr>
<tr>
<td>Seismic (SEE) (100%V + 30% H)</td>
<td>-2,700 kN/m (upstream)</td>
</tr>
</tbody>
</table>

### 10.4.3 Assumptions

The following key assumptions were made for the stability analyses:

- Horizontal seismic actions in both the upstream and downstream directions were considered along with coincident 30% vertical actions. Full vertical action (100%) coincident with 30% horizontal actions were also considered.
- The sliding stability analysis adopted the foundation rock/concrete friction angle of 45 degrees, and foundation rock/concrete cohesion of 500 kPa. Sliding factors of safety were considered with and without cohesion.
- Uplift water pressures have been assumed to be equal to reservoir water level at the upstream end, reducing linearly to zero at the downstream end, except where part of the base is calculated to be in tension. Cracked base analyses have been consider where the base was assessed as being not 100% in compression.
• The concrete face does not provide any passive resistance to sliding or overturning because it would require excessive movement and therefore possible damage to the face slab or joint.
• The rockfill loads on the back of the starter dam have been taken from the modelled pressures under static and seismic loading conditions and the design water level range. Lower bound values have been taken from the range of model output pressures to given conservative estimates of the loads for use in the stability analysis.
• Horizontal and vertical seismic actions have been considered in both the positive and negative directions (i.e. up and down, and upstream and downstream).

10.5 Shrinkage control

10.5.1 General

Shrinkage control measures have been considered for both the construction (curing) phase and for long term shrinkage.

Shrinkage control for mass concrete typically relies on construction controls and contraction joints rather than reinforcement (as is typical for standard reinforced concrete structures). It is not normal practice to provide reinforcement for mass concrete gravity dams for shrinkage control.

10.5.2 Construction controls

Construction controls for shrinkage typically include limiting pour geometry and sequencing to allow the concrete to cool evenly. Selection of suitable pour volumes and lift heights depends on a range of factors including concrete mix design, temperature controls and ambient temperature.

A maximum lift height of 1 m is typical, noting with the stepped profile on the downstream face lift that heights of 600 mm are expected. The actual lift height will be a requirement for the Contractor to establish.

10.5.2.1 Control joints (vertical contraction joints)

Additional measures were considered to control long term shrinkage in the exposed upstream faces of the starter dam (i.e. upstream face and crest only). The control of long term shrinkage can be achieved by placement of regular vertical contraction joints and reinforcement.

The spacing of the vertical control joints have been set generally in accordance with guidance from USACE (1995) Gravity Dams which recommends a maximum of 10 m spacing. A wider spacing of up to 13 m has been allowed to facilitate construction, given relatively low potential for long term shrinkage (due to the relatively small concrete mass and presence of reinforcement steel).

10.5.3 Long term shrinkage requirements

In accordance with NZS3101 Clause 8.8.2, the adopted reinforcement for the exposed faces is 1000 mm²/m (HD16-200 each way). This is appropriate because the starter dam design is not controlled by stress considerations.
11 Plinth

11.1 General

The plinth is a reinforced concrete block located at the upstream toe of the dam and forms the connection of the concrete face slab to the starter dam and rock foundation. The plinth is anchored to the foundation and a flexible perimetric joint provided between it and the face slab. The face slab is free to “float” on the rockfill face and the perimetric joint may open up slightly under water load (but not such that the tensile capacities of the water stops are exceeded).

11.2 Design basis

11.2.1 Standards and references

The plinth has been detailed in accordance with the following standards and references:

- ICOLD Bulletin 141 “Concrete rock fill dams”.
- Cruz et al. (2009) “Concrete Face Rockfill Dams”.
- NZS3101 “Concrete structures”.

11.2.2 Setout

A horizontal plinth arrangement has been selected (where the plinth sits on a horizontal bench rather than an inclined or sloping bench) to facilitate construction.

The upstream face of the plinth is inclined to be perpendicular to the concrete face (at the angle it intersects with the plinth) concrete. The downstream face of the plinth is inclined to match the apparent concrete face slope perpendicular to the plinth. This means that on the sloping abutments, the upstream face of the plinth is steeper and the downstream face is flatter. The plinth has been setout in discrete sections and to limit the number of changes in directions and associated plinth face geometries.

11.2.3 Dimensioning for hydraulic gradients

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

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

- Assessment based on foundation quality as shown at Table 11.1 below.
- Assessment based on Rock Mass Rating (RMR) values as developed by Cruz et al. (2009) and shown on Table 11.2.
- Assessment based on foundation classifications as shown at Table 11.3.
Table 11.1: Typical Hydraulic Gradients in terms of Foundation Quality (reproduced from ANCOLD, 1991)

<table>
<thead>
<tr>
<th>Foundation Quality</th>
<th>Acceptable Hydraulic Gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh</td>
<td>20</td>
</tr>
<tr>
<td>Slightly to moderately weathered</td>
<td>10</td>
</tr>
<tr>
<td>Moderately to highly weathered</td>
<td>5</td>
</tr>
<tr>
<td>Highly weathered</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 11.2: Typical Hydraulic Gradients in terms of RMR (reproduced from Cruz et al., 2009)

<table>
<thead>
<tr>
<th>RMR</th>
<th>Acceptable Hydraulic Gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 80</td>
<td>18 to 20</td>
</tr>
<tr>
<td>60 to 80</td>
<td>14 to 18</td>
</tr>
<tr>
<td>40 to 60</td>
<td>10 to 14</td>
</tr>
<tr>
<td>20 to 40</td>
<td>4 to 10</td>
</tr>
<tr>
<td>&lt; 20</td>
<td>2 (Generally handled by excavating to better material or providing a diaphragm wall)</td>
</tr>
</tbody>
</table>

Table 11.3: Classification of plinth foundations (reproduced from ICOLD Bulletin 141, 2010)

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Erodibility</th>
<th>Max Hydraulic Gradient</th>
<th>RQD</th>
<th>Weathering Degree (1)</th>
<th>Consistency Degree (2)</th>
<th>Discontinuities (3)</th>
<th>Excavation Class (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Non erodible</td>
<td>18</td>
<td>&gt;70</td>
<td>I to II</td>
<td>1 to 2</td>
<td>&lt;1</td>
<td>1</td>
</tr>
<tr>
<td>II</td>
<td>Slightly erodible</td>
<td>12</td>
<td>50-70</td>
<td>II to III</td>
<td>2 to 3</td>
<td>1 to 2</td>
<td>2</td>
</tr>
<tr>
<td>III</td>
<td>Erodible</td>
<td>6</td>
<td>30-50</td>
<td>III to IV</td>
<td>3 to 5</td>
<td>2 to 4</td>
<td>3</td>
</tr>
<tr>
<td>IV</td>
<td>Highly Erodible</td>
<td>3</td>
<td>0-30</td>
<td>IV to VI</td>
<td>5 to 6</td>
<td>&gt;4</td>
<td>4</td>
</tr>
</tbody>
</table>

(1) Weathering degree based on I for sound rock, VI for residual soil.
(2) Consistency degree based on 1 for hard rock and 6 for friable rock.
(3) Discontinuities based on weathered macro discontinuities per 10 m length.
(4) Excavation classes are 1 for blasting, 2 for heavy rippers with some blasting, 3 for light rippers and 4 for dozer blade.

The plinth foundation is expected to be moderately weathered to unweathered rock that is slightly erodible to non-erodible with a typical RMR value of 40 to 60 and an RQD of around 50.

Therefore where the plinth is founded on rock, it has been designed for a maximum hydraulic gradient of 10, giving a maximum plinth width of 4.5 m. The minimum width is generally considered to be 3 m. The basic slab is detailed for a 3 m width with wider slabs constructed as an extension under the rockfill. The extension is reinforced and anchored and is designed to be poured at the same time as the rest of the plinth.

Where the plinth does not provide adequate gradients for foundation defects, the clean-up is extended downstream of the plinth and further treatment is required. This treatment may include a reinforced shotcrete extension as directed on site to achieve the design hydraulic gradient. Foundation treatment is as noted in Section 8.
11.2.4 Structural

The design 28 day concrete strength for the plinth is 30 MPa as specified to meet the 100 year design life criteria (durability as per NZS3101). Shrinkage and temperature reinforcement is provided for the plinth as per NZS3101.

11.3 Plinth description

The plinth is anchored to the top of the starter dam (refer Section 10) at the base of the river channel and on excavated rock benches on the sloping abutments (refer Section 9). The plinth is dimensioned for the acceptable hydraulic gradients as per ICOLD Bulletin 141 (refer Section 11.2.3).

A horizontal plinth arrangement has been selected (i.e. sits on a horizontal bench) with a longitudinal grade on the sloping abutments to an angle of no greater than 30 deg. The design excavation profile for the plinth bench includes a 2 m wide bench on the upslope side of the plinth to facilitate construction.

The plinth is assumed be to poured in two stages with a horizontal construction joint and consists of a 300 mm thick horizontal slab (first pour) and a triangular head block (second pour). The geometry of the plinth head varies to suit the apparent interface angle with the concrete face, with the downstream face slope set perpendicular to the concrete face slab.

There are four main plinth geometry types (Types 1 to 4) with adjustments to the plinth head geometry to suit the plinth bearing (a, b, c) as shown on the Drawings and as follows:

1. Type 1 (a and b) below 170 m RL with an additional downstream apron (to lengthen the flow path) with a total plinth width of 4.5 m.
2. Type 2 on the starter dam with a total plinth width of approximately 1.8 m.
3. Type 3 (a, b and c) above 170 m RL with a total plinth width of 3.0 m.
4. Type 4 over the diversion culvert with total plinth width of approximately 1.8 m.

A single layer of reinforcement is provided in the top face of the base slab and plinth head to prevent cracking but provide sufficient flexibility for the slab to adapt to minor foundation movement. Secondary reinforcement is provided to connect the plinth head to the base and also around the perimetric joint waterstops.

The connection between the plinth and the concrete face is called the perimetric joint and is a type of free or contraction joint. The adopted perimetric joint detail features a PVC water bar and copper waterstop to limit seepage. The water stops are cast into the plinth head and protected during embankment construction until the concrete face slabs are poured. The plinth includes confining reinforcement either side of the PVC water bar.

The curtain and blanket grouting are undertaken through 80 mm diameter PVC pilot tubes cast into the plinth. This arrangement assumes full depth or downhole with packer method grouting.

The plinth features rows of galvanised HD32 diameter anchor bars grouted into the rock with bar rows at 2 m spacing’s along the plinth as shown on the Drawings. These anchors are provided to hold the plinth down during grouting operations. A minimum embedment depth into rock of 3 m is specified, noting longer bars may be specified on site to suit encountered rock conditions.

The adopted anchor design is based on precedent for the anticipated foundation characteristics. The design arrangements were also checked for tensile capacity of the bars, and pull out strength for grout uplift pressures of up to 100 kPa immediately underneath the plinth.
12 Grouting

12.1 General
Grouting is specified to reduce seepage immediately underneath the following components:

- Plinth.
- Starter dam.
- Ogee weir.
- Upstream end of diversion culvert.
- Selected locations under the crest ramp (at bridge abutment) and parapet wall (at true right abutment).

12.2 Design basis

12.2.1 Standards and references
The following standards and references informed the grouting arrangements specified:


12.2.2 Target closure permeability
The target permeability standard for the curtain and blanket grout is six lugeons for surface zones (upper 15 m) as recommended by Houlsby (1990).

12.2.3 Vibration control limits
Damage to the installed grout due to adjacent excavation and filling operations is a key risk to the performance of the grout system following closure. Maximum allowable peak particle velocities (PPV’s) are set in the Specification to provide control of vibrations at the grout (as is common for concrete structures). The allowable PPV’s were selected based on published guidance and experience from other projects, with lower PPV’s set for relatively fresh grout and higher limits allowed for cured grout.

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

The investigation drilling undertaken along the plinth line consisted of relatively short holes. The recorded permeability at the bottom of the holes is open to interpretation. The recorded water takes indicated rock dilation and this is generally accepted as being due to compression of joints above and below the test areas. In this case, the assigned permeability was usually taken as the results obtained by lower pressures and would indicate a low permeability at the bottom of drill holes. This is the preferred interpretation as adopted by Houlsby (1990).
An alternative interpretation (Quiñones-Rozo) adopts the water take obtained from a pressure equal to the storage head, and in this case permeability at the bottom of the holes would be generally high.

Site investigations seldom provide sufficient detail for a detailed grouting program. The proposed grouting arrangement provides for 20 - 33 m deep primary holes at 12 m spacing under the starter dam and the lower plinth. 15 m deep primary holes are specified under the ogee weir and upper plinth. These holes will be used to fully investigate the foundation.

Foundation grouting consists of a single line grout curtain for the full length of the plinth and spillway crest flanked by two rows of blanket grouting. An initial arrangement for secondary, tertiary and possibly quaternary holes is shown on the Drawings. The final depth, grout hole spacing and extent of the grout curtain can only be determined during construction, as the results of water pressure testing and grouting become available.

The specified method of grouting for the curtain grout is downstage with packers. Holes are to be percussion drilled with a minimum diameter of 30 mm.

In areas of higher permeability, grout takes may be high, and multiple applications of grout may be required. Based on a primary hole spacing of 12 m, grouting is likely to be required to at least tertiary spacing. Quaternary holes are shown provisionally on the Drawings. The depth of curtain grout holes reduces from the primary holes (20 – 33 m deep) to the secondary and tertiary holes (10 - 20 m) as per Houlsby (1990).

The defect pattern indicates that inclined holes at 70 degrees to the horizontal should intersect the main defect pattern on the left abutment. Vertical holes have been adopted for the base of the valley (under diversion culvert, starter dam and plinth) and the right abutment. Additional angled grout holes specially oriented across major shear zones may occasionally be required where identified during construction.

12.4 Blanket grouting

Blanket grouting will be required to reduce seepage immediately underneath the foundations of the plinth, starter dam, and ogee weir. Blanket grouting is especially important for the plinth stability noting foundation disturbance during excavation can result in more dilated and fractured rock. The design hydraulic gradient under the plinth is relatively high and blanket grout holes in this location are intended to consolidate the foundation to increase the seepage path length.

Blanket hole rows are specified upstream and downstream of the centrally placed curtain grout row. Rows that are approximately 1 m upstream and 1 m downstream of the curtain with holes at 3 m spacing along each row. Blanket grout holes of 5 m effective vertical depth are proposed with upstream inclined holes on the true left, and downstream inclined holes on the true right.
13 Embankment

13.1 General

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

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

There has been little change in CFRD practice for dams such as Waimea Dam, since the above mentioned Cooke and Sherard (1987). Those changes that have occurred are best summarised in the recent ICOLD Bulletin 141 (ICOLD, 2010) and Cruz et al (2009).

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

A large number of CFRD constructions have been completed in Australia, mostly in NSW and Tasmania, but also South Australia, Victoria and Queensland. The highest is the 122 m high Reece Dam in Tasmania. No serious problems have been encountered with these dams, nor with similar height dams constructed overseas (noting there are significant numbers of CFRD’s in Brazil and China).

Some dams have suffered from leakage through the concrete face, generally due to poor construction practice. Leakage is an operational risk and not a dam safety issue as the design can safely handle flow through the rockfill with significant leakage from the concrete face. The 40 m high Brogo Dam (New South Wales, Australia) filled and the spillway operated prior to construction of the concrete face. Although based on an older design with pervious Zone 2B material, the dam handled this situation without difficulty, passing an estimated discharge of 7 m$^3$/sec though the rockfill.

Current designs provide a reasonably impervious Zone 2B material that limits leakage from any face slab deficiencies. The exposed concrete face is able to be repaired if excessive leakage does occur, noting this would likely require diver inspections to identify significant leakage and reservoir draw down for repair work.

The dam embankment for the Waimea Dam is approximately 53 m high and 220 m long at the crest. The upstream and downstream face are sloped at 1V:1.5H. The minimum crest width is 6 m with a localised widening at the true right abutment up to 12.5 m width. The embankment details are shown on the Drawings.

The external batter slopes of 1.0V:1.5H provide a degree of conservatism for the high earthquake loads. They also allow the use of a processed gravel in the upstream Zone 2C and the use of coarse gravel material in downstream Zone 4.
13.2 Design basis

13.2.1 Standards and references

The rockfill embankment component of the CFRD has been designed in accordance with the following standards and references:

- Cooke (1993) “Concrete face rockfill dams”.
- Cooke & Sherard (1987) “Concrete face rockfill dam”.
- Cruz et al. (2009) “Concrete face rockfill dams”.
- FEMA (2011) “Filters for embankment dams best practices for design and construction”.
- ICOLD Bulletin 141 (2010) “Concrete face rockfill dams concepts for design and construction”.
- International CFRD precedents.

13.2.2 Material properties

The adopted material properties for the Waimea Dam embankment are based on guidance for CFRD design (as per the references above) and the expected on site materials based on the site investigations to date. The adopted bulk rockfill (Zone 2B) grading envelope, estimated shear strength and elastic modulus are based on the results of the on site trial excavation and embankments (with associated testing) undertaken in March 2018. The presented design reflects the anticipated rockfill properties from the on site excavations (predominantly from the spillway cut area).

The design material properties were used in the dam stability and seismic performance analyses as presented in Section 14 below. The specific material gradings and properties are included in the Specification and are based on the trial embankment testing results.

Some uncertainty remains in the rockfill quality and available quantities from the design excavation areas. Should additional suitable alluvial deposits be identified by the Contractor, these may be used in a separate zone in the upstream shoulder of the dam during construction (subject to confirmation by Designer at that time).

13.2.3 Trial embankments

Geotechnical investigations were undertaken to inform the Stage 3 design as reported separately in the Stage 3 design report (T+T, 2014). These investigations included excavation of moderately weathered rock and construction of two small trial embankments of the potential Zone 3B rockfill.

The trial embankments were compacted with a 7.5 tonne vibrating roller, a slightly smaller machine than the 10 tonne vibrating roller specified for embankment construction. The rockfill was placed in 300 mm thick layers and compacted with up to nine roller passes. The measured in situ densities were relatively high and up to 2.36 t/m$^3$. Minimal breakdown of the compacted rockfill was reported, noting the relatively small material size.
Additional excavation and embankment trials were undertaken in March 2018 to facilitate the design development for Stage 4 and development of pricing by the ECI Contractor. These trials also focused on the Zone 3B rockfill (bulk rockfill).

The March 2018 trials included opening up two excavation borrow areas; one each for where marginal and better quality rockfill materials were anticipated. Excavation was undertaken using a 30T excavator with a toothed bucket for the marginal material, and in the harder rock (e.g. where the rock was breaking down into smaller fragments under the bucket teeth) a single ripping tine/tooth was also used.

The excavated rockfill appeared to have a larger precompaction grading than suggested by the Stage 3 trials, and differed depending on the rock excavation method used. For example, the ripping tine/tooth resulted in much larger rockfill clasts than the toothed bucket.

The trial embankments were formed on an excavated platform approximately 2 m above the river level. The embankment footprints were approximately 20 m long by 7 m wide.

The rockfill was laid in roughly 600 thick loose layers and then hosed with water at a rate of approximately 200 l/m$^3$. Compaction of the fill was undertaken with a 12T vibratory roller, with up to eight passes in total. The measured settlement and compaction generally plateaued after six passes.

The rockfill surface showed significant breakdown/fracturing of the surface layer with each subsequent pass, noting this was most evident after the eighth pass. This confirms the design guidance that over-compaction of rockfill can be detrimental, especially where breakdown results in generation of significant fines and lower rockfill permeability.

Water was applied to the finished surface to observe whether it would pond and if so how long it took to drain away. There were select locations where finer dirtier rockfill was identified and these areas ponded water and took longer to drain than the rest of the placed rockfill. The majority of the rockfill only ponded water at high application rates (directly under the hose) and drained away almost immediately.

Samples were taken from the excavation stockpile for laboratory testing (solid density, grading (PSD), soundness, crushing resistance). The insitu density was measured via the water replacement test and NDM testing.

The results of the March 2018 excavation and embankment trials were used to inform the Stage 4 design as presented in the following sections.

13.3 Embankment zoning

13.3.1 General

The proposed embankment zoning is shown at Figure 13.1 below and a description of the materials is shown at Table 13.1. This zoning is similar to the Stage 3 arrangements with the following modifications:

- Further details are provided of the transition zones for materials behind the plinth along the abutments. Wider blanket zones of Zone 2B and 3A materials are provided in accordance with the recommendations of ICOLD Bulletin 141.
- The extent of the Zone 2C material has been adjusted to limit this zone to the upstream extent of the diversion culvert only.
- The material grading envelopes have been altered to reflect the results of the March 2018 trial embankment testing, and to meet non erosion filter compatibility requirements (as summarised in Fell et al., 2015).
### Table 13.1: Embankment material descriptions

<table>
<thead>
<tr>
<th>Zone</th>
<th>Description</th>
<th>Material description and design purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>2A</td>
<td>Perimetric joint fine filter</td>
<td>Processed sand filter to control potential leakage through the perimetric joint.</td>
</tr>
<tr>
<td>2B</td>
<td>Concrete face support</td>
<td>Processed gravel or crushed rock that provides a relatively stiff support layer to the concrete face. Has a secondary function to reducing seepage from the concrete face.</td>
</tr>
<tr>
<td>2BF</td>
<td>Transitional filter between Zone 2A and adjacent Zone 2B and 3A</td>
<td>Processed sandy gravel filter to provide filter compatibility between the Zone 2A material and the coarser rockfill layers downstream.</td>
</tr>
<tr>
<td>2C</td>
<td>River gravel</td>
<td>River gravel placed behind the starter dam around the diversion culvert to provide increased stiffness.</td>
</tr>
<tr>
<td>3A</td>
<td>Rockfill transition zone</td>
<td>Free draining rockfill obtained from slightly weathered to fresh rock excavated on site. Material grading to provide transition between finer face support material and the coarser bulk rockfill.</td>
</tr>
<tr>
<td>3B</td>
<td>Bulk rockfill zone</td>
<td>Free draining rockfill obtained from slightly weathered to fresh rock excavated on site.</td>
</tr>
<tr>
<td>3B(R)</td>
<td>Rockfill in reinforced zone</td>
<td>Larger sized rockfill placed within downstream reinforced rockfill (Temporary works design by GHD).</td>
</tr>
<tr>
<td>3C</td>
<td>Reinforced rockfill</td>
<td>Select fresh large rockfill placed on the downstream shoulder of the downstream coffer dam (Temporary works design by GHD).</td>
</tr>
<tr>
<td>3D</td>
<td>Facing rock on downstream face</td>
<td>Select fresh large rockfill placed on the downstream face to provide scour resistance.</td>
</tr>
<tr>
<td>3E</td>
<td>Quick rise berm</td>
<td>Larger sized rockfill placed centrally in a thin zone during construction to enable the design construction flood level to be achieved quicker (Temporary works design by GHD).</td>
</tr>
<tr>
<td>3F</td>
<td>Toe berm and seepage control bund rockfill</td>
<td>Lower strength rockfill placed on the downstream toe to form the toe berm for access to the outlet works, control building and fish pass inlet, and to form the seepage control bund for collection and monitoring of dam seepage.</td>
</tr>
<tr>
<td>3G</td>
<td>Reinforced rockfill facing</td>
<td>Exposed rockfill on downstream reinforced face with a 200/80 grading (Temporary works design by GHD).</td>
</tr>
<tr>
<td>4</td>
<td>Coarse gravel drainage</td>
<td>River gravel placed on the foundation under the downstream shoulder either side of the diversion culvert. River gravel is provided to facilitate collection of seepage and increase stiffness around the diversion culvert.</td>
</tr>
<tr>
<td>Armour</td>
<td>Rip rap armour stone</td>
<td>Select large angular fresh rockfill placed on the downstream face of the toe berm to provide erosion resistance under tailwater conditions.</td>
</tr>
</tbody>
</table>
13.3.2 **Zone 2A Perimetric joint filter**

Zone 2A is a fine sand filter used in small quantities immediately downstream of the perimetric joint and on jointed or sheared foundations downstream of the plinth. This, together with the adjacent Zone 2B material provides a high modulus fill directly behind the perimetric joint.

Zone 2A needs to satisfy conventional filter criteria for retention of joint infill and shear material in the foundation. In practice, a concrete sand is widely used as a fine filter (ICOLD, 1994). The grading adopted for the fine filter is as per ICOLD Bulletin 141 Table 10 Alternative Gradation. The adopted grading envelope for the Zone 2A material is reproduced below in Figure 13.2.

![Zone 2A perimetric joint filter design grading envelope.](image1)

13.3.3 **Zone 2B Concrete face support**

Zone 2B is a sandy gravel sized material with a maximum size of 75 mm that forms the outer upstream layer, upon which the concrete face slab of the main embankment is seated. It provides uniform support for the face slab and acts as a lower permeability layer to restrict leakage in the...
event of face slab cracking or joint leakage. Zone 2B is expected to be produced from crushed rock. It could also be produced by processing the river gravels.

The adopted design grading envelope for Zone 2B is based on the envelopes presented in ICOLD Bulletin 141 (2010) Tables 12 and 13. The modified ICOLD Bulletin 70 gradation was adopted with minor adjustments to the allowable fines and coarse envelope side. The percentage of fines (passing the 0.075 mm sieve) has been restricted to no more than 5% (as per the Keenleyside and Mohale Dams). The coarse side percentages passing the 4.75 and 19 mm sieves have been rounded up to 40% and 60% respectively to be consistent with the fine side envelope percentages. The adopted Zone 2B material grading envelope is presented in Figure 13.3 below.

**Figure 13.3:** Zone 2B concrete face support material design grading envelope.

Zone 2B materials do not satisfy conventional filter criteria for segregation such as those provided in ICOLD (1994). The broad grading has a coefficient of uniformity and D90/D10 ratios far higher than those required for earth dam filters. This is recognised by ICOLD (2010) and all of the CFRD literature. It has been well established that segregation is not a problem in these materials provided sand sizes exceed 35% and normal care is taken during placement. Lower sand proportions have been used on many dams but require additional care during placement.

Zone 2B is placed in 400 mm layers in a damp condition and compacted by four to eight passes of a 6 - 10 tonne smooth drum vibratory roller (subject to confirmation following compaction trials). Placement of this material typical uses lighter plant than the rockfill due to the proximity to the concrete kerbs (refer Section 13.3.5 below). The target minimum compaction standard for this material to achieve a stiff face support material is 98% of maximum dry density (using the standard laboratory compaction test). Dry densities of 2.0 – 2.2 t/m$^3$ are typically achieved as per ICOLD (2010).

### 13.3.4 Zone 2BF Transitional filter

The Zone 2A and Zone 2B materials do not meet the design criteria for no erosion filter compatibility. Therefore a modified Zone 2B material referred to as “Zone 2B Filter” is specified between the Zone 2A material and the adjacent downstream materials (i.e. Zone 2B, Zone 3A and Zone 2C). This material has restricted fines to provide better drainage when compared with the Zone 2BF material.

The design grading envelope for Zone 2BF filter was determined using the no erosion criteria summarised in Fell et al. (2015) and checked for internal stability using Wan & Fell (2008) and Kenny & Lau (1985). The design grading envelope is presented in Figure 13.4 below.
13.3.5  Face protection kerbs

The face of the relatively fine Zone 2B material requires protection from rainfall runoff and scour prior to placement of the face slab. Current practice generally uses concrete kerbs placed inside the concrete face slab as shown at Figure 13.5. In addition to protecting the Zone 2B material, the kerbs facilitate compaction of the Zone 2B material.

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

ICOLD (2010) gives a typical concrete mix with 75 kg/m$^3$ of cement, 19 mm maximum aggregate (1170 kg/m$^3$), sand (1170 kg/m$^3$), and 125 l/m$^3$ of water, noting weaker mixes using 60 kg/m$^3$ of cement have also been used recently. The reported typical extrusion rate is 40 to 60 m/hour. Concrete compressive strengths are around 2 to 5 MPa and Zone 2B can be placed and compacted against the kerbs as soon as one hour after extruding.

![Zone 2BF design envelope](image-url)
13.3.6 Zone 2C upstream support zone

Zone 2C is an unprocessed sand-gravel and is anticipated to be obtained from the alluvial gravel deposits on site. Alluvial gravel typically has a much higher modulus than rockfill and for the Waimea Dam is specified to limit the deformation at the starter dam perimetric joint around the diversion culvert.

The Zone 2C material is specified in a 12 m thick (horizontal) zone behind the starter dam and either side of the diversion culvert from the starter dam crest level down to the foundation level (sloping at 1V:1.5H). Zone 2C extends on the true right to the rock abutment, and on the true left side of the diversion culvert extends horizontally for 5 m before transitioning down at a slope of 1V:1.5H to terminate approximately 15 m from the culvert wall.

The Zone 2C material is covered by a 1.66 m thick layer of Zone 3A material, except where it is placed immediately behind the Zone 2BF transitional filter material on the starter dam crest.

The adopted grading curve for the Zone 2C material is identical to the Zone 3A material (refer Section 13.3.7 below) as determined from filter compatibility checks with the adjacent Zone 2BF material. It is noted that a higher fines content may also be acceptable (of less than 10% passing the 0.075 mm sieve).

13.3.7 Rockfill (Zones 3A and 3B)

The key design requirements for the rockfill are to provide durable, sufficiently strong, stiff and free draining material zones within the dam. Achieving these requirements for the Waimea Dam requires development of a placement and compaction methodology that achieves stiff well compacted rockfill, without resulting in undue breakdown of the rockfill clasts during compaction and low permeability.

If the placed rockfill is not free draining, the key safety feature of the CFRD design is lost and internal drainage zones would be necessary (e.g. a central chimney drain similar to earth embankment dams).

Stiff rockfill is highly desirable in the upstream shoulder supporting the concrete face to limit deflection and the potential for cracking of the face due to compression at first filling.

Based on the investigations undertaken to date, and the design excavation modelling, it is anticipated sufficient suitable rockfill material will be available from the on site excavations. Significant excavation is required for the spillway and the better quality rockfill is anticipated from...
the base of these excavations (especially in the upper chute area). Additional sources of suitable rockfill are anticipated from the diversion culvert and potentially some of the road excavations.

The rockfill is anticipated to be won via excavation with some blasting required for the fresh rock at depth. The selected methods of excavation in each rock weathering class will strongly influence rockfill production rates.

The method of excavation and degree of weathering is likely to influence the properties of the resultant rockfill. It is important that the Contractor develops and maintains suitable excavation procedures that do not result in excessive breakdown of the rock (e.g. ripping hard rock that fractures into small particles).

Zone 3B is the main rockfill zone. The rockfill in the upstream third of the embankment carries the water load from the concrete face to the foundation. The rockfill at the crest is an area subjected to high seismic loads. Adequate performance at both of these key areas especially relies on high strength stiff rockfill and consideration of these areas has informed the compaction requirements outlined in the Specification and the zoning shown on the Drawings. In the lower half of the downstream shoulder, thicker layers with less compaction could be considered as allowed for in the Specification under direction by the Engineer.

The design rockfill grading envelope for the bulk rockfill (Zone 3B) is presented in Figure 13.6 below. The envelope was developed from the March 2018 trial embankment compacted rockfill grading curves, with consideration of CFRD design recommendations for this zone.

The Zone 3B envelope meets the filter compatibility criteria for the upstream Zone 3A, but does not meet the permeability criteria of $D_{15F} \geq 4D_{15B}$. This is considered acceptable on the basis that the Zone 3B material is expected to be free draining and able to draw seepage away from the upstream zones. Zone 3B would still retain the upstream Zone 3A material.

As a general guiding principal zoned fill embankments should have increasing permeability from upstream to downstream to facilitate drainage in this direction. This principal is intended to reduce the possibility of pressure build up in the upstream zones resulting in heave or upstream erosion (e.g. rapid draw down scenarios). Providing the Zone 3B material is free draining, the adopted arrangements are considered appropriate for the Waimea Dam.

Further measures to facilitate drainage within the Zone 3B material, include the requirement to place the coarser rockfill in the upstream shoulder and encourage seepage to flow into the Zone 4 material which extend to the downstream shoulder. Finer Zone 3B rockfill can be placed on the downstream shoulder. This is consistent with the approach for stiffer material to be placed in the upstream shoulder and crest areas as shown on the Drawings.

Regrading the upstream materials to suit the finer Zone 3B material for the permeability criteria would require significantly finer materials. This would result in Zone 2A and Zone 2B (face support) materials that are finer than the ICOLD Bulletin 141 guidance. Finer Zone 2B material is to be avoided as this would result in a greater percentage of fines which could hold open a crack or pipe and compromise the integrity of this zone.
Figure 13.6: Zone 3B rockfill design grading envelope and March 2018 trial embankment gradings.

The March 2018 trial embankments indicated that the placed rockfill will break down along microfractures to produce a relatively small sized compacted rockfill with a $D_{50}$ of 25 – 35 mm, less than 20% passing 4.75 mm and less than 5% fines (<0.075 mm). This grading and the site testing indicate this is a free draining rockfill with a permeability of around $5 \times 10^{-3}$ m/sec that should satisfy the requirements for a CFRD embankment.

While successful rockfills have used finer materials these are generally regarded as soft rockfill where strength is provided by material density rather than point to point contact and have lower permeabilities.

The design layer thicknesses specified for the rockfill are substantially smaller than typical practice for larger size rockfill which would typically use 800 mm to 1,000 mm upstream of the centreline and 1,600 mm downstream. The thinner layers reflect the anticipated smaller size of the rockfill.

Additional test embankments will be required during construction to optimise layer thickness and the number of compaction passes required for a competent fill. Where significant changes occur in the excavated rock, new trial embankments will be required to recalibrate.

Zone 3A provides a narrow transition from Zone 2B to Zone 3B that generally satisfies filter criteria. Australian practice (ANCOLD, 1991) has been to specify Zone 3A only by layer thickness and not require a specific grading envelope. However, the design grading envelope adopted for the Zone 3A material (refer Figure 13.7 below) has also been checked for internal stability and filter compatibility with the Zone 2B and 2BF materials in accordance with the methods summarised in Fell et al. (2015).

Cruz et al. (2009) note that Zone 3A is sometimes processed but is generally obtained from finer rockfill selected in the quarry and stockpiled.
13.3.8 Rockfill Zones 3B within toe mesh, 3C, 3E and 3G

The downstream shoulder reinforced rockfill zone design is described in the GHD report “Waimea Community Dam Reinforced rockfill design” dated June 2018, and presented on the GHD drawings titled “Waimea Community Dam Temporary Diversion Works” Drawing Nos: 23-16255-C001 to C092.

The temporary works design (GHD) has developed the downstream coffer dam/reinforced rockfill design further from that presented in Stage 3. The following key changes were made to the Stage 3 design:

- Increase to the reinforced rockfill zone crest level from 173.4 m RL to 176.4 m RL to suit adopted Stage 4 diversion works design.
- Increase to the reinforced rockfill zone crest width from 6 m to 10 m.
- Alterations to the zone geometry in this area with Zone 4 finishing upstream of the reinforced rockfill zone, Zone 3B extending up to the reduced width of Zone 3C material (which extends to the downstream toe slab) as shown on the GHD drawings.
- Inclusion of two additional rockfill zones; a modified Zone 3B within the reinforcement (with larger size material and wider grading envelope) and a new Zone 3G rockfill zone on the reinforced downstream face (a larger screened 200/80 mm sized material).
- Extension to the anchor bar length and placing these horizontally (rather than inclined as per Stage 3).

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

The rockfill available from the Waimea Dam site is much smaller than the usual specification outlined above (as determined from excavation trials and geotechnical investigations to date), and during Stage 3, the mesh design was adjusted to use a smaller mesh (Type 333 or equivalent with 6.3 mm bars on a 75 mm grid). The smaller mesh approach has been developed for the Stage 4 design by GHD.

The Zone 3C material envelope specified by GHD requires a maximum size of 400 mm with 50% larger than 37.5 mm and no more than 5% passing the 2.36 mm sieve. The 300 mm thick Zone 3G material is placed over the Zone 3C material is a screened rockfill with a minimum effective diameter of 80 mm and a maximum size of 200 mm (to match the opening size of the mesh and to retain the rockfill during flow through conditions).

Zone 3E is intended to be larger size rockfill for the quick rise berm. This zone is intended to enable a rapid increase in the embankment height to enable larger construction flood levels and flow through the diversion culvert without overtopping the embankment. This material is required to withstand flow through during the construction diversion and be compatible with the final dam design Zone 3B material (which requires free draining material).

We have reviewed the reinforced rockfill aspects of the temporary works design in documentation supplied and the proposed arrangements appear to be compatible with the permanent works design. The compaction requirements for the reinforced rockfill shall be such that the design gradings are met post compaction and that the design strength and permeability criteria are met.

13.3.9 Downstream rockfill Zones 3D and 3F

Zone 3D provides a facing of stronger, larger sized material over the downstream face above the reinforced rockfill and is anticipated to be obtained by stockpiling larger rock in the quarry. This zone intended to improve the scour resistance of the exposed downstream face. The rockfill sizes presented in the Specification are based on precedent rather than specific analysis.

Zone 3F is intended to be the lower strength rockfill that acts as engineered fill form the toe access berm and to support the seepage control membrane. This zone is still required to be free draining but can be lower strength as it is not subjected to the amplification effects of the dam (i.e. lower seismic loads than the CFRD crest) and given its low height and wide crest (i.e. it is relatively stable). The stability of the toe berm is not critical to the overall dam stability.

13.3.10 Zone 4 drainage gravel

Zone 4 is a layer of coarse river gravel in the river section below RL 155 to provide drainage. It provides a source of high quality coarse drainage material in the initial stages of the construction when high quality rockfill material is expected to be difficult to obtain. Zone 4 has a higher permeability than the ripped or blasted rock and can also be used in the reinforced rockfill zones to provide a larger size material that will not be washed through the reinforcing fabric.

The design grading envelope of the Zone 4 material is presented in Figure 13.8 below.
The geotechnical investigations (T+T, 2012) have identified several potential borrow areas of suitable alluvial materials for use in the dam. Lee River alluvium contains high strength aggregate up to 600 mm diameter. Potential sources for Zone 4 material include:

- The existing armour layer in the river bed with the material smaller than 4.75 mm removed (i.e. sands and smaller).
- Upstream alluvial deposits near Waterfall Creek.

13.3.11 Potential internal drainage zone(s)

March 2018 embankment trials indicate that while clean small sized free draining rockfill is achievable there remains a possibility that rockfill will break down more than is anticipated producing a less pervious fill. If there is a concern with rockfill permeability during construction of the downstream stage, the embankment zoning may need to be adjusted to incorporate an inclined chimney filter that connects to the Zone 4 gravel zone in the base of the downstream stage.

The chimney filter material should provide filter stability for a finer rockfill with significant breakdown while still providing good drainage capabilities, and a grading similar to that specified for Zone 4 would be suitable. Additional filter zones between Zone 3B and the chimney filter should not be required but will need to be assessed at the time should additional drainage measures prove necessary.
14 Embankment stability

14.1 General

The stability of the Waimea Dam embankment was assessed for a range of static and seismic design cases to quantify the design performance of the embankment.

There are four components to the embankment stability assessments as summarised below and described in the following sections:

1  Seepage modelling of water flow through the embankment to determine the internal water surface profile for use in the stability analysis (using Seep/W software).
2  Calculation of slope stability based on:
   a  Published information for material strengths and investigations undertaken for the Waimea Dam.
   b  Limit equilibrium slope modelling to find yield accelerations and associated stability factors of safety (using Slope/W software).
3  Calculation of potential earthquake induced slip displacements based on:
   a  Pseudostatic limit equilibrium slope modelling to find yield accelerations as above (using Slope/W software).
   b  Published empirical slip displacement calculation methods.
4  Confirmation of the potential earthquake induced slip displacements using dynamic earthquake modelling based on:
   a  Equivalent linear dynamic modelling for four time histories/accelerograms determined as being suitable for the site (refer Section 2 for details) (using Quake/W software). The modelling outputs include:
      i  Calculated crest accelerations.
      ii  Newmark sliding block displacement calculations.

The assessed earthquake induced slip displacements at the dam crest are also relevant for the design of the parapet wall and crest ramp structures, and the concrete face. Separate analyses were undertaken to assess the displacements of the parapet wall and crest ramp structures relative the embankment crest (as sliding blocks) as described in Section 16.

The detailing of the concrete face at the crest was developed to allow for displacement of the parapet wall relative to the crest only. Significant deformation of the dam crest would likely result in some cracking to the concrete face above the NTWL based on the analysis described in this section. Repair of the concrete face above NTWL is expected to be relatively achievable (especially when compared to hypothetical repair work lower down near the starter dam for example). Further details of the concrete face arrangements are presented in Section 15.

14.2 Results summary

The assessed embankment stability is consistent with the adopted design criteria, the NZSOLD Guidelines 2015, and international precedents for concrete faced rockfill dams. The static factors of safety range between 1.4 and 1.8 and are above the minimum criteria of 1.5 (usual) and 1.3 (post earthquake).

The stability results give a range of seismic displacements that are within tolerable bounds for loss of freeboard and horizontal deformation (e.g. the parapet wall would remain on the dam crest following the SEE and aftershock events). The assessment shows that minor deformation during the OBE seismic events is possible but within acceptable limits (for minor repairable damage), and more
significant but tolerable deformations are possible during the SEE (e.g. horizontal displacements of between 300 and 570 mm) and aftershock seismic events (e.g. horizontal displacements of between 30 and 200 mm). The cumulative deformation from the SEE and aftershock events is between 330 mm and 770 mm (horizontal) and 220 – 530 mm (vertical).

The results of the embankment stability analyses described in this report are summarised below in Table 14.1. Further details of the design methods, inputs, assumptions and sensitivity checks are described in the subsequent sections.

Table 14.1: Embankment stability analyses summary

<table>
<thead>
<tr>
<th>Design case</th>
<th>Slope stability FOS</th>
<th>Crest displacement (downstream direction) (base estimate and range)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static stability at NTWL with no flow through.</td>
<td>1.72 (1.6 – 1.8)</td>
<td>N/A</td>
</tr>
<tr>
<td>Static stability at IDF peak reservoir level with no flow through.</td>
<td>1.68</td>
<td>N/A</td>
</tr>
<tr>
<td>Static stability at NTWL with flow-through (construction case and post SEE).</td>
<td>1.47 (1.4 – 1.6)</td>
<td>N/A</td>
</tr>
<tr>
<td>Seismic stability and performance at NTWL for OBE cases.</td>
<td>&lt;1.0 (~&lt;1.0 to 1.0)</td>
<td>Horz. &lt;10 mm (0 – &lt;10 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vert. approx. 7 mm (0 – 7 mm)</td>
</tr>
<tr>
<td>Seismic stability and performance at NTWL for SEE cases.</td>
<td>&lt;1.0</td>
<td>Horz. 300 mm (300 – 570 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vert. approx. 200 mm (200 – 380 mm)</td>
</tr>
<tr>
<td>Seismic stability and performance at NTWL for SEE aftershock event with flow-through.</td>
<td>&lt;1.0</td>
<td>Horz. 100 mm (30 – 200 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vert. approx. 70 mm (20 – 250 mm)</td>
</tr>
</tbody>
</table>

The estimated long term crest settlement at the maximum embankment height is between 160 to 330 mm over the 100 year design life. Given the logarithmic scale this means most of the settlement is estimated to occur in the first 10 years.

14.3 Design basis

14.3.1 Standards and references

The following standards and references where used to inform the embankment stability analyses:

- Barton & Kjaernsli (1981) “Shear Strength of Rockfill”.
- Chwang & Housner (1977) “Hydrodynamic pressures on sloping dams during earthquakes”.
- GNS (2017)”Seismic Hazard Assessment for the Proposed Waimea Dam”.
- Makdisi & Seed (1978) "Simplified procedure for estimating dam and embankment earthquake induced deformations".
Makdisi & Seed (1979) “Simplified procedure for evaluating embankment response”.
Rathje & Antonakos (2011) “A unified model for predicting earthquake-induced sliding displacements of rigid and flexible slopes”.
Bureau et al. (2008) “Influence of vertical shaking on embankment dam seismic response”.

14.3.2 Methodology
The embankment stability and seismic displacements were primarily assessed using pseudostatic and empirical methods, with additional dynamic analysis undertaken for the SEE as a check of the pseudostatic seismic displacement estimates.

The embankment stability was assessed against the design criteria (refer Section 2) and as per the N兹SOLD Guidelines 2015. The following design cases were assessed:
1. Static stability at NTWL with no flow through.
2. Static stability at IDF peak reservoir level with no flow through.
3. Static stability at NTWL with flow-through (construction case and post SEE).
4. Seismic stability and performance at NTWL for OBE cases.
5. Seismic stability and performance at NTWL for SEE cases.

It is acceptable for some deformation/displacement of the embankment to occur under seismic loads provided the extent of deformation does not result in unacceptable performance (i.e. in the case of the SEE it is such that catastrophic failure of the dam does not result).

The adopted design approach to assess the potential embankment displacements due to seismic loadings based on pseudostatic analyses and empirical methods with sensitivity analyses to obtain a range of displacement estimates. Dynamic analysis were also undertaken as an additional check of the displacements for the SEE design case.

Dynamic analysis provides more detailed assessment of embankment performance but is more complex and time consuming than the pseudostatic methods. For the Waimea Dam, the dynamic analyses undertaken supported the results obtained from pseudostatic methods and therefore further dynamic modelling was not considered necessary or beneficial for the purposes of design.

14.4 Rockfill properties

14.4.1 Properties for the pseudostatic analysis

14.4.1.1 Shear normal functions
Shear normal functions were adopted for the rockfill and gravel materials based on Barton & Kjaernsli (1981). These functions were selected following review of a range of published functions including Mohr Coulomb, Douglas (2002), Hardin & Kalinski (2005), and ICOLD Bulletin 141 (which references Leps (1970)). The calculated shear-normal relationship for the rockfill material in Zone 3B is shown in Figure 14.1 below. Sensitivity checks were made on the Barton and Kjaernsli strength function for the Equivalent Roughness parameter to consider a range of probable rockfill characteristics.
14.4.2 Properties for the dynamic analysis

14.4.2.1 Shear modulus

The small strain shear modulus ($G_{\text{max}}$) is defined in the model according to the function and factors presented in Jia & Chi (2012). Figure 14.2 below presents the calculated $G_{\text{max}}$ vs effective stress function adopted for the Waimea Dam dynamic analyses.
Figure 14.2: Adopted small strain shear modulus function for Waimea Dam rockfill.

Figure 14.3 below presents a function that can be used to calculate $G_{\text{max}}$ from measured shear wave velocity. When the actual shear wave velocity is measured in the final embankments this can be used to derive $G_{\text{max}}$ and compared with the function used (displayed in the above figure).

Figure 14.3: Adopted small strain shear modulus to shear wave velocity function for Waimea Dam rockfill.

The shear modulus reduction function ($G/G_{\text{max}}$) used for the rockfill is presented in Figure 14.4 below. The function was set to the best fit line for the testing data presented in Jia & Chi (2012) (also presented in Figure 14.4).
14.4.2.2 Damping ratio

The damping ratio function used for the rockfill, $\xi$, is presented in Figure 14.5 below. The function was set to the best fit line for the testing data presented in Jia & Chi (2012) (also presented in Figure 14.5).

Figure 14.4: Adopted rockfill shear modulus reduction function (background reproduced from Figure 1 in Jia & Chi (2012)).

Figure 14.5: Adopted rockfill damping ratio function (background reproduced from Figure 2 in Jia & Chi (2012)).
14.5 Input motions/time histories

The selected earthquake records/time histories/input motions and scaling factors used for the dynamic analyses are summarised below in Table 14.2. These records were selected on the basis of the 2011 seismic hazard assessment (GNS, 2011) and reconfirmed as being suitable based on the procedure for selecting ground motions outlined in NZS 1170.5. Further details on selection of these records is provided in Section 2.5.

Table 14.2: Adopted time histories/accelerograms

<table>
<thead>
<tr>
<th>Accelerogram</th>
<th>Period of interest, T (s)</th>
<th>Primary component</th>
<th>Secondary component</th>
<th>k1*k2</th>
<th>OBE</th>
<th>SEE</th>
<th>Aftershock</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Centro</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.4</td>
<td>S00E</td>
<td>N90W</td>
<td>0.57</td>
<td>1.75</td>
<td>1.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>S00E</td>
<td>N90W</td>
<td>0.48</td>
<td>1.46</td>
<td>0.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>S00E</td>
<td>N90W</td>
<td>-</td>
<td>1.26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abbar Iran</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.4</td>
<td>S22W</td>
<td>N68W</td>
<td>N/A</td>
<td>1.11</td>
<td>0.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>S22W</td>
<td>N68W</td>
<td>N/A</td>
<td>1.16</td>
<td>0.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>N68W</td>
<td>S22W</td>
<td>N/A</td>
<td>1.11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Izmit Turkey</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.4</td>
<td>N90E</td>
<td>S00E</td>
<td>0.79</td>
<td>2.43</td>
<td>1.72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>N90E</td>
<td>S00E</td>
<td>0.76</td>
<td>2.34</td>
<td>1.59</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>N90E</td>
<td>S00E</td>
<td>-</td>
<td>2.16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tabas Iran</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>0.4</td>
<td>N16W</td>
<td>N74E</td>
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<td>0.61</td>
<td>0.43</td>
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</tr>
<tr>
<td>0.5</td>
<td>N16W</td>
<td>N74E</td>
<td>N/A</td>
<td>0.59</td>
<td>0.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>N16W</td>
<td>N74E</td>
<td>N/A</td>
<td>0.53</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

14.6 Seepage modelling

14.6.1 General

The completed dam embankment may be subject to varying degrees of flow-through (seepage through the permeable rockfill) during its operating design life. The modelled situations where flow-through could occur are:

- Case 1 - During construction, when the embankment is complete but prior to construction of the concrete face slab with a significant flood occurring that results in impoundment during routing.
- Case 2 – During usual operation with reservoir at NTWL.
- Case 3 – During operation, following a significant earthquake. The concrete face slab and/or plinth may crack and leak.
- Case 4 – During a Probable Maximum Flood.

The potential impact of flow-through has been estimated by seepage modelling, and subsequent stability modelling. This section describes how the magnitude of flow-through has been estimated and the assessed effects on embankment stability.

The stability analysis (refer Section 14.9) show the embankment remains stable under the flow-through scenarios considered.
### 14.6.2 Embankment permeability characteristics

The permeability characteristics of the construction materials have been estimated based on:

- Published relationships and data
- Data gathered from the site investigations and testing
- Laboratory testing
- Trial embankment construction and testing

The placement of rockfill in the embankment during construction typically results in segregation of coarse and fine particles in each layer (Janson, 1981) such that horizontal permeability is higher than vertical permeability. This is often referred to as ‘anisotropy’. Fell et al. (2005) recommends that all earthfill embankments should be designed on the assumption that the ratio of horizontal permeability to vertical permeability is 15 or higher. For large or more sensitive dams, Fell recommends that they are designed such that embankment stability is not sensitive to the ratio. Cruz (2009) describes the ratio of horizontal permeability to vertical permeability is 10. For this assessment, we have carried out modelling with the anisotropy within the main embankment zones ranging from 10 to 15.

Table 14.3 below presents the material parameters that were adopted early in the design phase in order to carry out the seepage analyses. The anticipated embankment performance was assessed using the permeability parameters presented in Table 14.4 below, and with consideration of the potential effects of Zone 4 providing drainage.

#### Table 14.3: Adopted embankment zone permeability characteristics for seepage analyses

<table>
<thead>
<tr>
<th>Material</th>
<th>Estimated saturated water content</th>
<th>Permeability (m/s)</th>
<th>Anisotropy (k_h/k_v)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Base</td>
<td>Lower permeability</td>
</tr>
<tr>
<td>Concrete face</td>
<td>0.001</td>
<td>1x10⁻⁸</td>
<td>1x10⁻⁸</td>
</tr>
<tr>
<td>Zone 2B Face support</td>
<td>0.35</td>
<td>5x10⁻³</td>
<td>5x10⁻³</td>
</tr>
<tr>
<td>Zone 2C gravel</td>
<td>0.35</td>
<td>1x10⁻³</td>
<td>1x10⁻⁴</td>
</tr>
<tr>
<td>Zone 3A, 3B, 3C, 3D rockfill</td>
<td>0.35</td>
<td>1x10⁻³</td>
<td>1x10⁻⁴</td>
</tr>
<tr>
<td>Foundation rock</td>
<td>0.05</td>
<td>2.6x10⁻⁶</td>
<td>2.6x10⁻⁶</td>
</tr>
<tr>
<td>Grout curtain</td>
<td>0.05</td>
<td>2.6x10⁻⁷</td>
<td>2.6x10⁻⁷</td>
</tr>
</tbody>
</table>

#### Table 14.4: Anticipated embankment zone permeability characteristics

<table>
<thead>
<tr>
<th>Material</th>
<th>Estimated saturated water content</th>
<th>Anticipated permeability (m/s)</th>
<th>Anisotropy (k_h/k_v)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete face</td>
<td>0.001</td>
<td>1x10⁻⁸</td>
<td>1</td>
</tr>
<tr>
<td>Zone 2B Face support</td>
<td>0.35</td>
<td>5x10⁻⁴</td>
<td>15</td>
</tr>
<tr>
<td>Zone 2C gravel</td>
<td>0.35</td>
<td>1x10⁻²</td>
<td>15</td>
</tr>
<tr>
<td>Zones 3A and 3B rockfill</td>
<td>0.35</td>
<td>1x10⁻³</td>
<td>15</td>
</tr>
<tr>
<td>Zones 3C and 3D rockfill</td>
<td>0.35</td>
<td>1x10⁻²</td>
<td>15</td>
</tr>
<tr>
<td>Zone 4 drainage</td>
<td>0.35</td>
<td>1x10⁻²</td>
<td>15</td>
</tr>
<tr>
<td>Foundation rock</td>
<td>0.05</td>
<td>2.6x10⁻⁶</td>
<td>1</td>
</tr>
</tbody>
</table>
14.6.3 Embankment flow-through estimates

Estimates of potential flow, and of the exit level of seepage on the downstream face have been made using the software package Seep/W. In cases 1 to 3 the reservoir is assumed to be at NTWL (197.2 m RL).

Case 1 represents the construction flow-through case and assumes the concrete face has not yet been constructed and a large flood filled the reservoir up to 197.2 m RL. The assumed construction sequencing means that the Zone 2B (face support zone) is would be protected by the extruded concrete kerbs (which are installed progressively with each embankment lift/layer). However, the Seep/W modelling does not account for the concrete kerbs assumes Zone 2B acts at the primary seepage control (Zone 2B is designed to have a lower permeability than the rockfill zones).

Case 2 represents the normal operation of the dam with minor seepage entering the dam from under the foundations and abutments. Case 2 is presented in Figure 14.6 below as an example of the Seep/W outputs.

![Figure 14.6: Seep/W model example showing base case seepage under usual operation (Case 2).](image)

Case 3 represents the post-earthquake flow-through case (e.g. following the SEE) (refer Figure 14.7 below). In this case the permeability of the concrete face only was adjusted to be 100x more permeable than the value presented in Table 14.3 to approximate a damaged concrete face.
The Seep/W modelling results are summarised in Table 14.4 below. Of note is that the seepage may be used to offset residual flows required to be released through the dam outlet works.

### Table 14.4: Seep/W embankment flow-through results

<table>
<thead>
<tr>
<th>Case</th>
<th>Permeability</th>
<th>Flow-through</th>
<th>Seepage exit elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1, construction case</td>
<td>Lower permeability (Table 14.3)</td>
<td>1.1 l/s per m width</td>
<td>193 m RL</td>
</tr>
<tr>
<td></td>
<td>Base (Table 14.3)</td>
<td>10.9 l/s per m width</td>
<td>193 m RL</td>
</tr>
<tr>
<td></td>
<td>Higher permeability (Table 14.3)</td>
<td>104.7 l/s per m width</td>
<td>193 m RL</td>
</tr>
<tr>
<td></td>
<td>Anticipated permeability (Table 14.4)</td>
<td>12.8 l/s per m width</td>
<td>188 m RL</td>
</tr>
<tr>
<td>Case 2, operational case</td>
<td>Lower permeability</td>
<td>0.11 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
<tr>
<td></td>
<td>Base</td>
<td>0.13 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
<tr>
<td></td>
<td>Higher permeability</td>
<td>0.14 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
<tr>
<td></td>
<td>Anticipated permeability</td>
<td>0.12 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
<tr>
<td>Case 3, post-earthquake case</td>
<td>Lower permeability</td>
<td>0.9 l/s per m width</td>
<td>185 m RL</td>
</tr>
<tr>
<td></td>
<td>Base</td>
<td>3.1 l/s per m width</td>
<td>165 m RL</td>
</tr>
<tr>
<td></td>
<td>Higher permeability</td>
<td>4.9 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
<tr>
<td></td>
<td>Anticipated permeability</td>
<td>3.9 l/s per m width</td>
<td>151 m RL (seepage collection system)</td>
</tr>
</tbody>
</table>

### 14.6.4 Construction cases

#### 14.6.4.1 Unravelling of the downstream face

The potential for seepage to cause unravelling of the downstream face was investigated during Stage 3 for the construction case (Case 1) described in Section 14.8 using methods developed by Olivier (1967), Stephenson (1979) and Solvik & Skoglund (1995). With an unprotected rockfill face
unravelling was estimated to occur between 166.8 and 173.3 m RL. The proposed meshing to constrain the rockfill was set at a height of 173.4 m RL is based on this Stage 3 analyses.

GHD reviewed the precedent based Stage 3 reinforced rockfill design and amended this as part of the Stage 4 temporary works design, as presented in their design report titled “Waimea Community Dam Reinforced Rockfill Design” dated June 2018. These changes included raising the reinforcement level to 176.4 m RL, adjusting the rockfill zone extents and anchor bar arrangements, and increasing the anchor bar lengths. We understand the reinforcement level was raised for the increased diversion culvert height.

GHD undertook static slope stability analysis using Seep/W and Slope/W that accounted for the reinforcement. The adopted rockfill parameters in the temporary works design are within the design envelopes for the permanent works design (as per Section 14.2). The presented slope stability FOS for the critical failure surface under overtopping conditions was 1.44.

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

The unravelling of the quick-rise berm under flow-through was assessed during Stage 3 by the same methods used for the reinforced rockfill. Based on an adopted hydraulic conductivity value of $1 \times 10^{-2}$ m/s a D$_{50}$ of at least 90 mm was assessed as being required for stability during Stage 3. No slope stability analysis was undertaken during Stage 3 for the quick rise berm.

The required grading for the quick rise berm to withstand flow through is larger than the Zone 3B material. The actual grading of this material is subject to the temporary works arrangements adopted, noting inclusion of a downstream riprap zone may enable the Zone 3B material to be used. The adopted material grading for the quick rise berm should be compatible with surrounding Zone 3B material.

The location and dimensions of the quick rise berm should be confirmed as part of the temporary works design including slope stability analyses.

14.7 Static stability results

14.7.1 General

The static stability of the embankment was assessed using the material properties outlined in Section 14.2, the phreatic/piezometric seepage water surface in the dam (from the seepage modelling described in Section 14.6 above), and the limit equilibrium method. The software package Slope/W was used to model the embankment stability.

The global stability of the downstream face was also specifically assessed for the design cases where the significant seepage flows-through the embankment. The most onerous static stability case identified is the post earthquake case (e.g. as the result of damage to concrete face and/or plinth following the SEE), where embankment seepage to exits the downstream face at 165 m RL (refer Section 14.6 above).

A separate assessment of the embankment static stability during construction was undertaken during Stage 3 and this is summarised briefly in Section 14.6.4. The temporary works stability assessment for Stage 4 was undertaken by FHTJV’s temporary works designer (GHD) and is reported separately.

14.7.2 Usual operation static stability

During usual operation of the dam (reservoir at NTWL), the base analysis seepage flow rate is 0.08 l/s/m and the phreatic surface is at approximately 152 m RL. In this case the minimum slope stability factor of safety is 1.58, which is greater than the criteria of 1.5 (as per the NZSOLD Guidelines 2015).
The Design Criteria Report included a target seepage rate of less than 100 l/s.

### 14.7.3 IDF static stability

During the IDF (reservoir at 202.53 m RL), the base design seepage flow rate is 0.11 l/s/m and the phreatic surface is at approximately 156.6 m RL at the downstream shoulder. In this case the minimum slope stability factor of safety is 1.68 (for the average rockfill material properties) which is greater than the usual criteria of 1.5 (as per the NZSOLD Guidelines 2015).

### 14.7.4 Construction flood static stability

For the construction case where the embankment is complete but the concrete face is not yet installed, the base design seepage flow rate is 10.9 l/s/m and the seepage exits the downstream face at approximately 193 m RL. In this case the minimum slope stability factor of safety is approximately 1.2.

### 14.7.5 Post-earthquake static stability

The static stability of the embankment under the full effect of flow through (e.g. following a large earthquake where damage to the concrete face occurs) was assessed. This assessment assumes the reservoir remains at NTWL (197.2 m RL) (i.e. prior to any intervention to drawdown the reservoir), and that seepage flows are as per the base permeability case. The results of this assessment are summarised in Table 14.5 below.

<table>
<thead>
<tr>
<th>Case</th>
<th>Stability criteria</th>
<th>FOS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static stability (Base case rockfill strength)</td>
<td>FOS &gt;1.2 to 1.3</td>
<td>1.47</td>
</tr>
<tr>
<td>Static stability (Lower bound rockfill strength)</td>
<td>FOS &gt;1.2 to 1.3</td>
<td>1.41</td>
</tr>
<tr>
<td>Static stability (Upper bound rockfill strength)</td>
<td>FOS &gt;1.2 to 1.3</td>
<td>1.58</td>
</tr>
</tbody>
</table>

The static stability post-earthquake complies with the NZSOLD Guidelines 2015 and is considered to be satisfactory based on this assessment.

### 14.7.6 Summary

The results of the static stability analyses are summarised in Table 14.6 below. All FOS meet the design criteria and are greater than 1.5 for static, 1.3 for the aftershock static case.

<table>
<thead>
<tr>
<th>Rockfill strength (1)</th>
<th>Permeability (2)</th>
<th>Usual case static FoS</th>
<th>IDF case static FoS</th>
<th>Aftershock static FoS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower bound (R = 6)</td>
<td>Base</td>
<td>1.58</td>
<td>-</td>
<td>1.41</td>
</tr>
<tr>
<td>Base (R = 7)</td>
<td>Base</td>
<td>1.72</td>
<td>1.68</td>
<td>1.47</td>
</tr>
<tr>
<td>Upper bound (R = 8)</td>
<td>Base</td>
<td>1.84</td>
<td>-</td>
<td>1.58</td>
</tr>
</tbody>
</table>

(1) Rockfill strength is as per shear normal functions presented in Section 14.2.
(2) The permeability scenarios considered are presented in Section 14.6.
14.8 Seismic performance

14.8.1 General

The stability of the dam during the design seismic case has been assessed primarily using pseudostatic and empirical methods. The embankment seismic stability was modelled in Slope/W to obtain the yield accelerations (i.e. where minimum factors of safety are <1.0) which were then compared to the empirically derived design crest accelerations. Where the design crest accelerations exceed the modelled yield accelerations, further assessment was undertaken to quantify the potential extent of displacement at the crest.

The potential extent of displacement at the crest is a key consideration as this may affect the residual freeboard, parapet wall stability and post-earthquake performance of the dam.

14.8.2 Embankment yield accelerations

The Slope/W modelling undertaken to determine the embankment yield accelerations used the material properties summarised in Section 14.2. The method of analysis for the pseudostatic modelling was Morgenstern-Price.

The hydrodynamic forces from the impounded water have been calculated via the method presented in Chwang & Housner (1977) for water against a sloping face. The force has been applied as a series of point loads in Slope/W up the upstream embankment face at approximately 2.5 m vertical centres. The following assumptions have been made for the calculation and application of the hydrodynamic force:

- The hydrodynamic force has been calculated using the crest acceleration of the dam. This is a conservative assumption.
- The hydrodynamic forces are constant throughout the earthquake, i.e. the peak hydrodynamic force is applied at the same time as the peak downstream acceleration in the embankment. This is a conservative assumption.

14.8.3 Design crest accelerations

During an earthquake, fill embankments typically amplify the horizontal earthquake ground motions such that at the crest, accelerations can be significantly higher than at the base. Simplified calculations for the expected peak crest accelerations were undertaken for the design embankment geometry based on the mean unweighted site spectra provided by GNS, and the calculation method described by Makdisi & Seed (1979). Three values for small strain shear modulus ($G_{\text{max}}$) were used to assess the impact of $G_{\text{max}}$ on the resulting crest accelerations. The results are presented in Table 14.7 below.

Table 14.7: Estimated crest accelerations (as per Makdisi & Seed, 1979)

<table>
<thead>
<tr>
<th>Small strain shear modulus ($G_{\text{max}}$) (MPa)</th>
<th>OBE (foundation PGA(0) = 0.17g)</th>
<th>SEE (foundation PGA(0) = 0.64g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Damping (%)</td>
<td>Strain (%)</td>
</tr>
<tr>
<td>300</td>
<td>18.4%</td>
<td>0.172</td>
</tr>
<tr>
<td>400</td>
<td>17.3%</td>
<td>0.138</td>
</tr>
<tr>
<td>500</td>
<td>16.0%</td>
<td>0.112</td>
</tr>
</tbody>
</table>
14.8.4 Seismic deformation/displacements

Earthquake ground motion may result in permanent deformation of the embankment, with the extent of deformation relative to the earthquake size (the ground acceleration and magnitude) and embankment geometric and material properties.

Permanent deformations of the dam embankment have been assessed using a range of empirical methods (based on recorded seismic displacements). These methods use the yield acceleration, crest acceleration and earthquake magnitude to estimate the displacements. We have utilised the following methods in our calculation:

- Bray and Travasarou (2007), which is generally considered to be an appropriate method for assessment of seismic displacements for embankment dams.
- Makdisi & Seed + Rathje et al (vector method) to assess the sensitivity of the results to the effects of a flexible sliding block.

The procedure is based on simple limit equilibrium analysis methods and is typically carried out to assess whether significant deformations may be sustained by an embankment, and whether more detailed analyses are required to assess them.

The yield acceleration is that acceleration that is just large enough to result in development of small permanent deformations within the embankment. When actual accelerations exceed the yield acceleration, deformations increase with increasing acceleration. The empirical methods used are based on ‘Newmark sliding block’ type analysis of earthquake records that are commonly used to evaluate displacements due to earthquake shaking.

The embankment yield accelerations have been estimated using a limit equilibrium slope methodology implemented in the stability program Slope/W (refer Section 14.8.3). This method assumes that in an earthquake slips will form that are discrete masses of soil that move in isolation from material below on a slip surface (a “slip”). In reality, the material deforms in a wide zone, and hence these analyses provide an estimate of maximum displacement along the theoretical slip plane, not an estimate of the distribution of displacements within the embankment.

The adopted earthquake magnitudes used in the empirical analysis are 7.2 M\text{w} (OBE), 7.2 M\text{w} (mean estimate SEE), 6.8 M\text{w} (aftershock) based on GNS (2017).

Potential slip/failure surfaces have been assessed at various depths within the embankment, in order to compare the yield acceleration for the particular assumed slip geometry with the maximum average embankment acceleration (which itself varies with depth). Only those theoretical slip surfaces that encompass the dam crest (and hence relate to deformation with the potential to compromise water retention) have been considered. An example failure surface is presented in Figure 14.8 below.
Figure 14.8: Dam embankment showing an example failure surface (red zone on downstream face) as considered in seismic displacement assessments.

A summary of the results are presented in Table 14.8 below and show the potential range of displacements for the empirical methods considered.

Table 14.8: Summary of pseudostatic displacement results

<table>
<thead>
<tr>
<th>Rockfill strength (R)</th>
<th>Function exceedance probability (2)</th>
<th>Coincident vertical action</th>
<th>Maximum slip circle displacement (for a slip that would affect the crest) (3)</th>
<th>Aftershock (GNS recommendation – M6.8 Waimea South and Alpine)</th>
<th>Aftershock (EGL suggestion – M6.5 Waimea Central South)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower bound (R = 6)</td>
<td>84% (Mean + 1 standard deviation)</td>
<td>30% k_v</td>
<td>&lt;10 mm (4)</td>
<td>B&amp;T = 430 mm M&amp;S+R&amp;A = 360 mm M&amp;S+J = 570 mm</td>
<td>B&amp;T = 190 mm M&amp;S+R&amp;A = 180 mm M&amp;S+J = 110 mm</td>
</tr>
<tr>
<td>Base (R = 7)</td>
<td>50% (Mean)</td>
<td>0% k_v</td>
<td>&lt;10 mm (4)</td>
<td>B&amp;T = 120 mm M&amp;S+R&amp;A = 150 mm M&amp;S+J = 70 mm</td>
<td>B&amp;T = 40 mm M&amp;S+R&amp;A = 90 mm M&amp;S+J = 30 mm</td>
</tr>
<tr>
<td>Upper bound (R = 8)</td>
<td>16% (Mean - 1 standard deviation)</td>
<td>30% k_v</td>
<td>0 mm (4)</td>
<td>B&amp;T = 70 mm M&amp;S+R&amp;A = 140 mm M&amp;S+J = 50 mm</td>
<td>B&amp;T = 20 mm M&amp;S+R&amp;A = 40 mm M&amp;S+J = 10 mm</td>
</tr>
</tbody>
</table>

(1) Rockfill strength is as per shear normal functions presented in Section 14.2.
(2) The exceedance probabilities relate to the methods used. The combination of rockfill strength and coincident vertical seismic actions may give results that are of higher or lower probability.
(3) Method abbreviations refer to B&T (Bray & Travasarou, 2007), M&S+R&A (Rathje & Antonakos (2011) method for a flexible sliding mass with average slip maximum ground accelerations from Makdisi & Seed (modified as per Rathje & Antonakos), and M&S+J (Jibson (2007) with average slip maximum ground accelerations from Makdisi & Seed).
(4) High probability of no displacement.
The results presented in Table 14.8 include sensitivity analysis checks. The three rockfill strength cases have assigned exceedance probabilities of failure as described below:

- **Lower bound (Reasonably conservative estimate).** This consists of the result of the calculation process using reasonably conservative soil properties, but not extremely conservative properties. It uses the 84th percentile results of slip displacement prediction method (i.e. 84% of the distribution results would exhibit a lower displacement than the calculated value, 16% would be higher).

- **Base (Middle of the range estimate).** This consists of the result of the calculation process using middle of the range soil properties, but not extremely conservative properties. It uses the 50th percentile results of slip displacement prediction method (i.e. 50% of the distribution results would exhibit a lower displacement than the calculated value, 50% would be higher).

- **Upper bound (Reasonably non-conservative estimate).** Reasonably non-conservative soil properties, but not extremely non-conservative properties. It uses the 16th percentile results of slip displacement prediction method (i.e. 16% of the distribution results would exhibit a lower displacement than the calculated value, 84% would be higher).

GNS (2017) describe that following an SEE event, an aftershock might be expected at one magnitude less than the main shock. We have assessed the aftershock stability on the basis of a 6.8 Mw event (based on 84th percentile SEE of 7.8 Mw). The aftershock water table has been taken from a Seep/W model for the higher permeability case where the concrete facing permeability has been increased 100x.

EGL undertook a methodology review of the GNS work (refer attached letter in Appendix F) and identified that the aftershock recommended by GNS (M6.8 Waimea South and Alpine), while one order of magnitude less than the SEE, does not give the highest aftershock spectral accelerations of the three cases outlined by GNS. EGL recommended that the M6.5 Waimea Central–South aftershock also be considered (this gives a peak ground acceleration at C(0) of 0.58g which is approximately 32% larger than the M6.8 aftershock at 0.44g). We have run another displacement sensitivity check using the M6.5 aftershock and this gives displacements within the acceptable range also as presented in Table 14.9 above.

It should be noted that the aftershock scenario adopted is considered (appropriately) conservative. It is possible that such an aftershock could occur well before steady state flow through develops in the embankment, and equally that such an aftershock occurs after the reservoir has been lowered below the NTWL assumed in the analysis. In both these cases, displacements estimated would be less than those reported in Table 14.8.

### 14.8.5 Risks / Uncertainties

Two key areas of uncertainty remain in the inputs for the seismic performance analysis. These relate to the assumed material properties and are:

- The rockfill strength (shear normal function).

- The shear wave velocity through the embankment (used to obtain crest accelerations in the Makdisi & Seed (1979) method).

Sensitivity analyses were undertaken to quantify the effects of a range of rockfill strengths, and shear wave velocities on the crest accelerations and slope displacements.
14.9 Dynamic analysis

14.9.1 Overview

The design approach was to assess the potential embankment displacements due to seismic loadings based on pseudo-static analyses and empirical methods with sensitivity analyses to obtain a range of displacement estimates (as summarised in Table 14.8 above).

The dynamic analysis was then performed as a check for the SEE design case with coincident 30% vertical acceleration. The purpose of the dynamic analysis is to provide another check to confirm the range of seismic displacements calculated using the pseudostatic and empirical methods.

The following sections describe the equivalent linear dynamic Quake/W model and the modelling results for embankment stability and deformation. The models have been run for the Safety Evaluation Earthquake (SEE) event only.

14.9.2 Model description

The Quake/W model uses the same embankment zones as the static/pseudostatic Slope/W model with additional material characteristics and seismic load parameters added for the equivalent linear dynamic time history analysis (as described in Section 14.3 above). Equivalent linear dynamic analysis is considered appropriate for this application given it use a check of the pseudostatic/empirical methods.

14.9.3 Results and comparison

The estimated displacements determined using the equivalent linear dynamic time history analyses for the SEE event (100% horizontal with coincident 30% vertical actions) are presented in Table 14.9 below. The results of the dynamic analysis fall within the range of displacements calculated using the pseudostatic and empirical methods, albeit suggesting that the ‘Middle of the range estimate’ to the ‘Reasonably conservative estimate’ are more representative.

Horizontal displacements in the downstream direction in the order of 300 to 400 mm are predicted from the analysis, and in the context of the Waimea Dam (which features a 6 m wide crest and a parapet wall founded on rockfill approximately 1.9 m above the NTWL) this level of displacement can be accommodated without resulting in catastrophic failure of the dam. Following large earthquake events, it is expected that the reservoir will be drawn down as a precautionary measure to reduce the risk of seepage occurring near the crest (especially should a large flood occur after the SEE).

The corresponding vertical displacements from the analysed slope failure surfaces are expected to be approximately in the order of 70 - 240 mm (based on slip direction parallel to downstream slope) and therefore the corresponding loss of freeboard following an SEE is considered very minor (especially as the top of the parapet wall is approximately 5.9 m above the NTWL).

Table 14.9: Estimated horizontal displacements from SEE for dynamic analysis

<table>
<thead>
<tr>
<th>Type of analysis</th>
<th>Earthquake record used</th>
<th>Seismic actions</th>
<th>Displacement (assuming T=0.5s)</th>
<th>Displacement (assuming T=0.6s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent linear dynamic time history</td>
<td>El Centro, Imperial Valley, S00E</td>
<td></td>
<td>220 mm</td>
<td>220 mm</td>
</tr>
<tr>
<td></td>
<td>Abbar, Iran, S22W/N68W</td>
<td>Scaled k_v &amp; k_h from records both included</td>
<td>250 mm</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>Izmit, Turkey N90E</td>
<td></td>
<td>330 mm</td>
<td>270 mm</td>
</tr>
<tr>
<td></td>
<td>Tabas, Iran, N16W</td>
<td></td>
<td>360 mm</td>
<td>360 mm</td>
</tr>
</tbody>
</table>
14.10 **Settlement**

14.10.1 **General**

Rockfill embankments typically settle and compress during construction (under self weight), upon commissioning (at first fill of the reservoir) and in the longer term (consolidation under load). The extent of settlement depends on the elastic modulus achieved for the rockfill, and the estimated total settlement informs selection of the dam crest level (including precamber). Limiting the extent of settlement under the concrete face is also important for reducing the potential for cracking of the face and undue seepage.

14.10.2 **Methodology**

Settlements have been estimated for the Waimea Dam using the empirical Fell & Hunter (2003) method. This method provides estimates for construction, first filling and long term settlements based on the embankment and reservoir heights, rockfill strength classification (i.e. medium to high strength rockfill, or high strength rockfill).

The rockfill parameters assumed in this assessment are:

- $D_{80}$ particle size equal to 150 mm based on a $D_{50} = 35$ mm size (as informed by the site investigations).
- Unit weight of rockfill equal to 2.2 t/m$^3$.

14.10.3 **Settlement during construction**

The expected rockfill settlement during construction was estimated as follows:

- Using Figure 5.2 presented in Hunter & Fell (2003), the representative $E_r$ (MPa) at the end of construction for medium to high strength rockfill was calculated to be 51.8 MPa.
- The $E_r$ was then factored to account for vertical stresses due to overburden give an equivalent $E_{rcc}$.
- To account for medium strength rockfill, a 0.7 reduction factor was applied to medium to strong strength rock fill.
- The $E_r$ of medium and medium to strong rockfill was calculated for first fill using an $E_r/E_{rcc}$ ratio presented in Figure 5.4, Hunter & Fell (2003).

The calculated vertical settlement during construction for layers of 10 m heights were calculated for both medium and medium to high strength rockfill as presented in Table 14.10 below.

<table>
<thead>
<tr>
<th>Fill total depth</th>
<th>Total Settlement (m) – Medium to high strength rockfill</th>
<th>Total Settlement (m) – Medium strength rockfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 m</td>
<td>0.04</td>
<td>0.06</td>
</tr>
<tr>
<td>20 m</td>
<td>0.08</td>
<td>0.11</td>
</tr>
<tr>
<td>30 m</td>
<td>0.12</td>
<td>0.17</td>
</tr>
<tr>
<td>40 m</td>
<td>0.16</td>
<td>0.22</td>
</tr>
<tr>
<td>50 m</td>
<td>0.06</td>
<td>0.09</td>
</tr>
</tbody>
</table>
14.10.4  Post construction crest settlement

14.10.4.1  First Fill

The crest settlement (as a % of embankment height) was found for both medium and medium to high strength rock at first fill using Table 4 presented in Hunter & Fell (2003). These values are representative of first filling alone. They neglect time dependent deformations.

Tangential deformations at first fill were calculated at 10 m lifts for both medium and medium to high strength rockfill are presented in Table 14.11 below:

Table 14.11: Estimated tangential settlement/compression during first filling

<table>
<thead>
<tr>
<th>Elevation (m RL)</th>
<th>Total Settlement (m) – Medium to high strength rockfill</th>
<th>Total Settlement (m) – Medium strength rockfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>197.2</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>187.2</td>
<td>0.06</td>
<td>0.08</td>
</tr>
<tr>
<td>177.2</td>
<td>0.09</td>
<td>0.12</td>
</tr>
<tr>
<td>167.2</td>
<td>0.08</td>
<td>0.12</td>
</tr>
<tr>
<td>157.2</td>
<td>0.05</td>
<td>0.07</td>
</tr>
<tr>
<td>147.2</td>
<td>0.002</td>
<td>0.002</td>
</tr>
</tbody>
</table>

14.10.4.2  Long term

The long term rate of crest settlement post construction (as a % or embankment height) was found for both medium and medium to high strength rock at first fill using Figure 5.5 presented in Hunter & Fell (2003).

The results for post construction crest settlement behaviour (based on a percentage of embankment height and total settlement) are presented in Table 14.12 below. For a 100 year design life, the estimated long term crest settlement is between 160 to 330 mm. Given the logarithmic scale this means most of the settlement is estimated to occur in the first 10 years.
Table 14.12: Long term crest settlement

<table>
<thead>
<tr>
<th>Intact Strength</th>
<th>Crest Settlement (% of H)</th>
<th>Settlement (mm) per log cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium Strength Rock</td>
<td>0.20</td>
<td>110</td>
</tr>
<tr>
<td>High Strength Rock</td>
<td>0.10</td>
<td>53</td>
</tr>
</tbody>
</table>

Clements (1984) also estimates long term post construction crest settlement for compacted rockfill. Though Clements does not give direction to the strength of rock the parameters relate to, it is assumed that medium to high strength rock fill was used to derive the equation. Using Clements (1984) guide to long term settlement, it was found that for compacted rockfill the initial impounding was 20 mm and the 10 year service settlement was 40 mm.

14.10.5 Earthquake induced settlements

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

The calculated Earthquake Severity Index (ESI) for the Waimea Dam OBE and SEE cases, are 3.3 and 12.6 respectively. These are based on a magnitude 7.2 earthquake for both the OBE and SEE events as presented in the GNS report (2017). Figure 14.10 presents the calculated ESI’s compared with performance data presented by ICOLD (2010), Swaisgood (2003) and Bureau et al (1985).

Swaisgood’s published empirical equation was used to estimate the amount of deformation to be expected as a result of the OBE and SEE events. These expected deformations are as follows:

- OBE relative settlement of 0.06%, or < 30 mm.
- SEE relative settlement of 0.99%, or < 530 mm.

It is worth noting that the deaggregation of the Waimea Dam hazard shows nearly 80% of the contribution to the OBE and SEE hazard comes from events of lower magnitude than the 7.2 for which the above ESI’s are calculated. The mean magnitude of the contributions to the PGA hazard ranges from about 6.3 to 6.5 for return periods from 150 years to 10,000 years (GNS, 2017).

The OBE foundation PGA is 0.17g and the SEE foundation PGA is 0.64g as per GNS 2017. If the average magnitude presented by GNS is used to calculate the OBE ESI, the value reduces down to approximately 1.
Figure 14.10: Earthquake induced deformations of rockfill dams.

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

The New Zealand Dam Safety Guidelines (NZSOLD, 2015) allow for some minor repairable damage at OBE level shaking. Cruz et al (2010) recommend the OBE design criteria as damage that can be repaired whilst the dam is still operational. Thus, for the Waimea Dam we have not endeavoured to eliminate the risk of minor damage at OBE level shaking. Based on the published information available, the expected Waimea Dam OBE and SEE deformations reported above are considered to be within tolerable limits for a CFRD dam.

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

14.11 Discussions and conclusions

The permanent deformations estimated are of an order that would not be expected to compromise the dam function at the OBE level. The displacements estimated (10 to 35 mm maximum) would be expected to be accommodated by the dam structure and result in little, if any, significant damage.

At the SEE level event, the permanent deformations estimated (300 to 500 mm maximum) would likely contribute to damage to the embankment structure, with cracking in the dam face, and in the
parapet wall. It is expected that extensive damage of the box culvert would also occur. The damage associated with the permanent deformations would not be expected to be sufficient to compromise the required performance of the embankment immediately following the seismic event. They are likely to compromise the performance of the embankment and the dam appurtenant structures such as the spillway and access bridges to the extent that repair, potentially of a very significant nature, would be required for the embankment to remain in service. It is possible that decommissioning of the dam would be required.

It is of note that the SEE event is a devastating earthquake that would cause widespread damage and destruction of homes and infrastructure in the region. It may be many weeks or even months before significant repairs could be undertaken and as such it is likely that should the outlet works be functioning that the dam would require to be dewatered following such a significant event.

The additional permanent deformations that are estimated to result from the adopted aftershock event (a further 100 to 200 mm), are of a magnitude that would not be expected to result in an uncontrolled release of water from the reservoir following an SEE event.

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

15.1 General

The concrete face of the dam provides the primary seepage control for CFRD’s, and features two stage waterstops at the vertical and horizontal contraction joints to control leakage at these interfaces.

Acceptable performance of the concrete face relies on the supporting embankment materials (i.e. the Zone 2B face support and the upstream shoulder rockfill Zones 3A and 3B) being relatively stiff to reduce the deformation under load. Excessive deformation of the supporting embankment can result in stress concentrations, cracking and excessive leakage through the concrete face (refer Sections 13 and 14 for embankment details).

15.2 Design basis

15.2.1 Standards and references

Design of concrete face slabs is based on precedent (from successful operational CFRD’s) rather than specific analysis. For relatively low height CFRD such as the Waimea Dam, the typical concrete face detailing is considered to be conservative as the face is unlikely to operate with significant compressive and tensile stress zones.

The following standards and references informed the adopted concrete face details for the Waimea Dam:

- ANCOLD (1991) “Concrete face rockfill dams”.
- Cooke (1987) “Concrete face rockfill dam”.
- Cruz et al. (2009) “Concrete face rockfill dams”.
- ICOLD Bulletin 141 (2010) “Concrete face rockfill dams”.
- NZS3101 “Concrete structures”.
- International CFRD precedents.

15.3 Description

15.3.1 Concrete thickness and strength

Face slabs for dams less than 100 m in height generally have a uniform thickness of 300 mm and this has been adopted for the Waimea Dam.

Concrete strength is not critical and a 25 MPa 56 day compressive strength (or 21 MPa at 28 days) is typical. Maximum size aggregate of 38 mm, air entrainment and use of flyash (25% of total cementitious material) is standard practice. The concrete mix requirements are covered in the Specification.

15.3.2 Formwork

The concrete face slab is slipformed in a continuous operation from foundation level to parapet base level. Where the plinth level varies across the width of a face slab, particularly on the abutments, starter slabs are constructed using conventional formwork. This provides a horizontal surface for commencement of the slip form operation.
15.3.3 Joints

Vertical contraction joints (i.e. free joints where the reinforcement is not continuous) are typically 12 m to 18 m apart to suit the slip form with 15 m being the most common width. The presented design shows 15 m wide panels noting this is to be confirmed prior to construction to suit the Contractor’s slipform machine.

Manually formed starter slabs are specified on the abutments where the plinth and slab geometry limit the use of a slipform machine (e.g. due to insufficient clearance or layout space). The face slab that supports the intake rails may also require manual forming or alternative cast in insert arrangements to facilitate slipforming.

The starter slab vertical and horizontal joint locations shown on the Drawings (especially at the abutments) are provisional and to be confirmed during construction to suit the adjacent plinth geometry and construction requirements. Joints perpendicular to plinth are specified to provide construction clearance for the adjacent slipformed face slab panels consistent with CFRD practice (e.g. Karahnjukar Dam in Iceland completed in 2007). A minimum provisional joint length of 1.5 m is shown on the Drawings.

Horizontal construction joints (i.e. joints for facilitate construction where the reinforcement is continuous) are allowed for as shown on the Drawings. Construction joints will be required at the top of the starter slabs and otherwise to suit the concrete pour volumes. Horizontal joints are shown on the Drawings at provisional levels to be confirmed by the Contractor and agreed with the Designer during construction.

All construction joint surfaces should be prepared by green cutting (i.e. as per Type B construction joints in NZS3109). It is recommend that the Contractor include sufficient allowance for extra construction joints above those shown on the Drawings to allow for unforeseen circumstances such as flooding during slip forming, equipment failure, concrete delivery delays.

15.3.4 Reinforcement

Typical reinforcement ratios for CFRD face slabs are 0.3% used horizontally and 0.4% vertically. The reinforcement specified for the Waimea Dam concrete face is a single layer of centrally located reinforcement, and specifically:

- 16 mm bars @ 250 mm centres horizontally and 20 mm bars @ 250 mm centres vertically in the central compression slabs, equivalent to 0.33% and 0.41% respectively.

Additional reinforcement is provided for the slab thickening (500 mm thick) at the crest (underneath the parapet wall and crest ramp), two layers of reinforcement are detail in this location to provided additional structural capacity and to confine the vertical tear web type waterbar (which is intended to control seepage between the parapet wall foundation and the concrete face).

Confining reinforcement is provided at the slab adjacent to the perimetric joint to support the central waterbar. This consists of two layers of reinforcement bars bent either side of the waterstop. The cover on the upper hairpin has been reduced to 60 mm to improve the spall resistance of the exposed concrete. While this doesn’t comply with the cover requirements in NZS3101 for 100 year design life, this arrangement is considered preferable given the improved spalling resistance.

The starter slabs are polygon shaped panels due to the perpendicular joints. As with international CFRD precedent, this panel shape does not introduce additional loads to the slab or require additional reinforcement. The starter slabs on the abutments (especially true right) are located on shallow rockfill with limited potential for construction and/or long term settlement, even with the lower bound rockfill strength and modulus analysed as part of the design. Therefore, differential settlement induced loads on the slabs are expected to be well within the flexural capacity of the
slabs. Similarly, tensile loads and movement at the vertical joints due to differential settlement of the rockfill are expected to be within the capacity of the waterstops to accommodate.

Should construction layouts present more acute angles at the interface between the plinth and a starter slab this is likewise not anticipated to result loads exceeding the slab capacity, noting if this remains a concern additional trimmer bars may be added at acute corners to provide additional reassurance.

15.3.5 Perimetric joint

The perimetric joint is the horizontal/inclined joint at the base of the concrete face slab where is abuts the plinth. This joint is a critical interface and careful detailing and construction is essential for achieving adequate performance of the concrete face as a method of water control.

A typical perimetric joint detail used in Australian dams (ANCOLD, 1991) has been adopted for the Waimea Dam as shown on the Drawings. This detail comprises of:

- A copper waterstop located at the rear of the face and supported by a mortar joint pad.
- A PVC centre bulb type waterstop located centrally.
- A compressible joint filler placed on the supporting face area of the plinth to prevent edge concentrations of compressive stress during construction and before first filling due to the rockfill settlement. After first filling the perimetric joint typically opens slightly (reducing the compression on the joint filler but not exceeding the tensile capacity of the waterstops) as the rockfill compresses in the downstream direction.

Recent practice overseas has been to use a water face seal at the perimetric joint, either as a third seal or as a replacement for the PVC centre-bulb type water bar. A water face seal comprising a mastic secured by a PVC or Hypalon membrane has been tried in Australia but proved expensive and difficult to construct. An alternative is to use an Omega EPDM type of joint as shown at Figure 15.1 and this could be adopted as an alternative to the central PVC waterstop if preferred by the Contractor.

![Figure 15.1: External Omega Seal.](image)

15.3.6 Crest termination detail

The detailing of the concrete face at the crest was developed to allow for displacement of the parapet wall relative to the crest only. Significant deformation of the dam crest would likely result in some cracking to the concrete face above the NTWL (based on the analysis described in Section 14).
Repair of the concrete face above NTWL is expected to be relatively achievable (especially when compared to hypothetical repair work lower down near the starter dam for example).

The crest termination detail adopted for Waimea Dam is consistent with international precedents for CFRD in high seismic environments. Specific precedents include the 110m high Potrillos CFRD Dam in Argentina (constructed between 1999 and 2003) designed for a crest acceleration of 1.4g Carmona et al (2004).

15.4 Intake Rails and Fixings

15.4.1 General

The intake pipework and intake screens are supported by 250UC73 rails on the concrete face of the dam as shown below. Rails are attached to the concrete face by steel clamp plates and bolts screwed into cast-in threaded inserts.

15.4.2 Design Basis

Design of rails and fixings is based on loads applied to them from the intakes and screens, and under different load combinations. See methodology section below for load combinations.

The following standards and references have been used for the design of the rails and fixings:

- AS/NZS 1170 “Structural Design Actions”
- NZS 3101 “Concrete Structures”

15.4.3 Design summary

A full summary of the design basis for the intake rail fixing is included in Appendix K.
16 Crest (parapet) wall

16.1 General

The 4 m high parapet wall minimises rockfill volumes while providing an adequate width of rockfill for concrete face slipform operations.

The ramp structure is a U-shaped reinforced concrete box located at the dam western end connecting the upper bridge and the dam crest road and is 29.3 m in length. It provides transition from 204.46 m RL at the upper bridge to 201.53 m RL over the dam crest road.

The crest ramp is founded on excavated rock on its western end transitioning to dam rockfill towards its eastern end.

The wall has a 400 mm thick base slab and a reinforced concrete stem varying in thickness from 425 mm at its base to 300 mm at the top. Precast and cast insitu wall stem options are presented. The wall stems on the upstream and downstream sides are tied together using ReidBar tie rods. The design 28 day concrete strength for the crest ramp is 40 MPa.

The wall stem has contraction joints generally to match the vertical joints in dam face (construction joints only in footing), noting along the true right abutment the starter slab vertical joints may not be regularly spaced and may not align with the 7.5 m long parapet wall stem sections. This is not expected to result in unacceptable performance of the concrete face or parapet wall in this location due to the low differential settlement at the abutment (noting shallow depth of rockfill at abutment).

16.2 Design basis

16.2.1 Standards and references

The following standards and references have been used for the design of the parapet wall:

- MBIE Module 6 (2017) “Earthquake resistant retaining wall design”.
- NZS3101 “Concrete structures”.

16.2.2 Stability

The wall has been designed to resist the estimated wave impact loading from the landslide generated wave outlined in Section 5.4.

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

The seismic design for the crest ramp has adopted the peak ground acceleration used for the parapet wall where the crest ramp is on rockfill. The parapet wall seismic loading is based on amplification at the maximum embankment height and is significantly higher than the seismic design loads for the lower embankment heights on the abutments and the rock.

The crest ramp section that is founded on rock has the same wall thickness and reinforcement as the adjacent section on rockfill to facilitate construction, and the wall design arrangements have been checked for the peak ground acceleration for rock (rather than the maximum embankment height).
The design peak crest accelerations used for the parapet wall stability analyses are derived from the empirical Makdisi & Seed (1979) method as per Table 14.8 in Section 14 above.

The bearing pressures and rockfill capacity underneath the parapet wall were also considered for completeness as per MBIE Module 6 (2017).

**16.2.3 Strength**

The structural design for the parapet wall and crest ramp was undertaken in accordance with NZS3101 and MBIE Module 6 (2017).

The wall stems at the tie rod locations are designed as an integral beam with the tie rods as supports. The rockfill loads are applied to the wall stems using static and pseudostatic methods for the Zone 2B rockfill parameters summarised in Section 14.

Longitudinal (horizontal wall reinforcement) and shear steel designed have been designed for beam actions.

The wall design was checked as being cantilevered for construction loads (compaction pressure from a 12 tonne roller applied to whole height of the wall up to the anchor level) assuming the anchors are not in place.

**16.3 Description**

The parapet wall is approximately 4 m high and has a 4.55 m wide base slab. The width of the base slab has been determined using software GWALL as the minimum width required to prevent overturning during the SEE (i.e. with a factor of safety FOS = 1).

The wall stem has a vertical face and tapers from 350 mm thick at its base to 200 mm at its crest. The stem is designed to be precast or cast in situ in 7.5 m long sections. The vertical joints between the walls feature concrete shear keys with waterstop (precast) or sleeved dowels with PVC water bar (cast in situ).

The wall base is a 400 mm thick slab. The slab extends 550 mm upstream of the stem. The slab will be cast in situ, and sits on the concrete face extension and thickening at the upstream and otherwise on the compacted Zone 2B and Zone 3A rockfill. A vertical tear web shaped water bar is cast into the concrete face extension and the base of the parapet wall to provide seepage control at this interface.

The adopted connection detail between the base of the parapet wall and the concrete face is consistent with international precedents for CFRD in high seismic environments. Specific precedents include the Potrerillos Dam in Argentina (constructed between 1999 and 2003).

**16.4 Stability analyses results**

A summary of the parapet wall stability analyses results is presented in Table 16.1 below. The seismic stability calculations include a duration reduction factor of 0.5 for overturning consistent with pseudostatic retaining wall design (as per MBIE Module 6 and Eurocode guidance).
Table 16.1: Crest (parapet) wall design summary

<table>
<thead>
<tr>
<th>Description</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Static stability</strong></td>
<td></td>
</tr>
<tr>
<td>Sliding FOS</td>
<td>&gt;5.0</td>
</tr>
<tr>
<td>Overturning FOS</td>
<td>&gt;5.0</td>
</tr>
<tr>
<td><strong>Wave impact</strong></td>
<td></td>
</tr>
<tr>
<td>Sliding FOS</td>
<td>&gt;1.0</td>
</tr>
<tr>
<td>Overturning FOS</td>
<td>&gt;1.0</td>
</tr>
<tr>
<td><strong>Seismic stability</strong></td>
<td></td>
</tr>
<tr>
<td>Wall yield acceleration</td>
<td>0.42 – 0.89g (varies to suit vertical acceleration)</td>
</tr>
<tr>
<td>Crest acceleration (OBE)</td>
<td>0.55g (Horizontal)</td>
</tr>
<tr>
<td>Crest acceleration (SEE)</td>
<td>1.90g (Horizontal)</td>
</tr>
<tr>
<td>FOS sliding (SEE)</td>
<td>&lt;1.0</td>
</tr>
<tr>
<td>Calculated sliding during (OBE)</td>
<td>&lt;5 mm</td>
</tr>
<tr>
<td>(Jibson 2007 with 1 standard deviation)</td>
<td></td>
</tr>
<tr>
<td>Calculated sliding during (SEE)</td>
<td>80 mm to 580 mm (30 – 200 mm mean)</td>
</tr>
<tr>
<td>(Jibson 2007 with 1 standard deviation)</td>
<td></td>
</tr>
<tr>
<td>FOS overturning (SEE)</td>
<td>&gt;1.0 (2.1 – 4.6 for seismic load combinations)</td>
</tr>
</tbody>
</table>

The yield acceleration for the wall is lower than the OBE crest acceleration and therefore movement (i.e. sliding of the wall) relative to the foundation is not expected during the OBE. Because the joints have waterbar capable of accommodating approximately 10 – 20 mm vertical or horizontal movement, minor displacement is considered acceptable. After an OBE event (or higher) the joints should be inspected for damage and repaired if necessary.

The wall is calculated to slide during the SEE by between approximately 80 mm and 500 mm as calculated using the (Jibson 2007). This approach ignores coincident displacement of the embankment crest and assumes that the failure surface occurs at the base of the parapet wall. The displacements of the dam crest and parapet wall are not additive (i.e. the wall and embankment will generally displace together).
17 Spillway

17.1 General and background

This section describes the design spillway arrangement and the basis of the design shown on the Drawings. Spillway design standards are presented in Sections 1 and 2 and the Design Criteria Report (T+T, 2011).

The selected spillway arrangement for the Waimea Dam includes the following components:

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

Additional spillway and energy dissipation structure characteristics are presented in Table 17.1.

Table 17.1: Spillway and energy dissipation characteristics

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chute length (plan – Ogee crest to start of flip bucket)</td>
<td>124 m</td>
</tr>
<tr>
<td>Chute width, narrow section</td>
<td>20 m</td>
</tr>
<tr>
<td>Chute maximum grade</td>
<td>2H:1V</td>
</tr>
<tr>
<td>Chute horizontal transition length</td>
<td>71 m</td>
</tr>
<tr>
<td>Chute vertical curve length</td>
<td>21 m</td>
</tr>
<tr>
<td>Chute minimum height of concrete lining</td>
<td>3.0 m</td>
</tr>
<tr>
<td>Dissipation type</td>
<td>Flip Bucket</td>
</tr>
<tr>
<td>Flip bucket radius</td>
<td>20 m</td>
</tr>
<tr>
<td>Bucket lip level</td>
<td>156.6 m RL</td>
</tr>
<tr>
<td>Flip bucket exit angle to the horizontal</td>
<td>40°</td>
</tr>
</tbody>
</table>

17.2 Alternatives considered

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

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

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

17.3.1 Standards and references

The following standards and references have been used for the design of the spillway:

- Damle (1966).
- International precedents (including Tillegra Dam and Warragamba Dam).
- NZS1170 “Structural design actions”.
- NZS3101 “Concrete structures”.
- USBR (1971) “Uplift Control on Spillways for Dams”
- USBR (1990) EM42 “Cavitation in chutes and spillways”.

17.3.2 USBR (2007) “Uplift and Crack Flow Resulting from High Velocity Discharges Over Open Offset Joints”. Precedent spillway design

The previous (Stage 3) CFRD specialist and dam designer (Mr Phil Carter) was on the design team for the Tillegra Dam (NSW, Australia) and obtained permission from Hunter Water Corporation to use the physical model study findings to assist with the design of the Waimea Dam. Considering the advantages gained by having a model study to support the design, the Waimea Dam spillway design was therefore adjusted to match the Tillegra configuration as closely as possible given the site constraints. Hunter Water Corporation’s cooperation in this matter is acknowledged and greatly appreciated.

The Tillegra design basis was reviewed by the Stage 3 and Stage 4 design teams. Key aspects of the design arrangements from Tillegra (such as use of rearguard waterstops, and uplift anchors limited to 3 m net uplift head) were in turn developed from the Warragamba Dam auxiliary spillway design (design completed in 2001, and construction in). The Warragamba Dam auxiliary spillway design report by SMEC further references work undertaken by Reclamation (formerly USBR).

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

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

Adjustments to the Waimea Dam spillway configuration were required in order to take advantage of the measurement data from the Tillegra Dam spillway physical model study. These adjustments were:

- Decrease the chute length in the 2H:1V section by approximately 24 m (effect considered negligible on hydraulics as flow has reached steady state conditions at this point).
- Increase the wall heights at the ogee weir to above the IDF water level, and in the chute section for a minimum wall height (parallel to the chute floor) of 3.0 m.
- Increase the upper bridge soffit heights to 203.04 to provide freeboard to the IDF water level.
- Provide vertical bridge abutments at the upper bridge below the IDF water level (to maintain bridge span of 26.4 m).

### 17.4 Flood routing

Section 4 describes the flood hydrology for the dam and presents the flood hydrographs adopted for design.

Flood routing calculations were carried out using a spreadsheet which employed forward difference algorithms to determine the outflow based on the hydrograph inflows. Spillway outflow in any particular time step is determined by the water elevation in the reservoir. The spreadsheet calculations were validated using identical simulations in HEC-HMS software.

Key flood routing results are summarised in Table 17.2. Figure 17.1, Figure 17.2 and Figure 17.3 show plots of the routing results for the PMF, 200 year ARI and Mean Annual Flood (2.33 year ARI) respectively.
Table 17.2: Key flood routing results

<table>
<thead>
<tr>
<th>Flood Event ARI (years)</th>
<th>Duration (hours)</th>
<th>Peak inflow (m³/s)</th>
<th>Peak outflow (m³/s)</th>
<th>Flood Rise (m)</th>
<th>Freeboard (m)</th>
<th>Top WL (m RL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.33 (MAF)</td>
<td>48</td>
<td>210</td>
<td>179</td>
<td>1.89</td>
<td>3.74</td>
<td>199.09</td>
</tr>
<tr>
<td>5</td>
<td>48</td>
<td>268</td>
<td>239</td>
<td>2.21</td>
<td>3.42</td>
<td>199.41</td>
</tr>
<tr>
<td>10</td>
<td>48</td>
<td>314</td>
<td>285</td>
<td>2.45</td>
<td>3.18</td>
<td>199.65</td>
</tr>
<tr>
<td>20</td>
<td>48</td>
<td>359</td>
<td>330</td>
<td>2.67</td>
<td>2.96</td>
<td>199.87</td>
</tr>
<tr>
<td>50</td>
<td>48</td>
<td>416</td>
<td>388</td>
<td>2.93</td>
<td>2.70</td>
<td>200.13</td>
</tr>
<tr>
<td>100</td>
<td>48</td>
<td>457</td>
<td>427</td>
<td>3.09</td>
<td>2.54</td>
<td>200.29</td>
</tr>
<tr>
<td>200</td>
<td>48</td>
<td>502</td>
<td>472</td>
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<td>2.35</td>
<td>200.48</td>
</tr>
<tr>
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<td>48</td>
<td>601</td>
<td>568</td>
<td>3.67</td>
<td>1.96</td>
<td>200.87</td>
</tr>
<tr>
<td>10,000</td>
<td>48</td>
<td>742</td>
<td>708</td>
<td>4.17</td>
<td>1.46</td>
<td>201.37</td>
</tr>
<tr>
<td>PMF (IDF)</td>
<td>48</td>
<td>1094</td>
<td>1058</td>
<td>5.33</td>
<td>0.30</td>
<td>202.53</td>
</tr>
</tbody>
</table>

(1) All routing runs assume an initial reservoir level at NTWL.
(2) 300 mm camber (for settlement) excluded from freeboard assessment.

Figure 17.1: IDF flood routing.
Design routing runs use the reservoir storage elevation curve shown in Section 2 and incorporate an oggee weir rating curve based on the Tillegra Dam spillway physical model study results presented in Figure 17.4. Approach channel velocities are accounted for in the weir discharge rating curve.
17.5 Spillway approach channel

17.5.1 General

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

The true left side of the channel has a large radius curve to maintain a large radius of curvature to flow depth ratio (R/y). Khatsuria (2005) recommends that this ratio should be as large as possible but no less than three. For the Waimea Dam operating at peak 200 year ARI design flood discharge conditions, this ratio is greater than 13.

Concrete lining begins 3 m upstream of the curved ogee weir (at the pier) to improve the approach conditions. The approach channel floor has a 1V:200H grade to allow drainage away from the weir under low reservoir conditions.

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

17.5.2 Velocities

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

- At peak IDF discharge = 2.6 m/s.
- At peak 200 year ARI design flood discharge = 1.6 m/s.

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

The tapered nature of the approach channel means that velocities increase as flow moves downstream towards the weir. Interpolation of measured velocities on the upstream sloping side of the physical model study weir were 4.5 m/s and 3.6 m/s for Waimea Dam IDF and 200 year ARI peak flows respectively. Velocities calculated using HEC-RAS in the same vicinity are similar, though slightly less, being 3 m/s to 4.5 m/s and 2 m/s to 3.5 m/s for IDF and 200 year ARI flows respectively. These velocities are considered to be appropriate. The lower HEC-RAS computed velocities can be explained by the inability of the software to accurately calculate the brink depth at this location, and also the influence on the model of the deeper water on the right hand side of the approach channel.

17.5.3 Drawdown

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

- At peak IDF discharge, drawdown = 1.0 m.
- At peak 200 year ARI discharge, drawdown = 0.6 m.
Drawdown calculated using HEC-RAS in the same vicinity are less, being 0.63 m and 0.22 m for the IDF and 200 year ARI flows respectively. The differences may be explained by the inability of HEC-RAS to accurately calculate the brink depth at this location and also the influence on the model of the deeper water on the right hand side of the approach channel upstream of the weir.

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

17.6 Ogee weir

17.6.1 Hydraulic design

The weir at the upstream end of the spillway chute is shaped based on the details derived from the hydraulic model study performed by the Manly Hydraulics Laboratory (2009). The weir is a USBR type ogee shaped weir, which is commonly used on dam spillways around the world and in New Zealand.

To access the dam crest, a bridge across the spillway is necessary. To reduce the span length and cost of the bridge a 0.75 m wide central pier is included in the spillway. The spillway chute walls slope at 1.5V:1H.

The weir is 40 m long as measured along the axis of the dam crest. The effective hydraulic length of the weir is 41.89 m. The variation from the axis length is due to extra width from the sloping chute walls, and the reduced width due to the pier, and abutment losses.

The weir crest level is 197.2 m RL (NTWL) with a minimum approach depth of 2.5 m. During the 200 year ARI design flood, the design peak outflow is 472 m$^3$/s with an operating head of 3.3 m (based on a coefficient of discharge, $C_d$ of 1.90). During the IDF, the design peak outflow is 1058 m$^3$/s with an operating head of 5.33 m (based on a coefficient of discharge, $C_d$ of 2.05).

The weir rating curve is shown in Figure 17.4. This weir rating curve was checked against ogee crested weir equations presented in EM1110-2-1603 and found to be in close agreement.

![Spillway rating curve (from Tillegra Hydraulic Model Rating Curve)](image)

*Figure 17.4: Ogee weir spillway rating curve.*

The underside of the upper bridge over the spillway at the ogee location is set to 203.04 m RL. This level is approximately 500 mm above the IDF water surface before drawdown and approach velocity affects are accounted for (i.e. the actual water surface profile at the bridge may be less than the
reservoir level). As the bridge is skewed relative to the ogee and the ogee is curved the water surface profile at the upstream beam soffit will vary. Pre-camber in the bridge allows for dead load deflections to maintain the minimum bridge soffit level.

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

17.6.2 Spillway (Ogee) weir structural design

The ogee weir geometry results in an inherently stable weir due to the relatively short height relative to the length. Two load cases were previously identified in Stage 3 as the critical design cases for stability (i.e. overturning and sliding). Other design cases such as usual (static) and unusual (OBE) scenarios were considered but not formally reassessed during the Stage 4 design.

These two cases are:

1. Extreme – Flood IDF (as per Stage 3 design).
2. Extreme – SEE (100%H +30%V) with the reservoir at NTWL.

Uplift pressures have been incorporated into the analysis. These have been assumed to act as a triangular/trapezoidal stress distribution equal to static water level at the upstream end reducing to zero at the downstream end (where the toe drains are located). Cracked base analysis has not been included. The adopted uplift is considered to be conservative approach given that the grout curtain extends under the full width of the ogee weir (reducing seepage) and foundation drains are located under the vertical contraction joints halfway under the weir to draw away uplift seepage.

The flood case assessment allows for the overtopping flows. The potential for formation of negative pressures on the spillway crest during the IDF was considered, noting the ogee design shape was set based on the Tillegra Dam design IDF (appro. 1500 m$^3$/s) which is approximately 440 m$^3$/s larger than the Waimea Dam IDF (appro. 1060 m$^3$/s). This means that the Waimea Dam ogee weir is unlikely to develop negative pressures during the IDF.

6x RB25 hold down passive grouted bar anchors (i.e. not pre-tensioned anchors) are included for the weir block section that supports the pier wall for the seismic stability case. These anchors are included as movement of this weir block during an SEE event could destabilise the upper bridge which might collapse and block the spillway. These anchors are not required for the IDF flood stability (i.e. weir block is stable under gravity alone for this case).

Table 17.3 shows the results of the stability and structural performance assessments. The results met the design criteria recommended in the NZSOLD Guidelines 2015 for gravity dams and are considered acceptable.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Overturning</th>
<th>Sliding</th>
<th>Normal compressive stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resultant location</td>
<td>FoS</td>
<td></td>
</tr>
<tr>
<td>Extreme - Flood (IDF) (non-anchored section)</td>
<td>Middle third</td>
<td>4.4</td>
<td>&lt;0.3f’c</td>
</tr>
<tr>
<td>Extreme - SEE (non-anchored section)</td>
<td>Middle third</td>
<td>6.9</td>
<td>&lt;0.3f’c</td>
</tr>
<tr>
<td>Extreme - SEE (pier support block with anchors)</td>
<td>Middle third</td>
<td>3.8</td>
<td>&lt;0.3f’c</td>
</tr>
</tbody>
</table>

(1) Sliding assessment adopted concrete rock cohesion of 200 kPa and friction angle ($\phi$) of 37 deg. A minimum FOS of 3.0 applies for the normal case, 1.5 under the extreme flood case, and 1.0 under the SEE.

The weir blocks include skin reinforcement on the exposed face and the base of the weir for anti-spall and temperature/shrinkage control. The adopted reinforcement arrangements are consistent.
with the specimen Tillegra Dam design and NZS3101. The spacing of the vertical contraction joints is approximately 7 - 10 m and is consistent with USACE guidance for concrete gravity dams.

17.7 Spillway chute

17.7.1 General

The Waimea Dam spillway chute is generally based on the Tillegra configuration, for which physical model study data is available, as described in Section 17.1.

Downstream of the ogee weir the chute contains both horizontal and vertically transitions. In the vertical, the chute grade steepens from 10H:1V to 2H:1V over a 60 m radius convex vertical curve. Horizontally the chute contracts in a fan shape from 40 m at the crest to a minimum of 20 m in the steep portion of the chute (contraction ratio 0.5). The vertical curve is designed to maintain positive pressures over the invert and discourage flow separation. Spillway chute details are shown on the Drawings.

HEC-RAS modelling was undertaken to confirm the results of the physical model study and extract additional information required for specific design of the Waimea Dam spillway. HEC-RAS modelling was carried out over a range of Manning’s roughness values (n=0.008, n=0.014 and n=0.018) in accordance with USBR recommendations (USBR, 1987).

The horizontal contraction ratio of the spillway chute transition is 0.5, which is considered to be relatively modest (ICOLD, 1992). The fan shape avoids abrupt changes in side wall angles and improves flow conditions, meeting USACE recommendations that chute sidewalls be curved horizontally with long radii when Froude numbers are greater than 1.5 (USACE, 1990).

The spillway also has straight-lined contracting side walls running downstream from the ogee crest, followed by a curved transition. Experience has shown that this configuration can give better flow conditions than providing curved sidewalls immediately at the crest (Khatsuria, 2005).

Contractions may be subject to choking if local Froude numbers are close to 1 and a hydraulic jump is able to form in the contraction. To avoid this, ICOLD (1992) recommend that designs should be based on a minimum downstream Froude number of 2.

The Manly Hydraulics Laboratory scale physical model did not show evidence of choking in the spillway contractions of the modelled flow range equivalent to 286 to 1495 m$^3$/s. HEC-RAS check modelling of the Waimea Dam spillway (refer Section 17.7.2 below) calculated Froude numbers of 1 at the toe of the ogee weir increasing to 3.0 in the 10H:1V sloped contraction section, and greater than 3.0 from the vertical transition. The Froude numbers were typically higher at lower flows than the design peak flows. Given the side wall contraction effects on the flow profile, it is considered that the 1D HEC-RAS modelling is not adequately modelling the potential for contraction choking and the MHL physical model results have been relied upon for the contraction design.

17.7.2 Hydraulic model checks

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

The HEC-RAS model was run for a flow rate of 1495 m$^3$/s (1.4 times the Waimea Dam IDF flow of 1060 m$^3$/s) and the computed aerated water surface profile compared to the water surface profile reported for the physical model of the same flow. At this flow the HEC-RAS modelling showed that:

- Over most of the chute length the HEC-RAS model was found to overestimate the water surface profile by an average value of around 0.45 m (0.75 m maximum).
- The HEC-RAS model under predicts the water levels along the side walls to around 22 m downstream of the ogee crest due to the side wall contraction effects.
- The HEC-RAS model under predicts the water levels along the side walls between approximately 80 m to 100 m downstream of the crest by an average value of around 0.4 m (0.6 m maximum).
- The HEC-RAS model was then run for the Waimea Dam IDF using a conservative Manning’s roughness value of 0.018 (USBR, 1987) and the aerated water surface profile calculated.

17.7.3 Height of concrete lining

The concrete lining for the Waimea Dam spillway has been taken to the IDF (PMF) chute water surface profile, with additional nominal freeboard accounting for contraction wave action on the side walls and bulking due to air entrainment. The minimum wall height adopted was 3 m, noting this is slightly higher than the Tillegra Dam spillway design.

The spillway is located within rock excavation, with the true right batter flattened to form a bench level with the top of the wall. Splashing over the true right wall and onto the adjacent dam fill is not expected with the design levels selected.

17.7.4 Spillway cavitation

ICOLD conducted a survey of dam spillways in 1980. Where erosion of the spillway surface was found to be a problem, most were operating with maximum velocities greater than 30 m/s and specific discharges of over 50 m$^3$/s/m (Novak et al., 2007).

The Waimea Dam IDF maximum channel velocity is approximately 25 m/s and the maximum unit discharge is approximately 50 m$^3$/s/m. Both are close to but below the above cavitation thresholds.

Further assessment for potential for spillway cavitation was undertaken in accordance with USBR (1990) EM42 and Novak et al. (2007). This analysis suggest that the potential for cavitation is low and that spillway aeration devices would not be unnecessary. Therefore these have not been adopted.

17.7.5 Spillway floor anchorage and lining

In a typical concrete lined spillway chute the stability of the floor slab depends on multiple design elements including reinforcement, anchorage, joint and waterstop details, and a functioning underdrain system (USBR, 2007). Modern chute spillway design for spillways on solid rock foundations should account for potential failure modes including those due to high pressure water injection through slab joints creating hydrodynamic uplift pressures.

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

The build-up of pressures under spillway slabs due to high velocity flow depends on a combination of a number of relatively low probability events, at least for spillways with modern and well-constructed design details. Nonetheless, it is considered good practice for spillway chutes on rock foundations to provide suitable joint details and waterstops to discourage water entering through the joints as a primary measure, with a secondary drainage system to limit the buildup of hydrodynamic uplift pressure under the concrete lining, and should drainage be ineffective, design for uplift pressure by providing anchors into the spillway chute foundation. The Waimea Dam
spillway chute is considered to be a critical structure given the proximity of the adjacent embankment and thus defensive design features are provided.

Reclamation (USBR, 1971) recommends that chutes on rock should be designed for minimum hydrostatic uplift heads of 3 m. Other published methods such as that recommended by McLellan (1976) recommend the design uplift should be some fraction (k) of the velocity head (h=k v^2/2g). McLellan recommends k = 0.15 where drains are provided and k = 0.3 where there are no drains. Use of the velocity head approach with these factors is consistent with ICOLD Bulletin 58 also.

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

The hydrostatic uplift head selected for design of the Waimea Dam spillway chute is based on a proportion of velocity head with k = 0.15 but capped at 3 m. This approach is the same approach adopted to the Tillegra and Warragamba Dam spillway designs.

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

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

- Anchor bar grout to rock (country bond strength) strength of 500 kPa with an applied strength reduction factor of 0.50, based on moderately weathered to fresh greywacke rock. The actual country bond strength is subject to confirmation on site noting higher strengths are typical in similar South Island greywacke and the value adopted is considered to be a suitable conservative lower bound estimate.

The anchor design methodology takes account of overlapping pull out cones for adjacent anchors, assumes a submerged unit weight of rock and ignores side friction effects for the mobilised rock mass. The dowel length is shown on the Drawings and is subject to confirmation on site of the encountered rock mass.

The floor lining is formed of 300 mm thick reinforced concrete with all longitudinal and transverse joints provided with PVC waterstops. Reinforcing steel is provided throughout to control shrinkage and thermal cracking. Drainage provisions are discussed in Section 17.7.6.

17.7.6 Spillway floor drainage

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

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

As mentioned in the preceding section, providing spillway drainage is common practice to provide an additional redundant measure to control potential uplift pressures. This can include pipe underdrains or drilled eductor drains.

Nexus Hi-way type PE100 perforated underdrains have been selected for the Waimea Dam spillway. These drains are located under each of the transverse contraction joints (which are hypothetically exposed to a higher risk of uplift) and discharge into longitudinal collector drains running down either side of the chute. The inclusion of these drains also enables monitoring of the spillway joint performance during operation.

The drains are surrounded by porous concrete or filter material. The specific drain surround material will be subject to confirmation after inspection of the cleaned spillway foundation. The filter material has a design grading with a $D_{50}$ greater than the drains drilled hole size or maximum slot dimension as per FEMA “Filters for Embankment Dams” 2011. Additional filter layers may be necessary in select locations to filter finer particles along joints noting the spillway foundation relies on sound rock and will be prepared such that loose material and/or weak rock is removed.

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

- Potential unit flow rates into the defect calculated based on the HEC-RAS model spillway velocities and the USBR study findings (USBR, 2007).
- Drains sized to convey flow from a defect with a 3 mm (1/8 inch) gap and 3 mm offset with a vented cavity extending over 25% of the chute width.
- Drains assumed to be full flowing pipes with a maximum head of 3 m, corresponding to the hydrostatic uplift head for which the anchors are designed.
- Drain flow losses account for exit losses into the longitudinal drains as well as ‘screen’ entry losses and friction losses based on well screen research (Barker & Herbert, 1992a) (Barker & Herbert, 1992b).

An additional function of the underdrains is to relieve groundwater pressures due to potential seepage from the reservoir or upslope. Seepage analysis using Seep/W software suggests very low groundwater flows ($<< 1$ l/sec over the entire spillway) would occur through the rock mass from the reservoir, without consideration of the additional seepage control provided by the ogee weir and abutment grout curtain (which would further reduce the potential seepage flows). The potential for higher groundwater flows due to untreated defects in the rockmass (e.g. unfavourable dilated joints) requires attention during construction, noting the implementation of the grout curtain and foundation treatment requirements for the spillway and ogee are intended to address this risk.

The underdrainage system features multiple levels of redundancy in the highly unlikely event that uplift pressures occur due to groundwater seepage and/or water injection. The underdrain network has two independent collector lines (true left and right) and cross connections between the transverse and longitudinal drains to facilitate camera inspections and flow rerouting should an individual line be blocked.

In addition to the two collector lines, the drains are sized for a flow of at least 10 times the potential design flows (i.e. factor of safety of 10) and the collector drains downstream of the chute (i.e. at the flip bucket connection) sized to convey the same flow as the steeper chute drains (i.e. 1600D drains at 50% grade connecting to 355OD drains at 1% grade). The expected design performance is that the underdrainage network will have minimal to no flow, noting each drains line has a total capacity of approximately 100 l/sec.
Transverse overflow relief and access drains are provided on both sides of the chute at each transverse drain and selected intermediate locations (connected to the true left and right drain lines). The overflow drains are set to allow overflows in the highly unlikely event that uplift pressures occur and these exceed 2.5 m (noting spillway anchors are designed for 3 m net uplift pressure). The drain outfalls are routed to accessible locations to enable inspection as part of the routine surveillance of the dam. Camera access to the drains is provided by a separate inspection point at the top of the walls, noting special access procedures will be required to access some of these locations.

Eductor drains are included in the Waimea Dam spillway design in the upper chute section only and are specified for the true left wall to enable upslope groundwater pressure relief (i.e. acting as weepholes) with provision for inclusion of eductor drains in the upper chute floor should the encountered rock quality indicate potential for groundwater uplift. The use of eductor drains is likely to be limited to the upper chute section as this area has relatively low flow velocities and therefore the potential for water injection into the educator drains is limited. The educator drain details are as per the Tillegra and Warragamba Dam spillway designs and are angled upslope to reduce the potential for water injection.

To be effective against spillway operation induced uplift, eductor drains need to be close to the crack or defect that introduces high pressure and typical spacings may generally be too wide for this to occur. Also, the USBR study (USBR, 2007) demonstrates the considerable flow that may be generated by even a small gap into the subsurface drainage system. For these reasons the eductor drains are not included as a specific measure to relieve water injection due to spillway operation.

17.7.7 Discussion

17.7.7.1 Waimea Dam spillway philosophy

The Waimea Dam spillway design assumes a sound rock foundation. It is therefore essential that the entire spillway foundation is inspected by the Designer to validate this assumption. Where weaker rock is identified this will require removal and replacement with mass concrete. Significant rock defects where identified will similarly require specific treatment. Should significant areas of weaker rock be encountered during construction this will need to be over excavated and backfilled with mass concrete and/or alternative design details developed.

As covered in the preceding sections, multiple defensive design measures have been included for the spillway to limit the potential for uplift (chamfered joints, joint keys and waterbar, drainage) and the effects of uplift (rock anchors). The drainage features overflow relief drains that act as the final measure for limiting potential uplift to below the 3 m net uplift used for anchor design. These measures are considered conservative and appropriate for the Waimea Dam, noting the underdrain system with overflow relief is considered to be a final backstop measure to the primary measures.

17.7.7.2 Consideration of the February 2017 Oroville Dam spillway incident

The Oroville Dam service spillway (California, US) chute failed in February 2017 during operation at flows well below the design capacity and less than historic spillway flows. The chute was reported to have failed due to uplift of a concrete slab followed by erosion of the underlying weak rock foundation.

In January 2018 (during the Stage 4 design) a report1 was released by the independent forensic team for the February 2017 Oroville Dam spillway incident. This report outlined the forensic team’s

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1 Independent Forensic Team Report “Oroville Dam Spillway Incident” dated 5 January 2018.
findings and opinions as to the likely causes of the service spillway chute failure at the Oroville Dam and should be referred to for specific details related to that dam.

The Oroville spillway incident highlighted a number of important aspects that need to be accounted for during the design, construction and operation of chute spillways (such as the Waimea Dam). The independent forensic team report concluded that while a complex range of factors lead to the 2017 Oroville Dam spillway incident, significant contributors were:

- Inherent vulnerabilities in the spillway design and as-constructed conditions, and subsequent chute slab deterioration.
- Poor spillway foundation conditions in some locations.

The report states “The inherent vulnerability of the service spillway design and as-constructed conditions reflect lack of proper modification of the design to fit the site conditions. Almost immediately after construction, the concrete chute slab cracked above and along underdrain pipes, and high underdrain flows were observed. The slab cracking and underdrain flows, although originally thought of as unusual, were quickly deemed to be “normal,” and as simply requiring ongoing repairs. However, repeated repairs were ineffective and possibly detrimental”.

The Waimea Dam spillway design features different joint and underdrain details to the Oroville Dam spillway that mean one of the suspected mechanisms of causing the initial cracking in the Oroville spillway chute is highly unlikely to occur for the Waimea Dam, provided it is constructed to the design and the foundations are verified as being on satisfactory rock.

17.7.7.3 Alternative spillway joint details

International practice for the design of chute spillways places significant emphasis on the importance of adopting appropriate joint details that are suitable for the specific foundation and flow conditions applicable. Alternative joints detail arrangements were considered as part of the spillway design development including review of international precedents and design guidance as part of the Stage 3 and Stage 4 design phases. Some examples of alternative joint arrangements considered for the Waimea Dam are summarised below.

Reclamation Design Standard 14 “Appurtenant Structures for Dams (Spillways and Outlet Works)” (DS14) provides a summary of recommended design practice in the US, noting the latest revision to this standard was in 2014 before the 2017 Oroville spillway incident. DS14 shows a range of defensive design measures based on US practice for spillway joints consisting of waterstops, reinforcement across joints, filtered underdrains, concrete cutoffs at transverse joints and rock anchors.

Typical contraction joint details in DS14 feature dowels across the joints to reduce the potential for offset joints. In order for offset joints to occur, the chute slab needs to be subject to high differential uplift pressures (due to stagnation pressures and/or joint injection and/or groundwater pressures) and be able to move. The use of dowels/continuous reinforcement is understood to be focused towards high flow velocity spillways on weak rock/soil foundations where a higher potential for joint offset may be present.

The Waimea Dam spillway has relatively low flow velocities for a chute spillway which limits the potential for stagnation pressures to occur, and features multiple barriers to the potential development of high uplift pressures (chamfers, keys, waterbar, and underdrains with overflow relief). Further to this the Waimea Dam spillway features an extensive network of rock anchors and
shall be founded on solid rock or mass concrete, which further limit the potential for offset joints to occur.

DS14 includes foundation keys at transverse contraction joints for steep chute sections. Foundation keys are typically adopted for soil or weak rock foundations. This detail was not considered necessary for the Waimea Dam given the extensive floor anchors and the bond strength provided between the concrete slab and the rock foundation.

A summary article by P.J Mason on spillway chute design in the International Journal of Hydropower and Dams (Issue 5 2017) also shows a range of joint details based on the authors experience including details similar to those adopted for the Waimea Dam (featuring chamfers and anchors) and also DS14 (featuring foundation keys).

Drainage details in DS14 include 150 to 250 mm double wall HDPE pipe under joints, noting no basis for this sizing is provided. Mason (2017) provides commentary based on his experience that underdrains should not be less than 300 mm, noting no basis for this comment is included. We are aware of a number of international spillway precedents that use alternative drain configurations to DS14 and feature underdrains pipes less than 300 mm. The Waimea Dam spillway underdrains are sized based on established methodologies for assessing potential joint injection flows and groundwater seepage with appropriate factors of safety and are therefore considered to be appropriately sized and consistent with current international practice.

The use of rearguard waterbar for the Waimea Dam joints was made with consideration of the potential construction difficulties associated with placing centerbulb waterstops in a relatively thin slab with keys. Rearguard waterbar were used on the Warragamba Dam auxiliary spillway for this reason also. In terms of New Zealand precedents, the Kourarau Dams upgrade (Low PIC) (upgraded by Genesis Energy as designed by T+T between 2008 and 2010) included rearguard waterbar on the spillway.

It is common for spillway joints to feature centrebulb type waterbar within the slab and this is shown in the DS14 example details, noting use of rearguard waterbar is not discussed in this document. One of the authors of DS14, Thomas Hepler (formally from Reclamation) presented on spillway chute joints details at the 2017 ASDSO Conference and included base seal (rearguard) waterbar noting this may be good for thin overlays.

Use of rearguard waterbar is common in other water retaining civil engineering structures (e.g. tanks). The supplier (Sika NZ) for the specified rearguard waterbar advised that it would be suitable for the proposed application and water pressure range in the Waimea Dam spillway. These waterbars are fastened in place on the mortar pad or concrete foundation prior to concrete placement and operate as a water stop in both direction (i.e. against injection water from the spillway and groundwater uplift).

The supplier’s datasheet for the specified waterbar product shows that the waterbar can resist up to approximately 448 kPa water pressure. This is far in excess of either behind wall or in-chute pressures which could be up to a maximum of approximately 7 m (70 KPa) hydrostatic head in the unlikely and conservative case at the top of the spillway. The suppliers (Sika) data state that testing is in accordance with Reclamation standards and USACE EM 1110-2-2101 “Waterstops for Civil Works Structures” dated 30 September 1995. Sika consider the stated pressure heads to be ultimate values.

The joint details adopted for the Waimea Dam are consistent with international precedents including the Warragamba Dam auxiliary spillway, and are considered to be appropriately conservative with suitable defensive design measures for a solid rock foundation. The performance of the spillway joints and spillway chute in general relies on providing suitable rock foundation and correct construction practices. Poor execution of the joint details will reduce the effectiveness of the
design measures and full time construction observation is typically required for chute spillways to maintain quality control.

17.8 Flip bucket

17.8.1 General

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

17.8.2 Radius

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

The bucket lip or exit angle determines the throw distance and angle of the flow entering the water, which in turn has a large effect on the scour depth in the plunge pool. In practice, angles typically vary from 20° – 40° (Khatsuria, 2005). An angle of 40° was adopted to match the parameters used in the Tillegra hydraulic model study. This allowed the use of the results from the model study to estimate the location of the impact zone and provided greater certainty in design. The lip height is set to 156.6 m RL which is the approximate IDF tailwater level.

17.8.3 Hydraulic loads

The water pressures developed within the flip bucket whilst operating at the IDF have been investigated using three methods. Theoretical pressure distributions were calculated using methods developed in USACE (1987) and USACE (1994). The results from the Tillegra Hydraulic Model Study (Manly Hydraulics Laboratory, 2009) included measurement of static pressures within the flipbucket. These results were linearly interpolated to the IDF (1060 m$^3$/s) (which is lower than the IDF for the Tillegra Dam) and produced comparable results to the theoretical methods. The results from these methods are shown on Figure 17.5.
17.8.4 Stability assessment

Two load cases were investigated for stability against overturning and sliding. The IDF flow without seismic acceleration and with the reservoir at NTWL coupled with an SEE event. This analysis allowed for the formation of negative pressures on the spillway crest due to operation above the design flow as shown on Table 17.4 below.

Table 17.4: Flip bucket stability results

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Overturning</th>
<th>Sliding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FoS</td>
<td>Resultant location</td>
</tr>
<tr>
<td>IDF</td>
<td>1.74</td>
<td>Within middle third of base</td>
</tr>
<tr>
<td>NTWL with SEE</td>
<td>2.39</td>
<td>Within middle third of base</td>
</tr>
</tbody>
</table>

The estimated maximum pressure exerted on the bedrock supporting the flip bucket is 80 kPa. The resultant is located at the downstream edge of the flip bucket foundation. The maximum bearing capacity of the rock was estimated at 30 MPa using a method described by Bowles (1996). Thus, notwithstanding unforeseen conditions, based on interpretation we expect that there is adequate capacity in the rock to support the pressures exerted by the flip bucket. The foundation of the flip bucket will require inspection during construction and if necessary defects treated/over excavated and backfilled with mass concrete.

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

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

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

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

- Dead load & IDF dynamic water pressure.

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

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

The flipbucket lip should be supported on sound rock or mass concrete. The excavation profile for the flipbucket foundation and the corresponding structural dimensions and reinforcement are subject to confirmation on site following exposure of the rock to the design levels.

17.9 Plunge pool

17.9.1 General

The plunge pool comprises a trapezoidal unlined channel downstream of the spillway and flip bucket. The base of the pool is approximately 45 m long by 10 m wide. At the downstream end of the pool the channel invert rises back to river level (147.18 m RL). The pool is at least 3 m deep at the upstream end and 5 m deep at the downstream end.

17.9.2 Scour

To identify the location that scour is likely to occur and the extent of pre-excavation of a plunge pool that will provide the most benefit, it is necessary to predict the trajectory of the jet from the flip bucket during a range of design flows. The physical model study data from the design of Tillegra Dam was used to calibrate the Kawakami (1973) method for calculating trajectories of a free jet from the spillway. The effective lip angle and air resistance parameters were changed to replicate the real jet trajectories found in the physical model study, and these parameters were interpolated/extrapolated for the velocities and flow rates for the mean annual flood, 200 year ARI flood and IDF at Waimea Dam.

The hydraulic design parameters selected for the flip bucket and plunge pool design are based on water profile and jet trajectory measurements from the MHL Tillegra physical model study report, and are as follows:

200 year ARI design flood:

- Velocity - 24.0 m/s (at lip of flip bucket)
- Flow depth - 2.0 m (at lip of flip bucket)
- Effective lip angle - 34°
Froude number - 8.3 (at entry to flip bucket) (n=0.014 model)

IDF:
- Velocity - 27.3 m/s (at lip of flip bucket)
- Flow depth - 3.4 m (at lip of flip bucket)
- Effective lip angle - 36°
- Froude number - 6.2 (at entry to flip bucket) (n=0.014 model)

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

Mason (1985) reviewed a significant number of the empirical methods that assumed scour extent is largely independent of rock mass quality (as per Mason, 1993). The predicted scour depths at Waimea Dam based on Mason’s equation derived from this collation / review are summarised in Table 17.5. The table also presents the smallest scour depths predicted (Damle 1966) from the methods considered for the Waimea Dam. The predictions from all the methods that did not consider rock mass quality that were considered for Waimea Dam are bracketed by the predictions for Damle (1966) and the Mason (1985) upper bound estimates.

### Table 17.5: Scour depth estimates

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean annual flood</td>
<td>11.6 m</td>
<td>23.2 m</td>
<td>7.5 m</td>
</tr>
<tr>
<td>200 year ARI</td>
<td>18.7 m</td>
<td>28.1 m</td>
<td>12.1 m</td>
</tr>
<tr>
<td>IDF</td>
<td>27.8 m</td>
<td>41.7 m</td>
<td>17.9 m</td>
</tr>
</tbody>
</table>

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

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

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

We recommend that Waimea Water develop a stakeholder communications plan to effectively manage any perceived issue around erosion and scour that occurs at this location.

The downstream concrete lining is intended to be a sacrificial concrete lining to provide protection to the flip bucket. This lining will provide protection against scour and erosion when the flip bucket is not operating at high flows. Whilst a 3 m deep cutoff wall has been shown, it is expected that in the
long term that this may eventually be undermined and the lining will start to break-up. Given the high cost of preventing this from occurring now we consider it appropriate that this is best addressed during operation and maintenance over the design life of the project.

Future works may involve inspections by divers and if considered appropriate, filling scour holes with concrete or rock armour. The timing of future maintenance will depend on frequency of flood events in the river.
18 Bridges

18.1 General

There are two bridges across the spillway channel to enable access to the dam:

- Upper bridge to the dam crest from the crest access road. This bridge is to provide access to the crest of the dam, instrumentation, and the intakes.
- Lower bridge to the toe access road. This bridge is to provide access to the outlet works and provisional future power station.

We understand that Waimea Water may elect to delete the lower access bridge in favour of an alternative access point to the dam toe from the right abutment. Design of that access is by others.

18.2 Design basis

18.2.1 Standards and references

The bridges have been designed in accordance with the following standards and references:

- NZS1170 “Structural Design Actions”.
- NZS3101 “Concrete structures” - Bridge abutment and decks.
- NZS3404 “Steel structures”.

18.2.2 Bridge level selection

The bridge levels presented in the Stage 3 design were selected based on the estimated nappe drawdown at the ogee and the estimated flow bulked IDF water levels in the spillway. The levels were reviewed as part of Stage 4, and with consideration of uncertainties in design flow levels, implications of debris fouling, and potential risks to the bridges and overall dam safety.

Following the Stage 4 review, the upper bridge deck level was increased to 204.46 m RL (at centreline) to bring the bridge soffit to 203.04 m RL to give at least 500 mm freeboard above the IDF peak reservoir water level (202.53 m RL) without allowing for nappe drawdown. Estimation of nappe drawdown is complex given the three dimensional effects of the curved ogee (which give variable water levels along the upstream beam soffit) and is uncertain. Providing some freeboard to facilitate passage of floating debris is also recommended to reduce the risk to the upper bridge.

The lower bridge deck level was also increased from the Stage 3 design to 164.78 m RL (at centreline) which gives a soffit elevation of 163.39 m RL. This level gives approximately 1400 mm freeboard from the upstream bridge beam soffit above the estimated flow bulked IDF water level in the spillway chute.

The increased bridge deck levels required adjustments to the Stage 3 design arrangements including the road geometry, additional retaining wingwalls, increasing the mass concrete block height, and the length of the crest ramp (with corresponding reduction in parapet wall length).

18.2.3 Design vehicle and loadings

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

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

Whilst the design vehicle is a three axle, 11 m long truck with an 8.2 tonne design axle, we have also considered a single HN (maximum legal weight limit vehicle) vehicle (not acting concurrently with a UDL) on any given span of the bridge and a small mobile crane. The bridge design has adequate capacity for these alternative vehicle loadings at crawling speeds.

We recommend however that any vehicles that are not consistent (larger or heavier) with the agreed design vehicle shown in Figure 18.1 should be assessed on a vehicle by vehicle basis prior to use.

Figure 18.1 shows the key dimensions of the design vehicle.

![Figure 18.1: Waimea Dam design vehicle for bridge design.](image)

Should larger or heavier vehicles are required to gain access across either of the two bridges, then temporary support could be provided to the bridges. The temporary support would need to be designed appropriately for the loads under consideration. It is expected that temporary support of the bridges will be necessary during construction.

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

The design does **not** consider the UDL and vehicle load to act concurrently.

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

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

18.3.1 Bridge dimensions

The key dimensions for both bridges are summarised in Table 18.1 below. Given Waimea Water’s desire to keep costs to a minimum the bridge widths have been optimised accordingly.

Vehicle tracking (Autoturn) has been used to check using the design vehicle. This shows that the design vehicle can progress across the bridges and onto the upper dam crest. It is apparent however that the horizontal clearance is approximately 300 mm at the pinch points and that an adequately trained driver will likely be required.

Table 18.1: Bridge summary

<table>
<thead>
<tr>
<th>Description</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck width - overall (between kerbs)</td>
<td>4.4 m (4.0 m)</td>
</tr>
<tr>
<td>Bridge length</td>
<td>Single 25 m clear span (26.2 m bearing to bearing)</td>
</tr>
<tr>
<td>Bridge type</td>
<td>Steel beam sub structure with composite concrete deck</td>
</tr>
</tbody>
</table>

18.3.2 Bridge type

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

The primary reason that this bridge type was selected was for constructability. Alternative types such as concrete bridges (e.g. Super T’s) were also considered. Modern concrete bridges in New Zealand are normally constructed in pre-cast segments and then transported and lifted into position. Given the access road to the dam site T+T consider that there will be difficulties in transporting bridge beams in excess of 26 m long to the site.

We have discussed this with FHTJV who have communicated that they would likely splice the beams onsite. The advantage of transporting steel beams is that they can be manufactured in segments and spliced together onsite before being lifted into position. Splices will be full penetration butt welds.

18.3.3 Bridge deck

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

The nominal concrete deck thickness is 180 mm. The concrete deck spans perpendicular to the bridge span between the longitudinal beams.

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

No formal drainage for the bridges is to be provided. Surface water will be shed via cross fall to drain holes in the upstand kerbs.
18.3.4 Bridge beams

The beams are 1200WB249 custom welded steel beams. These beams could be manufactured by Steltech in Glenbrook (Greater Auckland), or an experienced steel fabricator could manufacture the beams locally if this is more cost-effective.

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

The bridge bearings will require future replacement and as such that concrete abutments have been sized to accommodate insertion of jacking points.

18.3.5 Bridge pier and abutments

The abutments of both bridges are reinforced concrete beams. The abutment beams are approximately 5 m long with the beam width and depth varying to suit the location, with specific details as follows:

- Upper bridge - The true left abutment beam is 1125 mm wide and approximately 2700 mm high. The beam will be founded on rock at 201.7 m RL and includes 10x 5 m long HD32 passive bar anchors to tie the abutment into the rock.
- Upper bridge - The true right abutment will be founded on the dam mass concrete block abutment at 201.7 m RL. The abutment beam in this location is integrated into the mass concrete block.
- Upper bridge - Central pier. The central pier is a 750 mm wide reinforced concrete wall with a 1650 mm wide cap that supports the steel beams via elastomeric bearings. The cap includes a 750 mm wide 900 mm long centrally located (between the two deck sections) to reinforced concrete plinth for seismic restraint.
- Lower bridge - The true left abutment beam is 1575 mm high with an 1125 mm wide and 650 thick base with a 150 mm thick headwall 925 mm high. The beam is partially founded on rock and the 500 mm thick spillway chute wall. 10x 5 m long HD32 passive bar anchors to tie the base of the abutment into the rock.
- Lower bridge - The true right abutment is similar to the true left, with the inclusion of the headwall acting as a retaining wall to the engineered fill that forms the adjacent road embankment. The foundation rock level is subject to confirmation of the actual rock quality in this location and the extent of overbreak. Foundation over excavation will be backfilled with mass concrete.
- All abutments other than the true right upper bridge (which is integrated with the mass block) feature 200 mm thick return walls to provide restraint against potential lateral movement (e.g. during a seismic event). Stainless steel linkage bolts are specified at the abutments and central pier for seismic restraint (bolts connecting through end of beams).

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

While no significant settlement is expected where the bridge abutment foundations are founded on rock, a provisional settlement slab detail is provided for the abutments with adjacent engineered fill. This detail is required for the lower bridge true right abutment and provisionally for other abutments subject to the extent of on site excavation.
18.3.6 Mass concrete block

The true right abutment of the upper bridge features a mass concrete block to support the bridge and provide a transition to the adjacent crest ramp. The mass concrete block also includes an upstream curved build out to provide a more hydraulically efficient transition around the corner into the spillway (refer Section 17 for additional details).

The geometry of the mass concrete block is relatively complex given the geometric constraints and excavation profile in this area. The design intent is to leave in situ as much suitable rock as is practicable and pour the mass concrete over the prepared surface to obtain the design finished levels.

The stability of the mass concrete block has been assessed for range of design cases (static, flood and seismic) and the assessed stability against sliding and overturning is within the design criteria for concrete dam stability as per the NZSOLD Guidelines 2015.

18.3.7 Fall protection and guardrails

Upstand kerbs are provided to prevent vehicles from falling off the bridge. These kerbs are 300 mm wide by 300 mm high and include cast in 50 mm diameter PVC pipe drain holes at 500 centres. The pipes will require regular clearance of silt and debris.

Side mounted handrails (CSP Pacific Nu-Guard PVB or equivalent) with galvanised steel barriers (CSP Pacific Bridge Flexi-Rail W-beam barrier or equivalent) are provided on the bridges.

Galvanised steel crash barriers (CSP Pacific Highway Flexi-Rail W-beam barrier or equivalent) are provided at the bridge approaches. The barriers flare out at the terminations to a standard curved trailing terminal installation, except for the true right abutment of the upper bridge where the crash barriers continue to the crest ramp wall (upstream) and extend along the entire length of the dam crest (refer Section 21 for further details on roads).
19 Reservoir

19.1 Debris boom

The catchment above the Waimea Dam is generally forested with commercial exotic forest species and pockets of indigenous vegetation. Although the reservoir area is to be cleared of vegetation, in the long term, localised or widespread mobilisation of forestry debris, associated with heavy rainfall events, needs to be managed.

It is considered highly likely that large debris rafts will form on the reservoir over the operating life of the dam and the risk of compromising spillway capacity needs to be appropriately mitigated.

The debris boom is intended to reduce the risk of debris fouling the dam face and spillway and to facilitate safe maintenance of debris in the long term. The debris boom will require regular inspection and clearance of accumulated debris by the dam owner, especially following large flood events.

The specified debris boom is a Worthington Products Incorporated (WPI) approximately 300 m long TUFFBOOM waterway barrier with 610 mm debris screens and an in-water mooring buoy to alleviate load and maintain stability. T+T has not designed the boom or anchor arrangements as these aspects are included in the suppliers design.

19.2 Boat ramps

19.2.1 Description

Two boat ramps are required for access either side of the debris boom. These are:

1. The dam side boat ramp (Boat Ramp 1) is formed by excavating into the existing slopes to rock level immediately upstream of the spillway inlet forebay.

2. The reservoir side boat ramp (Boat Ramp 2) is formed by using the existing Waterfall Creek access road (which will be inundated by the reservoir) as the upstream boat ramp. Using the existing road may require some limited scraping to remove soft material.

The ramps allow for access from the IDF peak water level of 202.53 m RL down to the minimum operating level of 166.5 m RL.

The boat ramps are 4.5 m wide and have a maximum design grade of 1:6.7 (15%). It is noted that forward lowering of the boat trailer (e.g. with a temporary tow bar on the front of the vehicle) would improve the operators safety during boat ramp use.

We recommend that both boat ramps are inspected prior to their use given long term siltation and local stability issues that may arise during the course of reservoir operation.

19.2.2 Design basis

Australian Standard AS3962-2001 Guidelines for design of marinas provides guidance for width and slopes of boat ramps. The specified minimum width in AS3962 for a single lane boat ramp without kerbs is 4.5 m (cl 7.2.3.2) with a recommended gradient of between 1V:9H to 1V:7H.

The gradient of the boat ramp at Waimea Dam varies with a maximum of approximately 1:6.7. Whilst this is steeper that preferred we consider that it is appropriate because:

- The steeper sections are only exposed at lower reservoir levels.
- The boat ramp is not expected to be regularly used (unlike at recreational boat ramps).
• A dam owner may elect to moor a boat or barge permanently in the reservoir and therefore access will be required less frequently.
20 Outlet works

20.1 Description

The outlet works comprise intake screens, pipework and valves. There are two intake structures, one a high level intake at RL 185 m and the other a low level intake at RL 166.5 m. The intake levels and need for two outlets was adopted previously based on the recommendations for water quality management from Cawthron (Dec 2009).

The main features of the outlet works are summarised in Table 20.1 below. The detailed design of the M&E works associated with the outlet works (excluding the intake screens) will be reported on by WSP|Opus in a separate document.

Table 20.1: Outlet works summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of outlets</td>
<td>2</td>
<td>Two outlets required high level and low level.</td>
</tr>
<tr>
<td>Flushing flow</td>
<td>5,000 l/s</td>
<td>Flushing flow (occasional releases) may be required through a single outlet.</td>
</tr>
<tr>
<td>Irrigation flow</td>
<td>2,230 l/s</td>
<td>Maximum forecast downstream release. May be through one or both outlets.</td>
</tr>
<tr>
<td>Environmental release flow</td>
<td>510 l/s</td>
<td>To meet minimum residual flow. May be through one or both outlets.</td>
</tr>
<tr>
<td>Minimum operating water level, upper intake</td>
<td>RL 185.0 m</td>
<td>Based on requirements for upper intake in Cawthron report No. 1701, Dec 2009.</td>
</tr>
<tr>
<td>Minimum operating water level, lower intake</td>
<td>RL 166.5 m</td>
<td>Based on requirements for lower intake in Cawthron report No. 1701, Dec 2009.</td>
</tr>
<tr>
<td>Intake Screen</td>
<td>-</td>
<td>To protect the downstream pipework and valves by preventing debris from entering the pipework. Also to provide protection to aquatic life by limiting the bar spacing and approach velocity. The intake bellmouth level is set below the minimum operating water level to prevent vortices forming and air being drawn into the pipework. The removal of the screens is intended to be achieved by winching the intake structure up the face of the dam or by flotation by divers.</td>
</tr>
<tr>
<td>Inclined Intake Pipework</td>
<td>1,000 mm diameter steel</td>
<td>The pipework design is based on epoxy coated and lined spirally welded steel pipework, bends and fittings. Steel pipework has been selected as it has less specialised manufacturing processes, and also provides the additional flexibility of being able to weld components together, either on-site or in the factory using routine techniques. The minimum diameter of the pipeline was determined by consideration of the maximum velocity through the primary isolation valve (see below) and to minimise erosion in the long radius bends at the base of the dam / inlet to conduit. For simplicity the diameter of the inclined intake pipework has been sized to be the same as the long radius bends and to reduce the long term internal erosion of the, difficult to access, pipework.</td>
</tr>
</tbody>
</table>
Minimum wall thicknesses are recommended based on requirements of internal pressure, shipping, handling, buckling, impact loads and robustness. The removal and adjustment of the inclined intake pipework can be achieved by use of divers and winching the individual pipe lengths up the face of the dam or by flotation.

<table>
<thead>
<tr>
<th>Primary Isolation Valve</th>
<th>1,000 mm diameter butterfly valve 160 m pressure rated (PN16)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Required to isolate the pipeline and valves in the conduit to allow maintenance of these items. This needs to be located as far upstream as possible in the conduit to minimise the risk to the conduit and dam caused by the pressurised pipework. A wedge type gate valve is the most secure and robust valve option as it has two separate sealing faces and the physical arrangement does not allow the gate to be dislodged. The valve sizing is based on the recommended maximum velocity from a reputable valve supplier. The valves are recommended to be electrically actuated to allow the valves to be remotely opened and closed without the need to access the upstream end of the conduit. The electric actuators will also provide a method of shutting the primary isolation valves in an emergency, should a major leak downstream of the valve prevent safe access to the valve actuator. The valves will also be capable of manual operation. The primary isolation gate valves will be provided with a small bypass valve to balance the upstream and downstream pressures on the gate valve to aid the operation of the gate valve. The valve is in a difficult to access location and will be difficult to maintain and remove (if needed). It is therefore critical that a high quality valve is installed and thoroughly tested and witnessed at the factory.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conduit Pipework</th>
<th>1,000 mm diameter steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The pipework design is based on epoxy coated and lined spirally welded steel as for the inclined intake pipework. To minimise the downstream pipework costs, a smaller internal diameter pipework is proposed downstream of the primary isolation gate valves. This is an area that can be more easily maintained through the use of the primary isolation valve. The pipework will be provided with air valves so that air can be released during filling and drawn in during emptying and to ensure that vacuums are not formed.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fixed Cone Valves</th>
<th>850 mm &amp; 350 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The fixed cone valves are required to discharge the downstream releases in a controlled and adjustable manner. Other valve options are possible but tend to be more expensive. The valves are sized to pass the flushing flow under the minimum gross head, i.e. at the minimum operating level. Both fixed cone valves are proposed as the same size to ease maintenance and operation. The valves are capable of operation over a wide range of opening and therefore allow for a good range of flow mixing from either intake. The valves are sited at the downstream end of the conduit and may need a hood to ensure the discharge envelope lands within the downstream channel.</td>
</tr>
</tbody>
</table>

### 20.2 Design basis

#### 20.2.1 Standards and references

The electrical and mechanical components of outlet works are designed by WSP|Opus as reported separately. The intake screens are yet to be designed and may be a specific design or an off the shelf proprietary system procured by the Waimea Water to the performance criteria set by WSP|Opus.
The required size of the outlet pipework has been determined to meet the design flow criteria and the required emergency dewatering capacity. The outlet works have been sized in accordance with the following standards and references:

- NZS 1170 “Structural Design Actions”.
- NZS 3101 “Concrete structures”.
- NZBC Compliance Document D1 “Access routes”.
- Tasman District Council resource consents RM140540, and RM140556 to RM140559.

### 20.2.2 Resource consent conditions

#### 20.2.2.1 General

The regulatory requirements for operation of the Waimea Dam are specified in the Resource Consents RM140540, and RM140556 to RM140559 granted by Tasman District Council (TDC) to Waimea Community Dam Limited. The resource consent conditions include specific requirements for discharge flows as summarised below.

#### 20.2.2.2 Environmental flow release

Minimum environmental release flows are specified under Condition 94 for the range of reservoir levels and require at least 510 l/sec to be released from the dam from the inflow design flood level (202.53 m RL) down to the minimum operating level (166.5 m RL). Higher flow releases may be necessary to meet the minimum flow requirements specified the Appleby Bridge flow recorder site.

There is also a requirement to release all inflows up 510 l/sec when the reservoir is below 166.5 m RL (noting lower intake bellmouth lip level of approximately 163 m RL sets the absolute minimum reservoir level for flow release). This condition effectively requires the inflows to be recorded into the dam.

Condition 95 requires direct or indirect measurement of the instantaneous rate of water release from the dam (can be from the outlet works and/or seepage). This condition also requires reservoir level measurement with an accuracy of at least +/- 5%. This condition requires the measured instantaneous flowrate and reservoir level to be provided to Council electronically in ’real time’ and an agreed format.

#### 20.2.2.3 Flushing flows

Condition 96 sets the minimum flushing flow release of 5,000 m$^3$/s for at least three hours and only at night time (10 pm to 4 am). Condition 98 requires a Flushing Flow Release Plan (FFRP) which is to include the rate of flow increase to avoid fish standing (i.e. the flow rate is required to increase slowly up to 5,000 m$^3$/s).

Condition 102 include provision for review of the effectiveness of the flushing flows two years after first filling of the reservoir. This includes recommendations to change the frequency, number or magnitude of flushing flow releases (i.e. the required flushing flowrate may be increased).
Note that an increase in the flushing flow may require changes to the outlet works.

### 20.2.3 Operational criteria

The operational flow criteria used for the design of the Waimea Dam outlet works are summarised in Table 20.2 below.

**Table 20.2: Operation criteria for the outlet works**

<table>
<thead>
<tr>
<th>Operational requirement</th>
<th>Proposed operational criteria/procedure for design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Required for dam safety and/or resource consent compliance</td>
<td></td>
</tr>
<tr>
<td>Environmental/residual flow release</td>
<td>At least 510 l/sec between 202.5 m RL and 166.5 m RL. All inflows up to 510 l/sec between 166.5 m RL and lower intake bellmouth lip level (163.0 m RL). Flow release above NTWL (197.2 m RL) includes spillway flows which are not directly measured (can be determined from reservoir level and spillway rating curve). Closing outlet valves during floods may not practical or beneficial.</td>
</tr>
<tr>
<td>Irrigation release</td>
<td>Flows from 510 l/sec up to 2230 l/sec from one or both outlets. Not set by resource consent. Peak irrigation release set in Stage 3 by demand study.</td>
</tr>
</tbody>
</table>
| Flow mixing | Flow mixing from both intakes between NTWL (197.2 m RL) and upper intake minimum operating level (185 m RL). Percentage of flow mixing is variable with head **with an approximate design ratio of 20:80 (i.e. 102 l/sec minimum flow from either intake line over this range)**. Design calculations by WSP suggest that the actual ratio will be 28:72 over most of the operating water level range.  
  - Minimum environmental flow release of 510 l/sec to be maintained.  
  - Additional smaller FCDV’s on each line (e.g. four valves in total. On each intake line a large valve for irrigation, flushing and emergency drawdown and a small valve for environmental release and flow mixing). |
<p>| Flushing flows | Flow ramping up from environmental release flow to at least 5,000 l/sec (held for 72 hrs each specified release before ramping down again) over the operating range of 197.2 m RL down to 166.5 m RL. |
| Emergency drawdown | Both intake lines operating with FCDV fully open for all reservoir levels from 202.5 m RL down. Damage to the FCDV’s may be acceptable under emergency situations provided they can fully open and the isolation valves function. |
| Isolate the outlet works discharge | Maximum emergency drawdown flow the isolation valves can close against. This load case occurs when a fully open FCDV cannot close. |
| Water level measurement | Two independent water level recorders connecting to telemetry system and transmitted to TDC control room in real time. Manual staff gauge for on site readings. Compliance with Resource Consent Condition 95. Duplicate systems (dual stilling wells and instruments) to provide redundancy as key dam safety monitoring instrument. |
| Discharge flow measurement | Compliance with Resource Consent Condition 95. An insertion type flowmeter installed in each penstock pipe at the downstream end (upstream from FCDV’s). FCDV valve opening position indicator. Flowmeters and opening position sensor connected to site telemetry system and transmitted to TDC control room in real time. |
| Intake design approach velocity | 0.3 m/s under normal operating conditions (does not include flushing or emergency drawdown flows or hydro flows). Resource consent condition 119. |</p>
<table>
<thead>
<tr>
<th>Operational requirement</th>
<th>Proposed operational criteria/procedure for design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake cleaning</td>
<td>Manual cleaning by lifting screens up face on rails to crest and cleaning from access platform on upstream face or by divers.</td>
</tr>
<tr>
<td>Penstock (pipe) pressure sensors</td>
<td>One pressure sensor on each pipeline connected to site telemetry system and transmitted to TDC control room in real time. Enables screen blockage to be identified and such that valves can be closed to prevent screen collapse.</td>
</tr>
<tr>
<td>Power supply</td>
<td>Two separate power supply systems (primary and backup). 11 kV and a diesel backup generator selected by Waimea Water.</td>
</tr>
<tr>
<td>Communications/telemetry</td>
<td>Two separate forms of reliable communication system enabling remote monitoring and operation of on site equipment. Communications system arrangements pending and could include radio, cellular network, satellite network, or fixed line (fibre).</td>
</tr>
<tr>
<td>Conduit lighting</td>
<td>Permanent lighting installed into conduit ceiling with manual operation. Required for inspections of penstock.</td>
</tr>
<tr>
<td>Conduit ventilation</td>
<td>Permanent ventilation system with manual operation prior to entry into conduit for penstock and isolation valve inspection and maintenance. A potential alternative to a ventilation system is for respirators (Breathing Apparatus) to be worn. We note that many confined spaces are being retrofitted to have ventilation systems and therefore industry practice is to provide ventilation. Physical access to the conduit should be restricted (e.g. steel mesh panel with locked door in conduit) to discourage casual entry. The dam owner will need to develop a procedure for operational access and that may include for any person that enters the conduit will require current confined space access training and the appropriate PPE including air quality meters.</td>
</tr>
</tbody>
</table>

The outlet works are required to supply relatively small residual environmental flows (e.g. 510 l/sec) over the full reservoir operating range (e.g. a 30.7 m vertical head/drawdown range) and much larger flushing and emergency dewatering flows (e.g. 5,000 – 11,000 l/sec). The reservoir operating range and flow release range are significant such that controlling discharge with a single fixed cone discharge valve for each line is difficult and would likely result in operational constraints in terms of discharge flow control.

For this reason, the Stage 4 design has adopted two FCDV’s for each outlet pipeline, with a smaller FCDV for the residual flows and lower irrigation flows and the larger FCDV for higher irrigation flows, flushing flows and emergency dewatering. This arrangement enables flexible outlet operation and flow mixing.

The proportion of flow mixing dictates the FCDV size, with the critical case being a small flow from the lower intake at a high reservoir level (e.g. 20% of environmental release flow though lower intake at NTWL). Selecting wider range of mixing flows for the purpose of valve selection increases the operational flexibility noting there is a practical limit to the minimum flow released from a valve.

The FCDV size is also set by the maximum flow requirement (i.e. flushing or emergency dewatering) and the lowest operating reservoir level. Under the emergency drawdown scenario (e.g. in the unlikely event that the dam is damaged and requires dewatering to prevent collapse) the valves may be fully opened when the reservoir level is at its maximum; which may result in damage to the valves. It is not normal practice that FCDV’s would be sized for usual operation at the peak flows under the emergency dewatering scenario. Valve size selection would be undertaken as part of the E&M works design package based on the performance specifications.

Operation of the isolation and fixed cone discharge valves will be electrically powered and automated based on set operational rules entered into the programmable logic controller (PLC). This
requires a number of instruments to be connected to the same control system (e.g. water level recorders, penstock pressure sensors, seismographs). This control system does not rely on external communications (i.e. on site communication only) but requires a reliable power supply.

It is anticipated that the PLC would be programmed for the following scenarios:

- Flow control using the FCDV’s to suit the pre-programmed total outflow based on reservoir level for:
  - Environmental flow release including intake source mixing regime.
  - Irrigation release including ramping up and down.
  - Flushing flows including ramping up and down.
  - Restrict maximum flow increase rate to prevent unsafe rapid release of large flows from the dam.
- Emergency automatic closure of the isolation valves should a fault be detected in the penstocks or FCDV’s. This requirement sets the type and size of the isolation valves.
- Emergency automatic closure of the isolation valves following a large earthquake. Manual override require to initiate emergency drawdown procedures.

It may be possible for Waimea Water to connect the Appleby Bridge flow recorder to the Waimea Dam control system which could potentially enable automated flow control based on downstream river flows (minimum flow release through the outlet works is set based on the flow recorded downstream).

Remote surveillance and operation of the valves from a remote control room is also intended with auxiliary on site operation (using the electrically powered actuators with provision for manual operation). The flow release can be manually set to override the PLC controls if necessary. This system works regardless of the number of valves installed. Remote operation and surveillance relies on the communications system set up on site (covered below).

The penstocks will also require a small diameter air vent pipe with an air release valve (and upstream isolation valve) on each line to protect the penstock pipe during priming, accidental valve closure and emergency operation situations. The size of this line depends on the air volume required and would be determined as part of the overall E&M work package.

The isolation valve will feature a bypass valve, the operation of which will be automated and included in the isolation valve operating sequences.

The provisional mini hydro scheme would operate with a single FCDV (not the likely four FCDVs). The mini hydro scheme’s mixing ability is less flexible: either one outlet or both at approximately 50:50 sharing. There are no restrictions on reservoir operation range (at Stage 3 design). The FCDV is capable of operating between flows of 510 l/s and 5,000 l/s. The hydro station can be designed to pass flows lower than 510 l/s (for the case of dam seepage contributing to environmental flow).

20.2.4 Access

A HAZOP workshop attended by the designer, the peer reviewer and representatives of WWAC was held at T+T on 27 March 2012. The workshop concentrated on the health and safety aspects of the operation of the outlet works in the conduits. A subsequent Safety in Design workshop was also held as part of the Stage 4 design. The outcomes of these workshops were documented (Refer Appendix E below) and the design has considered these aspects.

Platforms are provided to FCDV valves to enable routine inspection and maintenance. Platforms are also provided on the upstream face of the dam to assist in inspection and access to the dam face.
itself, and to provide a slinging/maintenance area for the intake structures when these are docked at the parapet wall.

The use of harnesses and appropriately skilled personnel will be required in many locations, potentially along with the use of crane assisted cages for inspection and intake removal.

Harness connection points have been provided along the parapet wall and the spillway walls for inspections and maintenance.

### 20.2.5 Ventilation

Ventilation for the conduits will be designed by WSP to enable permanent access to the conduits without the use of breathing apparatus. The including of ventilation was the outcome of a HAZOP during Stage 3 (T+T, 2012/2014) and the Safety in Design workshop in Stage 4 (2018).

### 20.2.6 Controls

Control equipment, instrumentation, telemetry and power supply for the outlet works are covered by others separately. Refer to Section 20.2.3 above and Sections 23 and 25 for a summary of the operational control requirements.

### 20.2.7 Civil works

The civil works associated with the outlet include thrust blocks, pipe supports, access platforms and fastening details. The outlet chamber at the end of the diversion culvert was specifically design to facilitate access to the outlet works.

The civil works have been designed in accordance with NZS3101, NZS1170, and the design criteria in Section 2.

### 20.3 Dewatering capacity

#### 20.3.1 Methodology

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

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

The outlet works arrangements are subject to confirmation of the E&M design by WSP | Opus.

Determining the evacuation period requires routing flows through the outlet facilities in conjunction with recommended reservoir inflows as follows:

- Reservoir filling - Inflow during filling should assume an average of the mean monthly inflows for the selected filling period as well as a flood with a recommended frequency of approximately five times the duration of the filling period.
- Reservoir evacuation - Reservoir inflows should be based on the highest consecutive mean monthly inflows for the duration of the evacuation period.

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

USBR TM3 general guidelines for determining High-Hazard dam emergency evacuation times are presented in Table 20.3. These values are based on USBR experiences and endeavour to reflect a balance between risks, hazards and costs. USBR TM3 states that the values are considered to be conservative and may be adjusted.

Table 20.3: General guide for determining emergency evacuation time (days)

<table>
<thead>
<tr>
<th>Evacuation Stage</th>
<th>High-Hazard High-Risk</th>
<th>High-Hazard Significant-Risk</th>
<th>High-Hazard Low-Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>75% Height*</td>
<td>10-20</td>
<td>20-30</td>
<td>30-40</td>
</tr>
<tr>
<td>50% Height*</td>
<td>30-40</td>
<td>40-50</td>
<td>50-60</td>
</tr>
<tr>
<td>25% Height*</td>
<td>40-50</td>
<td>50-60</td>
<td>60-70</td>
</tr>
<tr>
<td>10% Storage*</td>
<td>60-80</td>
<td>70-90</td>
<td>80-100</td>
</tr>
</tbody>
</table>

Note: Table reproduced from Table 4 in USBR TM3.

*The height and storage is considered to be measured from the NTWL to river bed level.

The Waimea Dam outlet facility has two distinct draw off levels. These can be adjusted by removing or adding pipes on the face of the dam using divers noting this is likely to be significant work. To achieve a minimum draw off level, it is also possible to disconnect the inclined pipes completely and connecting the screens directly to the thrust block above the starter dam noting this would require complete isolation of both pipelines and is expected to be significant work. For this reason removal of the inclined pipes and relocation of the screens is not allowed for in this dewatering assessment.

For the purpose of evaluating the filling scenario, an outlet rating curve was developed assuming the two distinct draw off levels shown on the Drawings. This scenario assumes:

- The pipework within the concrete conduits and the fixed cone discharge valves (FCDVs) remain in place to control the outflow.
- Only the large diameter FDCV’s are operating.
- The discharge rates through the FCDVs is not limited (i.e. velocities through the valves may exceed valve manufacturer’s recommended limits).

20.3.2 Design inflows

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

An analysis of the synthetic inflows has been carried out to determine the dam’s mean monthly inflows and the highest consecutive mean monthly inflows. Table 20.4 below presents a summary of the monthly data over the 52 year record.

Table 20.4: Waimea Dam synthetic record monthly inflow summary

<table>
<thead>
<tr>
<th>Month</th>
<th>Mean Monthly Inflow (m³/s)</th>
<th>Maximum Monthly Inflow (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>2.7</td>
<td>13.1</td>
</tr>
<tr>
<td>Month</td>
<td>Mean Monthly Inflow (m³/s)</td>
<td>Maximum Monthly Inflow (m³/s)</td>
</tr>
<tr>
<td>-------</td>
<td>---------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>Feb</td>
<td>2.0</td>
<td>9.2</td>
</tr>
<tr>
<td>Mar</td>
<td>2.5</td>
<td>11.1</td>
</tr>
<tr>
<td>Apr</td>
<td>3.5</td>
<td>14.8</td>
</tr>
<tr>
<td>May</td>
<td>3.4</td>
<td>10.8</td>
</tr>
<tr>
<td>Jun</td>
<td>4.1</td>
<td>10.7</td>
</tr>
<tr>
<td>Jul</td>
<td>4.3</td>
<td>17.2</td>
</tr>
<tr>
<td>Aug</td>
<td>4.2</td>
<td>13.2</td>
</tr>
<tr>
<td>Sep</td>
<td>4.7</td>
<td>15.7</td>
</tr>
<tr>
<td>Oct</td>
<td>4.6</td>
<td>15.1</td>
</tr>
<tr>
<td>Nov</td>
<td>3.8</td>
<td>10.9</td>
</tr>
<tr>
<td>Dec</td>
<td>3.3</td>
<td>14.3</td>
</tr>
</tbody>
</table>

NOTE: Monthly flows derived from synthetic daily record

The highest consecutive mean monthly inflows were determined by finding the maximum of a two month and three month moving average. The record shows that highest two month inflow period starts in October 2001, with a mean inflow value of 14.0 m³/s. The highest three month inflow period also starts in October 2001, with a mean inflow value of 11.8 m³/s.

20.3.3 Reservoir filling

Routing was carried out to assess reservoir filling rates and determine a design frequency storm to apply during filling. The two lowest consecutive mean monthly inflows occurred in August 1997, with a mean inflow value of 3.6 m³/s. Routing shows the reservoir could fill in less than two months using this rate with an allowance of 0.51 m³/s environmental release. Based on this relatively short filling duration, the synthetic design MAF hydrograph (without climate change) was adopted as the frequency storm to apply during filling.

The reservoir filling routing analysis assumes:
- The inclined pipework on the dam face is in place with two distinct draw off levels as shown on the Drawings.
- The outflow is not limited to the valve manufacturers’ recommended limits.

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

USBR TM3 recommends that the outlet works should have sufficient discharge capacity to maintain the reservoir levels reasonably constant for elevations above 50% of the reservoir depth for the established inflow conditions. At 50% reservoir depth only the low level outlet is available for release. However the discharge capacity of the low level outlet is greater than the mean monthly inflows presented in Table 20.4, thus we believe the outlet works will pragmatically meet the objectives of USBR TM3.

20.3.4 Reservoir evacuation

The results of the routing analysis are presented in Table 20.5 below for the mean, highest two and highest three consecutive monthly flows. The routing analysis assumes inflow goes back to the mean monthly inflow at the end of the two or three month period. The results show:
The dam could be dewatered to the lower intake height of 165 m RL in around 15 days assuming mean monthly inflows.

The outlet works do not have sufficient capacity to draw the reservoir down with the intakes in place while the highest two and three consecutive month flows occur.

With the intakes in place, dewatering to the minimum level in the highest two and three consecutive mean monthly inflow scenarios would occur approximately 30 days after the inflows reduce to the mean monthly (i.e. approximately three and four months to dewater). Comparing this to the USBR TM3 guidelines for a High-Hazard Significant-Risk dam, only the time to 10% storage criteria is met.

Based on an EV1 distribution, the highest two and three consecutive mean monthly inflow scenarios equate to events with 100 year ARI and 90 year ARI respectively. Therefore there is a relatively low probability that these scenarios could occur during a dewatering.

Both intakes are required to be lowered to the starter dam level (approx. 155 m RL) to enable dewatering in the higher flow scenarios. The time at which the intakes are relocated dictates the total drawdown time (e.g. moved when the outlet capacity matches the inflows to reduce delays to dewatering).

The dam could be substantially dewatered (50% Height or 10% Storage) in around 40 – 65 days for the two high flow scenarios, provided the pipework can be removed within the first month. We consider this to be consistent with USBR TM3 guidelines for a High-Hazard Significant-Risk dam.

### Table 20.5: Reservoir evacuation timeframes (intakes in place until reservoir level reduces to 166.5 m RL)

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Mean monthly inflows ((3.6 \text{ m}^3/\text{s}))</th>
<th>Highest two consecutive mean monthly inflows ((14.0 \text{ m}^3/\text{s}))</th>
<th>Highest three consecutive mean monthly inflows ((11.8 \text{ m}^3/\text{s}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evacuation Stage</td>
<td>Time (days)</td>
<td>Time (days)</td>
<td>Time (days)</td>
</tr>
<tr>
<td>75% Height*</td>
<td>7</td>
<td>30</td>
<td>17</td>
</tr>
<tr>
<td>50% Height*</td>
<td>20</td>
<td>75</td>
<td>74</td>
</tr>
<tr>
<td>10% Storage*</td>
<td>22</td>
<td>77</td>
<td>76</td>
</tr>
<tr>
<td>25% Height**</td>
<td>29</td>
<td>89</td>
<td>88</td>
</tr>
<tr>
<td>Minimum (21% Height**)</td>
<td>30</td>
<td>91</td>
<td>90</td>
</tr>
</tbody>
</table>

* Reservoir storage and height are measured from the NTWL to river bed level.

** Drawing the reservoir down to this level requires closure of both intakes and removal of the intakes and pipework.

ICE (2014) provide an alternative approach to reservoir dewatering that is based on a minimum recommend drawdown rate. For Category A dams (approximately equivalent to High PIC), the recommended minimum drawdown rate is 5% of full reservoir depth per day which is 2.4 m/day, with an “Upper cap on practical drawdown rate” of 1 m/day. The dewatering analysis for the Waimea Dam gives drawdown rates of 1 – 3.6 m and is consistent with the ICE guidance.

### 20.4 Outlet chamber

The outlet chamber is located at the end of the diversion culvert and provides access to the outlet works and inside the culvert/conduit. The upstream and side walls were designed as retaining walls based on the rockfill parameters stated in Section 14. Structural design for the walls included SEE seismic loading cases.
The chamber wall heights were set at the IDF tailwater level (156.6 m RL) to avoid flooding of the conduit. Sump pumps with discharge lines over the end wall are provided to drain seepage water from the conduit and rainfall over the end chamber.

The chamber opening length of approximately 8 m allows for provisional future removal of sections of pipework (cut into sections) and the isolation valves. Vehicle access (e.g. mobile crane) is allowed for up to the chamber upstream wall. Access platforms are provided over the chamber with lockable hatches and caged ladders to operator access to the conduit. Access platforms and stairs are also provided on the end wall for access to the FCDV’s and the adjacent seepage collection monitoring weirs.

The structural support members for these platforms have been designed for UDL’s as per NZS1170 and for a 500 year ARI earthquake ULS. The structural supports have been detailed to facilitate dismantling for provisional removal of pipework and valves from the chamber.

20.5 Thrust blocks and pipe supports

The outlet pipework includes two bends; the lobsterback bend at the upstream toe (where the pipe enters the conduit from the upstream face) and a 20 deg bend at the end chamber to enable the downstream FCDV’s to be mounted above the design 10 year ARI tailwater level. These bends introduce hydraulic forces and concrete thrust blocks have been provided to restrain these bends.

The thrust blocks have been designed in accordance with ASCE Steel Penstocks and NZS3101. The hydraulic loads include the emergency dewatering flow cases and transient load rejection.

The critical loading condition for the thrust blocks is the load rejection scenario, and the concrete encasement and anchor connections have been designed for this case. Anchors have been included for the downstream thrust block to stabilise this block in lieu of increasing the volume of concrete which would prevent access to the isolation valves upstream.

The steel pipes within the conduit are located off to one side to improve accessibility to the isolation valves. The pipes are simply supported on steel contact saddles which are fastened to the conduit floor with bolted anchors.
21  

Roads

21.1  

Site access

The main access to the dam site is from the Lee Valley Road, approximately 13.6 km south of the River Terrace Road/Lee Valley Road intersection in Brightwater. Road access to the forestry area is to be maintained during and following construction via the Lee Valley Road forestry road that also provides access to the dam site. Access to the dam should be controlled by appropriate security gates and fences.

T+T’s scope does not include design of improvements or upgrades for construction or permanent site access along the Lee Valley Road. T+T has communicated to Waimea Water and the Contractor that there are several areas of known instabilities (active landslides) along on Lee Valley Road access road that should be assessed by Waimea Water for both temporary (construction access) and permanent access.

21.2  

Description

Permanent road access is to be provided to the dam crest as well as to the outlet works at the toe of the dam. Access to the crest and toe of the dam will require two bridges across the spillway. The extent of the permanent access roads is shown on Drawing 27425-RDS-100.

The roads that have been designed for this project, and hence covered by this design report, are as follows:

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

- **Dam toe road** - This is a new access track to provide access to the toe of the dam. This access is via the lower bridge over the flip bucket, and the toe berm fill zone is included for this road. This road provides vehicle access to the outlet works and associated equipment, and the fish pass inlet. The toe berm level of 156.6 m RL was selected to be at the IDF tailwater level. The toe road turning area is located in cut on the true right to allow room for a provisional future powerstation.

- **Dam crest road** - The dam crest includes an access road for vehicular access to the intake screen platform and winching chamber. The embankment crest width increases at the true right abutment to provide a vehicle turning area.

The road includes 250 mm thick granular pavement (150 mm GAP65 subbase and 100 mm GAP40 basecourse placed at the end of construction) where the CBR is 20% (i.e. on weathered rock, or engineered fill). Where the CBR is less than 20% but ≥3% (i.e. roads in cut on soil), the GAP65 subbase thickness increases to 225 mm.

The dam crest and toe berm roads are chip sealed from the end of the bridges with Grade 3/5 chip and two-coat seal. The other access roads are unsealed.

Galvanised steel crash barriers (CSP Pacific Highway Flexi-Rail W-beam barrier or equivalent) with timber posts are provided at the bridges (refer Section 18 above) and along the downstream side of the dam crest road to the true right abutment.

Table/berm drains are provided on the upslope side of the road in intercept surface water for the slopes above and direct this beside the road to the nearest culvert. These drains have the same longitudinal grade as the adjacent road.
Culverts of 300 mm diameter are located underneath the road as shown on the Drawings. The inlets to these culverts are vertical DN675 precast concrete pipes with scruffy domes to discourage blockage. The culvert outlets are a standard outfall detail as per the Tasman DC Engineering Standards.

21.3 Design basis

21.3.1 Standards and references

The following standards and references have been used for the roading design:

- CSP Pacific Drawing FX360.
- NZTA (2007) “2007 On-road tracking curves vehicle: Large Rigid Truck Turn: 12.5m Radius”.

21.3.2 Geometric design criteria

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

The carriageway width will allow for one-way traffic for standard construction vehicles with occasional passing opportunities on straights. There are no specific passing bays designed, but there are turn-around areas at the right abutment on the dam crest and at the outlet works at the toe of the dam. There is also local widening at the intersection of the dam crest access road and the upper bridge to facilitate turning onto the bridge. The turning areas on the access roads, dam crest and dam toe have been assessed using Autoturn for the design 11 m long vehicle.

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

The selection of design parameters for vertical and horizontal alignments has been based on general guidelines for construction traffic and on the existing forestry access roads which the dam access roads replace. The NZTA geometric design criteria for a slightly larger 11.5 m long large rigid truck and Tasman District Council Engineering Standards and Policies have also been considered in developing the turning radii.

A summary of the geometric design parameters is shown in Table 21.1 below. The longitudinal gradient is consistent with the gradients already in use around the proposed dam site and reservoir.
Table 21.1: Road geometric design parameters

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent road width</td>
<td>4.5 m wide completed carriageway with 0.5 m deep table drain (1 m top width) when in cut and 1 m shoulder when in fill. 1 m wide shoulder on downslope side.</td>
<td>-</td>
</tr>
<tr>
<td>Construction access haul road width</td>
<td>10 m wide with 1 m wide shoulders (to enable two way vehicle movement.)</td>
<td>Included where these are intended to be incorporated into the permanent works. Dimensions as requested by FHTJV.</td>
</tr>
<tr>
<td>Horizontal curvature</td>
<td>Minimum desirable internal horizontal radius 27.5 m. Absolute minimum external turning radius of 13 m where there is no physical restriction on the inside of the bend.</td>
<td>Very low vehicle speeds apply for the minimum radius of 13 m as appropriate for hammerhead turning areas and bridge entry points.</td>
</tr>
<tr>
<td>Vertical curvature</td>
<td>Minimum vertical radius 120 m</td>
<td>K = 1.2</td>
</tr>
<tr>
<td>Longitudinal grade</td>
<td>Permanent access maximum 15% (1H:6.7V). Construction access/haul roads maximum 20% (1V:5V) (as advised by FHTJV).</td>
<td>Over 15%, additional pulling capability required and/or pavement improvements such as sealing may be required – particularly for transport of hydro equipment. Not included in the design.</td>
</tr>
<tr>
<td>Crossfall</td>
<td>Permanent roads design single cross fall of 2%. Construction access/haul roads no specific design crossfall.</td>
<td></td>
</tr>
</tbody>
</table>

21.3.3 Pavement design

The pavement layer thickness were determined on the basis of a lightly-trafficked pavement in accordance with Austroads and confirmed using CIRCLY pavement analysis software. The chipseal surfacing of Grade 3/5 chip with two-coat seal is specified based on the Tasman District Council Engineering Standards for residential sheets.

Table 21.2: Pavement design parameters

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade CBR</td>
<td>3% (soil) 20% (rock or engineered fill)</td>
<td>Subgrade CBR to be confirmed during construction.</td>
</tr>
<tr>
<td>AADT</td>
<td>0.29 (On site) 10 (Forestry access)</td>
<td>-</td>
</tr>
<tr>
<td>ESA/HVAG</td>
<td>0.4 (On site) 2.4 (Forestry access)</td>
<td>Local access in industrial area as per Austroads.</td>
</tr>
<tr>
<td>Growth factor</td>
<td>No growth allowed for.</td>
<td>-</td>
</tr>
<tr>
<td>DESA</td>
<td>2.1x10^3 (On site) 8.8x10^5 (Forestry access)</td>
<td>Lightly trafficked.</td>
</tr>
<tr>
<td>Design life</td>
<td>25 years</td>
<td>As per Tasman District Council Standards.</td>
</tr>
</tbody>
</table>

The Tasman District Council Engineering Standards require the top 150 mm of pavement to be TNZ M/4 AP40 basecourse on public roads. This requirement does not apply to private roads such as the forestry access road for access to the dam.
21.3.4 Drainage design

The road drainage system has been designed in accordance with the Tasman District Council Engineering Standards for the 20 year rainfall event with runoff calculated using the rational method.

Table drains on the roads have been designed 0.5 m deep and 1 m wide with side slopes of 1H:1V. Runoff is discharged from the drains through culverts under the road. The vee channel shaped table drains are expected to be excavated in rock and therefore scour is not anticipated to be significant. This should be monitored during operation and remedial measures installed if required.

The culverts have been sized for the 20 year ARI design rainfall event assuming that water can head up to the top of the table drains. Events larger than this may result in surface water flowing over the roads and this could erode the road surface if the culverts are not maintained.

21.4 Construction considerations

The following design considerations may apply to the construction of the access roads:

- Changes to the construction access road layout and/or grades may require realignment of the permanent access roads.
- Construction access from the true right bank of the river may enable removal of the lower bridge from the design.
- Road excavations may result in unstable slopes that require specific treatment subject to specific assessment on site during construction.
- Excavation of the plunge pool true left batter may encounter lower quality rock and require a batter slope flatter than the 1V:1H design. This may reduce the haul road width and require additional excavation to realign the permanent access road.
- The foundation conditions of the roads (where in soil rather than rock) may require additional granular fill placement (i.e. undercut and placement of subbase) where weak foundations are identified during construction. The current roads have been designed on the basis of a minimum CBR of 3% in soil and >20% when in rock or on engineered fill.
- The Contractor will need to design and install temporary stormwater drainage measures as required.
22 Fish pass

22.1 General

The fish passage requirements were extensively assessed and discussed from feasibility through the Stage 3 design and resource consent application (which was based on the Stage 3 design and work by the Cawthron Institute). The resource consent includes specific requirements for fish passage on the basis that upstream passage for target climbing species only (i.e. juvenile koaro and longfin eels) would be provided via an open channel type fish pass/ladder. Downstream passage was envisioned to be by adult eels and koaro only and be by trap and transfer arrangements.

The presented fish pass design is for upstream passage only is in accordance with the resource consent and is a development of Stage 3 design on which the resource consent was based. Separate to design, we understand that Waimea Water are in communication with the Department of Conservation (DoC) regarding the fish pass.

The downstream fish passage for adult fish is intended to be by trap and transfer methods as advised and designed by others. Consideration of adult fish being accidentally passed down the spillway has been considered by T+T, Cawthron and Waterways Consulting (meetings 7 June 2018 and 23 May 2018). Because the spillway is not gated no control is possible to prevent adult fish from passing downstream over the spillway. Fish that do go down the spillway will be able to continue downstream via the lip of the flip bucket and back into the river.

Fish passage during construction is not covered by T+T’s design or this design report.

22.2 Design basis

22.2.1 Standards and references

The following standards and references have been used for the fish pass design:

- Chow (1973) “Open Channel Hydraulics”.
- Marley “Pressure Pipe Technical Manual for PVC and polyethylene pipe systems” (pipe flow hydraulics).
- NIWA (2018) “New Zealand Fish Passage Guidelines For structures up to 4 m” (Design velocities and rest area spacing noting these are for swimming rather than climbing species).
- NZS3101 “Concrete structures”.
- USBR “Design of Small Dams” (inlet weir).

22.2.2 Basis of concept

Following a teleconference on 4 July 2012 between T+T and Cawthron, Cawthron (pers. comm.) advised that a preferred arrangement was a rock lined channel on the true right dam abutment interface. The steepness of this alignment was specifically considered and Cawthron advised that it was adequate for the target climbing species. Cawthron also commented that a fish pass of similar steepness and around 30 m total vertical climb for the same fish species operates successfully at the refurbished Brooklyn power station dam site in Motueka (refer below).

22.2.3 Precedents

The fish pass design is based on a number of operating precedent schemes for which anecdotal evidence suggest provide adequate performance for climbing species like the target species. The fish pass channel is developed from similar grouted riprap/riprap channels such as the Lake Magellan
outlet in Hamilton (Photo 22.1 below), the AMTA stream diversion (Photo 22.2 below) and the Brooklyn power station dam site in Motueka (Photo 22.3 below).

*Photo 22.1: Rock lined channel fish pass example (Lake Magellan, Hamilton).*

*Photo 22.2: Rock lined channel fish pass example (AMTA Stream diversion).*
The flushing box and discharge pipe arrangements are based on the Opuha Dam precedent for elver passes on a relatively low downstream weir (approximately 7 m high) and the main dam (approximately 50 m high). Photos 22.4 and 22.5 below shown the Opuha Dam arrangements.

22.2.4 Key design considerations

Successful performance of the fish pass is fundamentally relies on the fish pass being attractive to the target fish species and providing safe passage. Key design considerations are:

- Providing an inlet that encourages fish to enter the pass rather than continue upstream.
- Discharging a suitable attractant flow from the fish pass in terms of water quality (e.g. dissolved oxygen and temperature) and rate relative to river flows and residual flow discharge.
from the dam outlet works. Taking water directly from the reservoir surface was considered desirable due to concerns regarding potential fluctuations in water quality and reservoir level.

- Maximising the wetted perimeter of the open channel to provide an attractive surface for climbing fish.
- Providing arrangements that consider when the target fish species would migrate (both seasonally and daily).
- Providing safe passage to discourage predation and mortality due to environmental factors (e.g. heat).

22.2.5 Resource consent requirements

The resource consents (RM140540, and RM140556 to RM140559) include specific requirements for fish pass operation, and intake screening requirements for fish. These requirements are based on the design arrangements presented in Stage 3.

Resource Consent Condition 114 requires a naturalised rip rap lined channel with a pumped flow of 5 – 10 l/sec or such other means that at least achieves the same passage. The fish pass is required to operate over the migration season of the target fish species (eight months of the year) only.

Monitoring by a suitably qualified and experienced freshwater ecologist is required over the first migration season following the dam construction being completed to enable the inlet location to be recommended (i.e. the inlet is to be constructed following this recommendation). An assessment of the fish pass effectiveness is also required within the first five years of filling.

22.2.6 Performance criteria

No specific performance criteria have been set for the Waimea Dam fish pass.

Determining whether the fish pass is performing adequately is likely to be an assessment based on observations by the suitable qualified independent ecologist. These observations are anticipated to focus on the number of individual fish from a target species and their life stage (i.e. juvenile or adult) observed at the inlet, in the channel, at the flushing box and in the reservoir itself. As specified in the resource consent (Condition 118), temporary fish traps would be placed at locations of interest during migration periods to enable counting of fish to inform this assessment.

22.2.7 Hydraulic criteria

The fish pass is designed only for the target climbing species (juvenile koaro and longfin eels) and therefore the channel needs to convey flow sufficient to provide a continuous wetted margin, rather than a specific flow depth.

The key hydraulic criteria adopted for sizing the fish pass channel was a design water velocity determined in accordance with the methodology set out in the New Zealand Fish Passage Design Guidelines (April 2018). This velocity is based on the recommended velocities for migrating juvenile koaro in flow with provision for rest areas every 10 m of channel length. This requirement is not strictly applicable for climbing species as they are climbing but in the absence of specific science based design guidance this criteria has been adopted as the basis for setting the channel cross-section geometry.

The design flow rate of up to 10 l/sec down the channel was considered to improve certainty in the wetted perimeter given the sensitivity of the hydraulic calculations (especially roughness) to flow depth and channel grade. The maximum channel grade was set from the dam embankment abutment interface angle rather than adopting maximum grade and locating the channel on the downstream face (with a series of switchbacks).
Channel roughness was estimated based on the US FHWA HEC11 method for grouted riprap, and compared with the estimate roughness (as derived from known channel geometry and estimated flow and flow depths) from the constructed Lake Magellan fish pass as a check.

A design flow rate of up to 12 l/sec is adopted for the water supply system on the basis that up to 2 l/sec may spill down the flushing box (i.e. design flow is based on the channel flow with an allowance for losses).

### 22.3 Description

#### 22.3.1 Overall

The Waimea Dam fish pass consists of inlet arrangements located downstream of the dam outlet works on the true right bank of the channel, a triangular shaped concrete channel with embedded angular rock up the true right abutment contact to the dam crest, a flushing box on the crest, and a discharge pipe through the parapet wall and down the upstream face of the dam to below minimum water level with multiple slots to enable fish to exit over the full operating range.

#### 22.3.2 Inlet

An important aspect of the selected fish pass design is the ability of the inlet to attract the target species and encourage them to enter the fish pass channel. Sufficient flow of suitable quality and volume must be provided at the inlet to the channel in order to attract the fish (e.g. so that they will find the entrance to the fish pass).

As part of the Stage 3 design and following discussions with Cawthron, Fish & Game and WWAC it was agreed that the exact location of the outlet should be decided once the dam is constructed and monitoring is implemented. This is also a requirement of the resource consent (Condition 115).

The Stage 4 design has been prepared on this basis and the location of the inlet as shown on the Drawings is subject to confirmation at the end of construction. It is noted that construction of the rest of the fish pass is possible concurrent with dam fill placement.

The inlet structure features a 13 m long 1 m wide by 2.2 m high upstream concrete weir as an intentional barrier to discourage fish from migrating past the fish pass channel entry. The upstream weir features an overhanging plate (230 mm wide mild steel flat bolted to the crest) to discourage fish from passing over the weir. The crest level is lower near the sump (set based on the residual flow) (1 m long flat section) and inclined at 1V:3H towards the true left of the channel. This arrangement is intended to concentrate flow near the inlet to the fish pass channel to encourage fish towards this location.

The weir crest level is approximately 0.2 m above anticipated flushing flow (5 m³/s) tailwater level. The tailwater levels under usual operation (residual flow of 510 l/sec) and peak irrigation release (2,230 l/sec) are 0.7 m and 0.4 m respectively. The weir is likely to become drowned (i.e. lose its effectiveness as a fish barrier) above the mean annual flood event.

The weir was designed in accordance with the standard weir design formula and crest discharge coefficients as per USBR Design of Small Dams. The stability of the weir was assessed as per the NZSOLD Guidelines 2015.

The concrete sump located downstream of the weir is formed from an enclosing nib wall to give a 0.5 m deep and 1 m by 1 m sump. The function of this sump is to provide a pool during low flows to encourage to fish to enter the fish pass channel. The fish pass channel invert is set to match the base of the sump.
22.3.3 Channel

The open channel section consists of a triangular reinforced concrete channel with a single side slope of 1V:10H and embedded angular rocks at 100 mm spacing. The angular rocks are between 200 and 100 mm effective diameter and are to be placed into the fresh concrete using rubber mallets or similar.

The channel allows for up to 200 mm of flow and features nib walls on either side to contain the flow. The required minimum channel depth varies depending on slope and the adopted channel geometry allows for a nominal freeboard above the design water levels, noting the actual flow depth is highly uncertain. The fish pass provides for a wetted margin for climbing in the steep sections and swimmable flow in the flatter sections. The channel flow rate can be adjusted to improve the fish pass performance during operation.

The channel is approximately 170 m long and runs down the embankment and true right abutment interface with a grade that varies from 1% to 58%. The channel grade varies from an initial steep section of approximately 55% to 1% along the toe access berm, to a steep 80 m long section at 58% up to the dam crest, then around the edge of the crest turning area to the flushing box at 2%.

The channel features fish refuges at regular 10 m spacing in all sections (to provide rest areas for upwards migrating fish). Spat ropes are also provided in the channel to aid the climbing fish. Refuge areas consist of 200 mm diameter PVC pipe embedded in the channel with localised concrete thickening.

The channel is reinforced for shrinkage only. Rearguard type PVC waterbar are located at the concrete contraction joints to reduce leakage.

The channel may also take some runoff from the adjacent slopes during large rainfall events. Excessive flow in the channel may prevent fish passage or wash migrating fish out of the channel. The nib walls on each side of the channel and the bench on the upslope side are expected to limit the potential for high runoff flows entering the channel.

22.3.4 Water supply (pump station and pipeline)

The water supply for the fish pass consists of a small pump station, and pumping main up to the control valves and pipework at the flushing box on the dam crest. The water supply has a design flow rate of 12 l/sec.

The submersible pump is installed in a 2,300 mm diameter precast concrete manhole wet well at the toe of the dam. The pump has a duty point of 12 l/sec at 65 m head. The wet well is hydraulically connected to the river upstream of the weir via a 200 mm diameter slotted PVC pipe and loose gravel infiltration gallery.

The water supply pipe is a 125 mm outside diameter PE100 PN12.5 pipe. The pipe is buried beside the fish pass channel from the pumpstation up to the dam crest, and then under the dam crest road to the flushing box. Anchor trench blocks are included for the steep sections of pipe.

The water supply pipe connects to a series of DN100 DI fittings, a flow control valve (to enable flow adjustments), a DI reducer and steel pipe work to the 100 mm diameter spray bar within the flushing box.

22.3.5 Flushing box

The upstream end of the fish pass consists of a flushing box and a discharge pipeline to provide passage to the reservoir. The flushing box is located at the end of the fish pass channel on the true right abutment.
The main function of the flushing box is to provide a location that encourages the climbing fish species to enter the reservoir and discourage fish from remaining at the dam crest and/or climbing back up the discharge pipe.

The flushing box design is based on the Opuha Dam precedent and features a stainless steel sheet hopper encased in concrete with a connection to the discharge pipe. The flushing box includes a stainless steel sheet box cover (with a hinged roof for access) also to provide cover to the migrating fish in this area.

A 225 mm long section of 200mm dia. pipe is required to transition from the flushing box to the fish pass channel. The pipe will be located below normal water level and mussel spat ropes have been incorporated into this short section of pipe and associated concrete ramp up into the flushing box to facilitate passage. A ‘stiff broom’ finish has been specified for the short concrete ramp to aid fish passage. An overhanging lip has been incorporated at the flushing box lip tip discourage fish from climbing out of the box.

The 100 mm diameter spray bar is supported by PFC’s. The bar is perforated and spray holes and includes provision for adjustment of the bar angle to suit the observed performance of the flushing box system. Larger diameter spray bar can be retrofitted if necessary.

22.3.6 Discharge pipe

The discharge pipe consists of a 110 mm outside diameter PE pipe fastened to the upstream face of the dam (with stainless steel brackets on the plinth). The pipe size was selected based on precedent.

The discharge pipe will pass through the parapet wall and as such will require a non return valve to prevent reservoir flood from entering the fish pass. A non-return valve has been shown on the Drawings however it is expected that insitu testing will be required to assess its performance in respect of allowing fish to return to the reservoir. Therefore amendments either during or post commissioning are likely.

The crown of the pipe features slots 100 mm long by 20 mm wide at 200 mm centres to allow the fish to exit into the reservoir over the operating range of 202.53 m RL (IDF peak water level) to 166.5 m RL (minimum operating level).

22.4 Operational flow regime

The migration period of the target species is reported by Cawthron to be between November and April. This coincides with the shoulder and main irrigation seasons where the outlet works will be releasing higher flows. We anticipate that during the migration period, the outlet works will be releasing constant residual flows of 510 l/sec with irrigation releases of up to 2230 l/sec over sustained periods in the order of months.

The performance of the fish pass is likely to be significantly influenced by the outlet works operation, as this will influence the effectiveness of the attractant flows from the fish pass. A key function of the fish pass flow attractiveness to fish is the flow rate relative to the main channel flow (e.g. too low of a side channel flow may not encourage the fish to enter the fish pass).

International guidance gives a wide range of attractant flows of between 1%-15% of the main channel flows. The fish pass design flow rate of 5 - 10 l/sec gives attractant flows of 1 – 2% during residual flow release (510 l/sec) and flow percentages much lower than 1% during the peak irrigation release.

Further adjustments to the fish pass inlet may be necessary to increase attractant flows at the entry following commissioning and assessment of the fish pass. These adjustments in the future may
include installation of an additional pump in the wet well to discharge higher flows (e.g. up to 30 l/sec) into the initial section of the fish pass channel to increase attracted flows.

22.5 Uncertainties and potential mitigations

Like all fish passes, there remain a number of uncertainties associated with the fish pass design. While a number of precedents have been considered in the development of this design, no specific empirical data is available to fully determine the effectiveness of steep fish passes. There does not appear to be definitive arrangements and industry consensus regarding the suitability and effectiveness of specific fish pass designs for high dam and our target species. As such, the actual performance of the presented fish pass remains uncertain and will be subject to ongoing monitoring and assessment during operation (especially during the first few years).

This uncertainty was implicitly considered during the resource consent stage and is the basis for the resource consent conditions requiring monitoring and assessment during the first five years of operation.

These uncertainties were explicitly discussed during a number of meetings and teleconferences during the design development from the feasibility study onwards. The Stage 3 design (which has been developed further in Stage 4) was extensively discussed and agreed with WWAC (which included members from DoC, Fish and Game, and Tasman DC).

A final Stage 4 detailed design teleconference was held with Cawthron and Waimea Water’s independent ecologist Richard Allibone of Water Ways Consulting on 7 June 2018 and the following specific uncertainties were discussed along with potential future mitigations (noting this may not be an exhaustive list) as outlined in Table 22.1 below. The potential future mitigations if required are expected to be relatively minor retrofit options rather than requiring wholesale changes/reconstruction of the system.

<table>
<thead>
<tr>
<th>Identified performance uncertainty</th>
<th>Potential future mitigation options</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effectiveness of downstream barrier weir at discouraging climbing species from climbing beyond the inlet sump to the fish pass (especially during wet weather where the surrounding structures and areas are wet and potentially climable).</td>
<td>Modify the weir edge with a longer steel plate and apply additional sealant between concrete and steel plate to reduce fish climbing through small gaps.</td>
</tr>
<tr>
<td>The attractant flow rate may need to be increased to encourage fish to enter the fish pass.</td>
<td>Secondary pump added to wet well to increase the flow in the outlet section of the channel only to approx. 30 l/sec. Modification to barrier weir and inlet works to enable gravity bypass flows via fish pass.</td>
</tr>
<tr>
<td>Relative temperature of attractant flow relative to downstream river water may discourage fish from entering and climbing channel.</td>
<td>Fish pass water is obtained from the downstream channel, pumped to the crest and then discharges down the open channel which is likely to result in warmer flow than the river water. Adjusting channel water temperature is likely to be difficult. There is limited to no ability to provide vegetative shade along the fish pass alignment but a shade structure could be considered.</td>
</tr>
<tr>
<td>Predation of fish by birds and rats in rock lined channel.</td>
<td>Retrofit a predator proof fence around/over the channel (noting likely to have significant cost). Employ predator trapping programme and deterrents.</td>
</tr>
<tr>
<td>Identified performance uncertainty</td>
<td>Potential future mitigation options</td>
</tr>
<tr>
<td>---------------------------------------------------------------------------------------------------</td>
<td>------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Actual hydraulic roughness which affects water level and wetted perimeter.</td>
<td>Hydraulic roughness values may be less than those adopted for the design which reduces the water depth. Installation of additional roughing components and/or adjustment of the design flowrate to achieve acceptable performance.</td>
</tr>
<tr>
<td>Migration during the night with fish remaining in pipe refuges during the day may result in fish mortality due to overheating.</td>
<td>Install sunshades at refuge locations.</td>
</tr>
<tr>
<td>Potential for Dissolved Oxygen sag in flow down the fish channel discouraging fish passage.</td>
<td>Retrofit an oxygenator/bubbler device.</td>
</tr>
<tr>
<td>Fish attempting to climb back up the discharge pipe from the reservoir towards the flushing box and becoming exhausted and/or trapped.</td>
<td>Adjust pitch of flow spreader bar in flushing box to increase/reduce flow on discharge side. Include operational procedures for irregular manual flushing of pipe.</td>
</tr>
<tr>
<td>Predation for other fish (large eels and trout) at the outlet into the reservoir.</td>
<td>Install fish screens to keep exclude larger fish. Modify outlet pipe work.</td>
</tr>
</tbody>
</table>

Modifications to the fish pass system may be necessary during operation (post dam commissioning) (as per Table 22.1 above) should the fish pass prove to be ineffective when accessed by an independent ecologist. In this situation, subsequent retrofit modification may or may not result in satisfactory performance.

Ultimately, if the fish pass system is assessed (e.g. by the suitably qualified and independent ecologist required by the resource consent) as not having satisfactory performance for the target fish species (with or without modification) this would likely mean that major re-design of the fish pass or a trap and transfer type system (e.g. as per the downstream passage arrangements) would be required for upstream fish passage.

Installation of an upstream trap and transfer is not limited by the presented design. Provisional future retrofit of an upstream trap and transfer system is expected to have relative minor construction costs (and may be able to use the fish pass sump and weir arrangements) and long term operational costs.
23  Power, telemetry, and controls

23.1  Permanent power supply

Permanent power supply for control of valves, telemetry, pumps etc. is outside the scope of T+T’s design. The following section documents the dam safety requirements for power and the options considered during the ECI phase.

Power supply is required at the dam site for dam safety, resource consent compliance and operational purposes. The frequency of use, required reliability and capacity of the power supply system depends on the equipment being supplied (e.g. control valves for dam safety require higher reliability than the fish pass pump).

The NZSOLD DSG 2015 recommends that all gates and valves that fulfil dam safety functions and can only be electrically operated should be connected to at least two independent sources of power supply. The isolation, isolation bypass and fixed cone discharge valves include provision for on site manual operation noting in the case of the isolation valve this would require the operator to enter the conduit (which is a confined space) and hand operate the valve which is not desirable (unless under exception circumstances).

We recommend that two independent sources of power are installed at the Waimea Dam for dam safety reasons.

Two general options were considered for the provision of electrical supply to the Waimea Dam:

1  On site supply (e.g. diesel generator/s and battery backup, solar panels with battery for single instruments). We understand on site supply from two diesel generators was adopted for the costings undertaken by BondCM at Stage 3 cost review in 2015.

2  External supply from the electrical distribution network (involves extending the 11 kV Network Tasman distribution circuit to site either as overhead lines or underground cable or a combination of both). In order to provide the recommended two independent sources of power, the external supply option would also require one form of on site power supply (e.g. an onsite diesel generator).

The site electrical load (peak load or load pattern) is a function of the equipment selected and would be confirmed as part of the detailed design/procurement for this selected electrical equipment. Based on the anticipated equipment and associated power requirements, a peak load greater than 50 - 100 kW is unlikely. The anticipated power requirements suggest that on site only supply is viable (e.g. power loads do not require excessive diesel storage/transportation or batteries).

Waimea Water has instructed T+T to proceed on the basis of an external supply of 11 kV with on site backup option (the external supply itself is to be designed by others). The onsite backup supply selected by Waimea Water is a diesel generator/s to battery system with a small diesel storage tank. This arrangement requires transportation of diesel to site and the refuelling frequency would be a function of the tank size. On site power supply would also require frequent testing and maintenance to suit the type of power supply.

The reliability of the adopted power supply will depend on the systems selected. Considerations include on site capacity (i.e. no of days/weeks can operate without refuelling), machinery breakdown, easy of repair/replacement, emergency access to site, wind and snow loading (overhead lines), and lightning protection.

Waimea Water has advised they are engaging a specialist power transmission designer to design an 11 kV power cable to the site (from the network) as the primary power supply with the secondary system being an on site diesel generator. Bringing an external power line to the site provides the opportunity to install a fibre optic communications line at the same time (which is unlikely to be cost
effective on its own). Waimea Water has instructed T+T to select a termination location for the main power supply because the power supply to site will not be procurement by Waimea Water until a later date. There is a risk that a different location will be identified by the transmission line designer requiring changes to the onsite system.

The selection and design of the power supply systems and onsite electrical distribution will be by a specialist power supply designer/supplier to meet the requirements of the performance specifications and subject to the approval of Waimea Water. This approach is intended to result in a best for project outcome.

WSP | Opus has also advised that the provisional future mini hydro scheme could export power to the grid through a 11 kV transmission line, although to take advantage of all the energy available at the dam would require the installation of a more expensive 22 kV line (refer to Stage 3 Design Report).

23.2 Communications/telemetry

23.2.1 General

The communications/telemetry system(s) for the Waimea Dam would be selected with consideration of the reliability, cost (construction and operation), and instrumentation and control system bandwidth requirements.

For the Waimea Dam, it is recommend that two separate communication systems are installed for reliability (e.g. cellular network goes offline but the radio network remains operational). Typically, the communication system(s) for a High PIC dam that requires remote operation and surveillance use one or more of the following:

- Cellular network (noting there is no cell phone coverage currently at site and therefore requiring a new tower for each network).
- Satellite network.
- Radio network (noting currently the forestry operators use a radio network in the area).
- Fixed line internet (e.g. copper phone line or fibre where available).

Cellular and radio networks are typically installed for remote sites and can be used for general operational communications. This is beneficial where operational staff are working alone in remote areas.

Fibre has the benefit of significant data bandwidth which means higher resolution data can be transmitted from the site (i.e. security cameras). If external power supply was brought to site (i.e. by overhead or buried power cable) then a fibre line could be installed at the same time.

All communication system types require ongoing testing and maintenance from specialist contractors.

The site is expected to require a SCADA/RTU system to enable remote surveillance, access and control which would be connected to the telemetry system(s) (e.g. could include separate on site and external systems). This data would be received/transmitted at/from the control room and could also include a web based access system (which is especially useful for the ongoing routine of dam safety monitoring of items such as drain flows, reservoir level, and rainfall).

23.2.2 Fish pass

The fish pass pump can be operated using on site control only by use of a PLC connected to the water level recorder in the wet well (to enable the pump to turn off in a flood and/or if the well runs
dry) and the pressure sensor on the discharge side (to switch the pump off should the pipe become blocked).

The flowmeter on the fish pass pump discharge line is intended for monitoring of the discharge flows and can also be used to check the pump performance. Installation of this flowmeter is not a requirement of the resource consent but has been selected by Waimea Water for operational reasons. An alternative measure of monitoring pump operation and discharge would be via the electrical pump on/off switches and the pump curve.

Real time monitoring of the fish pass pump is also allowed for via connection to the site SCADA/Control system and telemetry/communication system.

23.3 Security cameras and alarms

High resolution security cameras are desirable at the site for surveillance and security purposes and monitoring given the dam is remote and likely to be unmanned. Installation of cameras is not a dam safety requirement.

Three cameras are specified on the Drawings at the following locations:

- One at each spillway bridge being the primary access points to the dam crest and the outlet works.
- One at the right abutment end of the dam crest.

The water spray from operation of the FCDV’s may mean locating a camera near the outlet works is not likely to give useful imagery and for this reason a camera has not be specified for this location.

The cameras are to be connected to the communications systems for real time monitoring. Motion activated alerts could also be included with data sent to the external control room.

The cost of a camera system depends on the type and resolution selected. If the selected communications systems have sufficient bandwidth then high quality real time monitoring cameras may be suitable. Otherwise, battery operated motion activated cameras with on site storage may be preferred (and are typically much lower cost).

Security alarms may be desirable to alert the control room of unauthorised access to the control system and communication system cabinets and/or outlet works area. This could include audible on site alarms.

Fire alarms may also be desirable to alert the control room of electrical fires from the equipment on site, noting the risk of electrical fires on the dam to the surrounding forestry area may warrant further consideration.

23.4 Associated civil works

23.4.1 General

The civil works associated with the power, telemetry and control equipment consist of the control building foundation, diesel generator and fuel tank pads, and other miscellaneous concrete works.

23.4.2 Design basis

23.4.2.1 Standards and references

The civil works associated with the power, telemetry and control equipment have been designed in accordance with the following standards and references:
- NZS1170 “Structural design actions”.
- NZS3101 “Concrete structures”.

Seismic loading cases have not been specifically considered for these minor works.
24    Dam safety instrumentation

24.1    General

The dam safety instrumentation specified for the Waimea Dam (i.e. the instruments to be used for surveillance of the dam) has been developed based on:

1    Typical instrumentation for CFRD’s (as per ICOLD Bulletin 141).
2    Instruments likely to facilitate early warning of one or more of the assessed credible potential failure modes identified during the design stage failure modes effects analysis workshop (March 2018, refer Appendix E below).
3    NZSOLD Dam Safety Guidelines 2015.

The key performance indicators of dam safety that require monitoring are reservoir water level, seepage through the embankment, settlement of the embankment and at site seismic accelerations. Specific instrumentation is included for these performance indicators.

Measurement requirements, intervals, and procedures for these instruments are covered in the surveillance manual (issued separately). Draft performance criteria are also set for these instruments (including alarm/alert criteria), noting that these are subject to confirmation during commissioning (refer to separate commissioning plan) and following establishment of a long term data record and trends.

Instrumentation that is purely for operational requirements (i.e. pump starts) rather than dam safety is not covered in this report section.

24.2    Description

The dam safety instrumentation consists of the following instruments as shown on the Drawings:

- Two reservoir water level loggers to measure water level from the IDF peak water level of 202.53 m RL down to the minimum operating level of 166.5 m RL and a barometric pressure logger.
- Manual staff gauges at the spillway ogee and on the upstream face of the dam.
- One rain gauge at the dam.
- An embankment seepage collection system at the toe of the dam consisting of a geomembrane faced rockfill bund and perforated HDPE pipe collector drains.
- Spillway underdrains.
- Four seepage measurement weirs with water level loggers for toe seepage and spillway underdrains.
- Settlement pins on the bridges, spillway chute wall and parapet wall.
- Survey constellation pillars to enable settlement survey.
- One profilometer buried under the crest at the top of the Zone 3B material to measure longitudinal embankment settlement (including access points at either end for measurement).
- Three settlement plate instruments located along the crest.
- Four insertion flowmeters in the outlet pipework upstream of the fixed cone discharge valves.
- Two pressure sensors on each outlet pipeline near the upstream isolation valve.
- Valve position indicators for the isolation and FCD valves.
- Security cameras on the dam crest and toe berm.
Relatively low dams (e.g. around 50 m high such as the Waimea Dam) that are constructed in a conventional manner with suitable rockfill, typically do not have specialised instrumentation such as inclinometers and crack meters installed.

Additional operational instrumentation for the fish pass consists of a full bore flowmeter, pressure sensors, wet well water level meter and pump on/off switches.

The dam safety and operational instruments that are required for real time monitoring and operation of the scheme are connected to the onsite telemetry system, and the communications system (i.e. from the on site control building to the external operations room). This enables remote operation and monitoring.

24.3 Design basis

24.3.1 Standards and references

The following standards and references have been used for the design of the dam safety instrumentation:

- Cruz et al. (2009) “Concrete face rockfill dams” - Design seepage flow.
- NZSOLD Dam Safety Guidelines 2015.
- NZS 1170 “Structural Design actions” - Geotechnical soil loads as per embankment design.
- NZS 3101 “Concrete structures”.

Where proprietary instruments have been specified (e.g. water level loggers), the design of these items is by the manufacturer to the performance requirements set by T+T.

24.3.2 Embankment seepage collection system

24.3.2.1 Concept

Collection of seepage at the toe of the dam is considered necessary to enable assessment of the effectiveness of the grouting and concrete face, and monitoring for early indications of unusual behaviour associated with potential failure modes. The Stage 3 seepage collection concept consisted of a low rockfill bund with geomembrane liner, collector pipe work and measurement weirs at the downstream toe. This concept has been developed further for Stage 4 with refinements to the design levels, pipe and weir sizes and connection details.

24.3.2.2 Alternatives considered

A number of alternative arrangements were considered to enable collection and monitoring of embankment seepage. These arrangements included:

- Reinforced concrete retaining wall at the toe of the downstream berm.
- A low concrete collector channel at the toe of the downstream berm with re-profiling of the Lee River downstream to reduce the tailwater levels at the collector channel.
- Higher and lower bund heights for the Stage 3 concept.
The adopted Stage 4 arrangements were selected with consideration of constructability, cost, durability and effectiveness.

24.3.2.3 Seepage collection bund

The seepage collection bund comprises of a rockfill (Zone 3F) bund located at the toe of the main embankment. The crest of the bund is set at 152.10 m RL to enable seepage collection and measurement via the collector pipes and measurement weirs (refer below). The maximum bund height is approximately 4 m (subject to confirmation of the foundation excavation profiles).

The bund comprises of two sections, one either side of the diversion culvert. The total bund crest length is approximately 58 m, with 46 m on the true left, and 12 m on true right of culvert. The bund is incorporated into the toe berm (also Zone 3F rockfill) and therefore will not be visible following completion of construction.

The upstream face of the bund features a geomembrane liner as the seepage control (refer below), and a perforated PE100 collector pipe (refer below) to drain the collected seepage into the measurement weirs.

The downstream face of the bund features a 1.5 m thick (horizontally) rock armour facing zone which extends up the toe berm to the toe access road. This armour zone is included to provide nominal scour protection to the bund during large flood events. Given the waves is this area are likely to result from turbulent eddies and other complex flow conditions, sizing of this armour layer has been based on experience and precedent. The armour is also exposed and accessible for routine surveillance and repair following flood events, noting this zone does not affect the integrity of the dam.

In the instance that the mini hydro power station is added then the monitoring on the true right will need to be relocated and redesigned. However until that occurs this bund needs to be installed such that any flows can be understood.

24.3.2.4 Geomembrane

The geomembrane runs up the downstream face of the reinforced rockfill (at 1V:1.5H) before folding back at 1V:1.5H up to the concrete anchor slab on the seepage bund crest (Zone 3F). The geomembrane is fully buried underneath the toe berm rockfill (Zone 3F) and the downstream armour layer.

The geomembrane consists of a 2.0 mm thick black HDPE (Geoshield or equivalent) liner with geotextile cushion layers top and bottom on a porous concrete base with a. The purpose of the cushion layers is to reduce the potential for puncture damage to the HDPE liner from placement on the reinforced rockfill toe (Zone 3G) and subsequent toe berm rockfill placement (Zone 3F).

The HDPE liner is fastened to the reinforced rockfill concrete toe slab (formed as part of the temporary works), outside walls of the diversion culvert, and crest anchor slab with stainless steel battens, Chemset type bolts and neoprene gaskets. The crest anchor slab is a 200 mm thick by 400 mm wide reinforced concrete slab formed specifically for the purpose of fastening the HDPE (in lieu of an anchor trench due to space constraints). This slab is also located on the abutments at the termination extents of the geomembrane connecting the crest anchor slab to the concrete toe slab.

24.3.2.5 Collector pipework

The embankment toe seepage collection system pipe work consists of 35SOD PE100 SDR 13.6 solid wall pipe with 6.5 mm diameter drilled holes at 76 mm spacings on the sides as per the NZTA Specification F/2. The pipe surround is a specially screened drainage metal (DM20/6) with a D100 of 20 mm, D50 > 10 mm and no material smaller than 6 mm.
The perforation hole diameter considers the FEMA (2011) recommendations for perforated drain pipe within embankment dams with the opening size being no greater than the D_{50} of the surrounding material. The commonly specified Nexus Hi-way type drains have perforations of 6 mm but only go up to 200OD size. The hole spacing is based on TNZ F/2 and was reviewed using a standard orifice equation with a discharge coefficient of 0.62 and the design flow rate below.

The design flowrate adopted for the pipe sizing was 150 l/sec as determined with consideration of reported flows in Cruz et al. (2009) from a limited number of similar operational CFRD’s (range of 3 – 16 l/sec), foundation seepage modelling results, and allowing for a capacity factor of safety of 10. The 355OD (approx. 300 mm internal diameter) pipe was selected based on a grade of 1% and a design flowrate of up 150 l/sec for each pipe (noting this includes a factor of safety of at least 10, especially for the true right seepage collection area).

The actual seepage flowrates that would be collected from the dam are highly uncertain and are a function of the effectiveness of the plinth grouting and foundation treatment, the seepage collection bund, and condition of the concrete face. Significant increases in seepage are possible should localised damage occur to the concrete face (e.g. large cracks and damage to the waterstopped perimetric and vertical joints).

Damage to the concrete face may occur following large seismic events (e.g. larger than the OBE) and this could result in significantly larger seepage flowrates than the usual post construction steady state flows (refer Section 14 for range of post SEE seepage estimates). In this instance it is expected that seepage could exceed the capacity of the drainage collection system in which case seepage could flow over the crest of the seepage collection bund and may be visible through the downstream face of the toe berm and/or dam.

As per Section 14, the dam is assessed as meeting the design stability criteria (for the design static and aftershock seismic cases) under seepage flow rates of up to approximately 400 l/sec (i.e. much larger than the seepage collection system).

24.3.2.6 Flow measurement weirs

There are two flow measurement weirs as part of the seepage monitoring system, one either side of the diversion bund. These weirs consist of reinforced concrete weir structures founded on Zone 3F rockfill at elevation 150.4 m RL. The 1.2 m high headwall and side walls of the weir retains the adjacent rock armour facing.

These weirs are accessed from the outlet works platforms via a Webforge type steel accessway. The access way features handrails (Monowills type) as a primary fall restraint for operational staff who will need to access the weirs on at least a weekly basis. The accessway sits on the top of the weir walls at 151.6 m RL and enables measurement up to the 10 year ARI design flood tailwater.

The specified 90 deg angle V notch weir plates have been designed for a design flow rate of up to 150 l/sec at 420 mm head (as per guidance in USBR Water Measurement Manual Chapter 7), noting higher flowrates are also possible.

It is noted that the spillway underdrain flows may be significantly lower than those from the embankment seepage collection system and as such a different V notch plate shape may be deemed more suitable following commissioning.

The invert of the V notch is set 100 mm above the 10 year ARI design flood peak tailwater level at 150.9 m RL. Automated flow measurement and recording of seepage flows is provided for by a water level logger located in a stilling well pipe in the back corner of the weir box (i.e. well away from the weir plate). The flow rate can be automatically calculated from the water head using a calibrated weir equation. Manual reading of the weir flows to check the accuracy of the automated reading would be undertaken using a bucket and stopwatch.
24.3.3 Settlement instruments

The following settlement instruments are specified to enable the long term embankment settlement to be monitored:

- Three metal settlement plates on the dam crest. These instruments consist of a 600 mm by 600 mm HDG MS plate founded on the top of the Zone 3B rockfill with a steel measurement rod in a PVC tube housing that extends to the dam crest. The level of the top of the steel rod is surveyed to determine the level of the buried Zone 3B rockfill.
- One profilometer buried in a 63OD PE100 pipe underneath the dam crest (with two instrument sections; true left and true right). This instrument enables measurement of embankment settlement over the length of the dam crest. The instruments are accessed from trafficable reinforced concrete chambers (with DI lids) at each end of the crest and read with a portable unit form the adjacent concrete pad provided for this purpose.
- Metal settlement pins are specified at 50 m spacings along the parapet wall and the spillway true right chute wall to enable survey and ongoing deformation assessment of these structures.

Metal settlement pins are also specified for the bridges (located on the outside kerbs) to enable survey and ongoing deformation assessment of these structures. Survey of these instruments will require a survey pillar constellation to be installed (to be designed by specialist registered surveyor).

24.4 Reservoir monitoring equipment

Remote monitoring and recording of the reservoir water level (and therefore operation of the service spillway) is provided by two independent reservoir water level probes and loggers located within metal pipe sleeves fastened on the concrete face either side of the intakes down to the minimum operating level of 166.5 m RL. An additional water level logger is also provided for additional monitoring of spillway operation at the ogee crest.

The electronic water level probes are backed up by staff gauges at the spillway and on the upstream face down the right abutment to enable manual reading of water level should the electronic instruments be out of service.

A rain gauge with data logger is provided at the dam crest to enable interpretation of seepage results during and following large rainfall events, and early warning of potentially large river floods.

24.5 Seismographs

Two seismograph sensors have been specified. One at the dam crest (housed in the winching chamber) and one at the toe of the dam (within the control building or on the adjacent rock slope).

The purpose of these instruments is to facilitate an appropriate and proportional level of response to an earthquake event. For example, low level seismic events would not trigger a special inspection, whereas a large event closer to the SEE would trigger a range of emergency action procedures including rapid review of instrumentation and site inspection (e.g. via helicopter).

The onsite seismographs enable the at site seismic intensity to be recorded and should be linked to telemetry to enable alerts to be automatically raised with operational staff. At site measurement is important as regional level seismographs are affected by the proximity to the source of a large earthquake, and require interpretation to estimate the at site intensity which introduces additional uncertainty. Sensors at the crest and near foundation rock level enable direct measurement of the accelerations applied to the embankment and appurtenant structures for use in monitoring, and understanding the response of the dam to seismic events.
24.6 Outlet works

The outlet works rely on key instrumentation for safe and effective operation. This key instrumentation is described below:

The pipework includes pressure sensors at the upstream end to enable monitoring of potential screen blockage (i.e. low pressure alarms due to flow restriction leading to closure of the isolation valve). These sensors would enable controls to be set to reduce the potential for pipe buckling due to negative pressures (noting aeration system for pipework is also included at downstream end).

Insertion type flowmeters are located at the outlet just upstream of the FCDV’s to enable monitoring of discharges and to enable automatic controls should an emergency event occur. These flowmeters are used as the primary means of setting operation flows at the outlet works (i.e. controls to release a set flowrate would rely in the flowmeters rather than just valve opening percentages).

Valve position indicators are included on the isolation valves and FCDV’s to enable confirmation of the valve positions and secondary estimation of discharge flows.

24.7 Uncertainties

24.7.1 Performance

The seepage collection and monitoring system for the dam has been sized based on limited flow records from other dams. Following commissioning it may be identified that the usual seepage flow range (relate to reservoir level) is lower/higher than adopted for the design and this may require amendments to the arrangements.

The ‘V’ notch type of monitoring weir typically provides reasonable flow measurement over its design flow range, with reduced accuracy outside this range. The presented design enables the steel weir plate to be readily removed and replaced with other plate should lower flows than the adopted design flow occur (e.g. on the true right of the culvert).

Other weir types such as H flumes can enable more accurate measurement over a wider range of flows and could also be retrofitted to the weir box in the future if required.

Algal build up within the flume box is a potential issue that could affect the accuracy of the water level (and therefore flow) measurement. This is typically addressed by a regular cleaning programme which relies on safe access to the weirs.

24.7.2 Additional instrumentation

During construction it may be identified that additional instrumentation is required. This additional instrumentation may include additional:

- Settlement pins.
- Crest settlement plates.
- Profilometers.
- Drains and drain flow monitoring points.
- Foundation piezometers (vibrating wire and/or standpipes).
25 Operational requirements

25.1 General

The regulatory requirements for operation of the Waimea Dam are specified in the Resource Consents RM140540, and RM140556 to RM140559 granted by Tasman District Council (TDC) to Waimea Community Dam Limited. The resource consents do not cover specifics around routine operation and surveillance and emergency operation of the facility.

The resource consents do not cover specifics around routine operation and surveillance and emergency operation of the facility.

Conditions 92 and 93 outline the requirement for an Operational Management Plan (OMP) to be prepared for and certified by Council (Tasman District Council) prior to commencement of reservoir filling. This OMP is to include procedures and frequencies for dam surveillance and dam safety, and assessment and management of floating debris in the reservoir.

Current good industry practice for dam safety is outlined in the NZSOLD DSG 2015 and includes preparation and implementation of a Dam Safety Management System. This DSMS includes specific documents for the operation, surveillance and maintenance of the dam and emergency action procedures (EAP). Draft OMS and EAP documents were prepared as part of the Stage 3 design and will be updated and finalised for the Stage 4 design. The DSMS is expected to comply with the OMP requirements specified by resource consent Conditions 92 and 93.

These requirements will need revision following completion of detailed design, procurement of M&E items, commissioning and appointment of a dam operator.

25.2 Specific resource consent conditions

25.2.1 General

The resource consents outline specific operational requirements that relate to the outlet works, fish pass and reservoir water quality. The operation requirements for the outlet works are summarised in Section 20 and for the fish pass in Section 22.

25.2.2 Reservoir water quality sampling

Condition 106 requires monitoring of the reservoir water quality at or near the deepest point in the reservoir. This includes monthly manual water sampling (e.g. from a boat in the reservoir) and laboratory testing for a range of parameters. Condition 106 also requires continuous measurement and recording (hourly logged values) of reservoir temperature (at eight levels) and dissolved oxygen (at three levels continuously from November to April inclusive).

We understand that others are advising Waimea Water on resource consent compliance and will provide specific guidance and direction on the intended sampling methods and associated telemetry required.

25.3 Intake screen cleaning and maintenance

Cleaning and maintenance of the intake screens will be required at regular intervals over the operating life of the structure. The design includes a winch and rail system to enable a diver to attach the winch cable to the intake screen structure and a winch on the dam crest to haul the intake screen structure up to the crest via rails fastened to the concrete face. Pulling an intake screen structure out of the water is not envisaged as a regular activity (e.g. for maintenance only), with condition assessments intended to be undertaken using divers.
The depth of the screens below reservoir level, the screen opening size (20 mm) and design velocity (0.3 m/s) reduce the potential for debris to become pinned against the screen. Fouling of the screens due to algal growth is possible noting that the screen opening and design blockage allowance reduce the effects of algal growth. This means routine screen cleaning is expected to be an annual or less frequent event.

An alternative to manual cleaning of the intakes is to install an automatic cleaning system such as a compressed air system. Compressed air cleaning systems are highly specialised and we understand that these generally require a large air volume to be effective (based on pipe length, screen area and water levels). This type of system is likely to be expensive noting specific costs would need to be confirmed based on a specific design. We understand that this type of system could be retrofitted if frequent manual cleaning of the screens was found to be necessary noting this would likely be difficult.
26 Electrical and mechanical design

26.1 Design summary

The electrical and mechanical design for the Waimea Dam was undertaken by WSP and included the following elements:

- Outlet works pipework and valves, including:
  - DN1000 inclined pipework with slide support carriages on the upstream face (attached to rails, and finishing at intake screens to be designed by others).
  - DN1000 Lobsterback compound mitre bend.
  - DN1000 pipework within the conduit with thrust type dismantling joints and bolted flanged joints.
  - 20 deg bend and DN1000 to DN850 reducer with thrust flange (to be encased in concrete thrust block).
  - DN850 and DN300 pipe works including offtake fitting 90 deg bend and reducer.
  - Procurement specification for valves and actuators to suit penstock mechanical design.

- Electrical design including:
  - Primary power supply connection in conjunction with procurement specification for transformer (to 11 kV transmission line brought to site and designed by others).
  - Backup power supply provisions in conjunction with performance specification for diesel backup generator.
  - Power distribution design to dam equipment and instrumentation including cables, cable routing, and access/pull pits.
  - Lighting.
  - Earthing.

The mechanical and electrical design does not include the follow items which are to be procured by the Contractor to meet the performance specifications:

- Intake screens.

Similarly, the following specific plant items are to be procured by the Contractor to meet the procurement specifications:

- Crest winch.
- Fixed cone discharge valves.
- Isolation butterfly valves and bypass valves.
- Air release valves.
- Penstock drains.
- Fish pass pump.
- Conduit sump pump.
- Conduit ventilation system.
- Lighting.
- Communications systems.
- Control systems.
Further details on the electrical and mechanical design are presented in the WSP design documents enclosed in Appendix H. Electrical and mechanical specifications are enclosed in Appendix B.
27  Construction considerations

27.1  Reservoir clearance

We understand that the Contractor is responsible for forestry clearance in the reservoir in accordance with the Vegetation Clearance Plan prepared and advised by others.

We recommend that the debris is removed prior to commencing dam construction works and that the Contractor regularly monitors the reservoir and the weather to identify any areas of debris, wind felled trees that may be washed into the river during diversion, and takes action to mitigate as required.

27.2  Construction diversion

Refer Section 7.

27.3  Bridge assembly

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

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

27.4  Plinth

The following construction details were assumed for the plinth:

- Excavation of the plinth on the true right may require intermediate benches and specific rock slope protection to suit the encountered rock. Rock slope protection (temporary and permanent) requires confirmation on site once defects have been mapped and stability analyses have been undertaken.
- Allowance should be made during pricing for placement of site concrete under the plinth.
- The setout for the plinth alignment was selected to maintain a plinth excavation bench longitudinal slope of not greater than 30 deg to suit construction as requested by FH-Taylors.
JV. Adjustments to the setout may be necessary to suit the encountered rock quality which may require steeper sections of plinth and/or further excavation into the abutment slopes (esp. true right abutment).

- The design provides for a two stage construction pour of the plinth concrete, with the horizontal slab poured first and then the plinth head.
- The temporary protection shown on the Drawings for the perimetric joint water stops is indicative only. All temporary works are by the Contractor.

### 27.5 Parapet wall

The parapet wall design provides for either in-situ or precast concrete wall stems. If in situ stems are constructed, formwork will be required to construct the wall stems that will be located on the dam crest. Safety from falling for construction staff will need to be addressed given the working at height issues.

The crest width provides limited working area for construction of the crest ramp. Allowance should be made for placement of site concrete under the crest ramp base.

The ramp is to be backfilled with compacted rockfill. The crest road is a 400 mm thick road pavement with a chipseal surface.

### 27.6 Fish pass

Construction of the fish pass should be consider the following:

- The fish pass channel features a very steep section (58%) running down the true right interface of the downstream shoulder and the abutment rock. Construction access and fall restraint arrangements will require consideration.
- Work within the river bed to form the upstream weir and sump may require temporary diversion structures and/or closure of the outlet works. Dewatering may also be necessary.
- Excavation of the fish pass channel on the true right may require intermediate benches and specific rock slope protection to suit the encountered rock. Rock slope protection (temporary and permanent) requires confirmation on site once defects have been mapped and stability analyses have been undertaken.
- Allowance should be made for placement of site concrete under the concrete channel.

### 27.7 Concrete works

Mass concrete mix design and concrete placement requires specific specialist inputs and is different for conventional reinforced concrete requirements. Hot and cold weather concrete requirements and methodologies should be considered by the Contractor. Too rapid changes in concrete temperature would result in thermal shock and cracking of the mass concrete requiring rework and/or extensive grouting.

### 27.8 Designer inspections

Designer inspections will be required throughout the construction. Full time observation and designer input is required in accordance with the NZSOLD Guidelines 2015. The key items that will require designer input include:

- All foundations shall be inspected by the Foundation Committee and approved prior to placement of material.
- Permanent slope face mapping and protection works.
- Rockfill borrow operations.
- Rockfill placement and compaction.
- Prepour inspection for all reinforced concrete structures (especially the diversion culvert, spillway and plinth).
- Grouting.
- Commissioning.
28 Design work by others

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

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

Table 28.1 summarises the main contractor design elements. Note that this is not exhaustive.

We further understand that Waimea Water is responsible for procuring (including design) of other items such as:

- The dam intakes and screen.
- The permanent power supply (transmission line) to the dam toe.
- Dam permanent road access.

Table 28.1: Contractor design elements

<table>
<thead>
<tr>
<th>Location</th>
<th>Item</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>Craneage and crane platforms</td>
</tr>
<tr>
<td></td>
<td>Haul and access roads</td>
</tr>
<tr>
<td></td>
<td>Borrow and spoil disposal areas</td>
</tr>
<tr>
<td></td>
<td>Contractor power supply and utilities</td>
</tr>
<tr>
<td></td>
<td>Temporary slope protection (cut or fill)</td>
</tr>
<tr>
<td></td>
<td>Erosion and sediment control measures</td>
</tr>
<tr>
<td></td>
<td>All temporary works</td>
</tr>
<tr>
<td>Diversion</td>
<td>Debris protection</td>
</tr>
<tr>
<td></td>
<td>Coffer dam(s)</td>
</tr>
<tr>
<td></td>
<td>Mesh protection</td>
</tr>
<tr>
<td></td>
<td>Height and extent of quickrise bund</td>
</tr>
<tr>
<td></td>
<td>Diversion wall</td>
</tr>
<tr>
<td></td>
<td>600 dia diversion pipe and inlet and valves</td>
</tr>
<tr>
<td></td>
<td>Temporary slope protection</td>
</tr>
<tr>
<td></td>
<td>Temporary inlet stoplogs</td>
</tr>
<tr>
<td>Outlet works</td>
<td>Gantry crane in conduit</td>
</tr>
<tr>
<td></td>
<td>Valves</td>
</tr>
<tr>
<td>Bridges</td>
<td>Temporary stability (including propping) during construction</td>
</tr>
<tr>
<td></td>
<td>Beam splices</td>
</tr>
<tr>
<td><strong>Concrete works</strong></td>
<td>All formwork and falsework</td>
</tr>
</tbody>
</table>
29 Dam safety management system documents

Draft dam safety management system (DSMS) documentation has been prepared in parallel with the design and in accordance with the NZSOLD Guidelines 2015. The draft documentation is attached in Appendix G, noting updates to these documents will be necessary following construction and commissioning along with the necessary inputs for the Dam Owner and Operator. The following draft documents have been prepared as attached:

- Emergency Action Plan (requires further input from Waimea Water).
- Commissioning procedures (not included in first issue).

Additional documents covering governance and organisation aspects will be necessary as part of the DSMS and these should be prepared by Waimea Water prior to commissioning of the dam.

DSMS's are live documents that require regular use, review and updating.
30 References

12. Brazilian Committee on Dams (2002) “Large Brazilian Spillways: An Overview of Brazilian Practice and Experience in Designing and Building Spillways for Large Dams”.
38 ICE (2014) “Guide to drawdown capacity for reservoir safety”.
40 ICOLD (1986) “Bulletin 48a River control during dam construction”.
51 Khatsuria (2005) “Hydraulics of Spillways and Energy Dissipators”.
59 Mason P. J. (1993) Practical guidelines for the design of flip buckets and plunge pools.
67 New South Wales Dam Safety Committee (2010). Demonstration of Safety for Dams – DSC2D.
82. Standards New Zealand AS/NZS 1170.0 to 5 “Structural Design Actions”.
83. Standards New Zealand AS/NZS 3101 “Concrete Structures”.
84. Standards New Zealand AS/NZS 3404 “Steel structures”.
89. Tasman District Council (2013) “Engineering Standards & Policies 2013 Section 7 Stormwater and Drainage”.
108 USBR (1971) “Uplift Control on Spillways for Dams: A Value Engineering Study by Team”.
31 Applicability

This report has been prepared for the exclusive use of our client Tasman District Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:
- Dominic Fletcher (CPEng), Design Manager, Senior Water and Dams Engineer
- Mark Taylor (CPEng), Project Manager, Senior Civil Engineer
- David Leong (CPEng, FEngNZ), Senior Water Resources Engineer
- Dewi Knappstein (CPEng), Senior Water and Dams Engineer
- Eric Guilleminet, Senior CFRD Specialist
- Philippe Cazalis de Fondoue, Senior CFRD Specialist

Reviewed by:
- John Grimston (CPEng), Technical Director, Senior Hydropower and Dams Engineer
- Mark Foley, Project Director, Senior Engineering Geologist
- David Bouma (CPEng), Senior Dams Engineer

Authorised for Tonkin & Taylor Ltd by:

...........................…...............
John Grimston
Technical Director

DAF t:\auckland\projects\27425\27425.100\workingmaterial\27 stage 4 documentation\stage 4 design report\2019-01-24.daf.rpt.waimea dam stage 4 detailed design report rev 08.docx
Appendix A: Producer statements

- Mott MacDonald Design Producer Statement PS1
- T+T Design Producer Statement PS1
- WSP Design Producer Statements PS1
- WSP Opus Producer Statement PS2
Appendix B: Specifications

- T+T Civil and Dam Specification
- WSP Electrical Specification
- WSP Mechanical Pipework Specification
- WSP Mechanical Pipework Installation Specification
- WSP Procurement Specifications
- WSP Performance Specifications
Appendix C: Drawings

- T+T Civil Works Drawings (Bound separately)
- WSP Electrical and Mechanical Works Drawings (Bound separately)
Appendix D: Failure Modes and Effects Analysis

- Failure Modes and Effects Analysis workshop meeting minutes
Date: 8th March 2018
Time: 9am-4.30pm
Venue: Brightwater

Waimea Community Dam - ECI
Failure Modes & effects Analysis (FEMA) Workshop

AGENDA

Date: 19 (preferred) or 20 March 2018

Attendees:
Fulton Hogan  Peter Wissel
Taylors Contracting  TBA
Waimea Water  Andy Nelson
TDC  Richard Kirby or Joseph Thomas
OPUS  Ian Walsh
Damwatch  Ian Davidson
Tonkin + Taylor  David Bouma (Facilitator), Mark Foley (optional); Mark Taylor; John Grimston
WSP  Luke Gallagher or David King (by Skype)
Mott MacDonald  Eric Guilleminot or Philippe Cazalis De Fondouce Optional (Skype)
GHD  Would be beneficial to have Richard Frost call in (FH to decide)

Purpose of the Workshop:
FMEA: Definition: “An inductive method of analysis where particular faults or initiating conditions are postulated and the analysis reveals the full range of effects of the fault or the initiating condition on the system” (NZSOLD Dam Safety Guidelines, 2015)

Purpose: 1. To understand the risks associated with potential dam failure to enable appropriate risk management/mitigation measures to be implemented; 2. To fulfill the recommendations in NZSOLD Dam Safety Guidelines for high PIC dams

To identify the failure modes that are credible, and identify which failure modes represent the greatest risk (ie probability x consequence) and document steps to address/mitigate.

This workshop will concentrate on the completed (constructed dam).

This is not an optioneering workshop and is focussed on the proposed design.
Outcomes required:
Identify all potential failure modes and assess which are credible
Identify potential consequences of credible potential failure modes
Categorise/screen the failure modes as:
- Significant risk,
- Low risk (combination of low probability and/or low consequence)
- Unknown – more information required to assess.
Identify and prioritise further information required to understand risks associated with PFM
Initial recommendations regarding risk management or mitigation measures that should be considered.

Preparation Required:
FMEA process document as background
Draft list of failure modes as starting point

Agenda Items:
9:30am Welcome and Introductions - Introduction to FMEA – What it is, why it is needed, how we will do it
9.40am Workshop Objectives and Programme - Confirm purpose of workshop and the outputs we need from the day
9.50am Background on Design, Construction, Operation, Surveillance and Dam Performance (T+T)
10:10am Background on Earlier Identified Potential Failure Modes (T+T)
10:30am Morning tea break
10:50am Identification & Assessment of Potential Failure Modes for Embankment
12pm Identification & Assessment of Potential Failure Modes for Service Spillway
12.30pm Lunch break
1.00pm Identification & Assessment of Potential Failure Modes for Outlet Works
2.00pm Identification of More Likely Potential Failure Modes
3.00pm Afternoon tea break
3.30pm Appropriateness of Current Design and Proposed Surveillance & Monitoring Procedures
4.00pm Summary
4.30pm Close
Waimea Community Dam - ECI
Failure Modes & effects Analysis (FEMA) Workshop

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Date: 19 (preferred) or 20 March 2018

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Taylors Contracting
   TBA
Waimea Water
   Andy Nelson
TDC
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OPUS
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Damwatch
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## Outline/Agenda

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</tr>
<tr>
<td>10:30am</td>
<td>Morning tea</td>
</tr>
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</tr>
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</tr>
<tr>
<td>4.00pm</td>
<td>Summary</td>
</tr>
</tbody>
</table>
What is FMEA

Definition: “An inductive method of analysis where particular faults or initiating conditions are postulated and the analysis reveals the full range of effects of the fault or the initiating condition on the system” (NZSOLD Dam Safety Guidelines, 2015)

Why are we doing it? The purpose

• It is considered good practice to undertake FMEA for all High PIC dams
• For the dam designers, construction team, and owner / operator to understand the risks associated with potential dam failure modes to enable appropriate risk management / mitigation measures to be implemented during design, construction and ongoing operation
• To fulfil the recommendations in NZSOLD Dam Safety Guidelines for high PIC dams
How we will do it – the process

1. Based on knowledge of the dam, identify and list all potential failure modes (PFM)
2. Identify which PFMs are credible
3. For credible PFMs, complete qualitative estimates of the likelihood (probability) and consequences of failure based on the available information and engineering judgement
4. Rank the credible PFMs based on their risk rating (likelihood x consequence)
5. Identify and discuss appropriate mitigation measures to manage the risk to an acceptable level
6. Identify which credible PFMs require further information to justify the risk
7. Provide recommendations where there are opportunities to reduce risk through design, construction controls, or operational controls (e.g., surveillance and control system)
Outputs required from workshop

• Incorporated into design report consisting of:
  • Summary of the process, and including recommendations
  • Results presented in summary tabular format
Failure mode description

Three elements of a potential failure mode description are:

• The Initiator (e.g. Reservoir load, Deterioration/ageing, Operation malfunction, Earthquake)

• The Failure Mechanism (including location and/or path – step by step how the failure progress / develop)

• The Resulting Impact on the Structure (e.g. Rapidity of failure, Breach characteristics)
Example: Potential failure mode sketch and description

- Unedited (insufficient detail): Piping from the embankment into the foundation

- Edited: During a period of high reservoir elevation, piping of the embankment core initiates at the gravel foundation interface in the shallow cutoff trench near Station 2+35 (where problems with the sheet pile and sinkhole occurred). Material might or might not exit at the toe of the dam. Backward erosion occurs until a “pipe” forms through the core exiting upstream below the reservoir level. Rapid erosion enlargement of the pipe occurs until the crest of the dam collapses into the void, and the dam erodes down to the rock foundation.
### Risk matrix

#### Consequences

<table>
<thead>
<tr>
<th>Likelihood:</th>
<th>Insignificant</th>
<th>Minor</th>
<th>Significant</th>
<th>Major</th>
<th>Critical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Almost certain</td>
<td>Low</td>
<td>High</td>
<td>Extreme</td>
<td>Extreme</td>
<td>Extreme</td>
</tr>
<tr>
<td>Likely</td>
<td>Low</td>
<td>Moderate</td>
<td>Very High</td>
<td>Extreme</td>
<td>Extreme</td>
</tr>
<tr>
<td>Possible</td>
<td>Low</td>
<td>Low</td>
<td>Very High</td>
<td>Extreme</td>
<td>Extreme</td>
</tr>
<tr>
<td>Unlikely</td>
<td>Low</td>
<td>Low</td>
<td>High</td>
<td>Very High</td>
<td>Extreme</td>
</tr>
<tr>
<td>Rare</td>
<td>Low</td>
<td>Low</td>
<td>Moderate</td>
<td>High</td>
<td>Very High</td>
</tr>
<tr>
<td>Nil or negligible</td>
<td>Low</td>
<td>Low</td>
<td>Low</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

#### LEVEL OF RISK

<table>
<thead>
<tr>
<th>Level of Risk</th>
<th>Required action for residual risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extreme</td>
<td>Modify design to reduce risk</td>
</tr>
<tr>
<td>Very High</td>
<td>Ensure appropriate defensive design measures, and surveillance / instrumentation are in place to minimise this risk.</td>
</tr>
<tr>
<td>High</td>
<td>Ensure appropriate defensive design measures, and surveillance / instrumentation are in place to minimise this risk.</td>
</tr>
<tr>
<td>Moderate</td>
<td>Acceptable but look for opportunities to reduce and manage the risk.</td>
</tr>
<tr>
<td>Low</td>
<td>Acceptable</td>
</tr>
<tr>
<td>Rank</td>
<td>Probability Assessment</td>
</tr>
<tr>
<td>------</td>
<td>------------------------</td>
</tr>
<tr>
<td>A</td>
<td>80% - 100%</td>
</tr>
</tbody>
</table>
| B    | 33% - 80%              | Probably occur in most circumstances | Organisation/project: More than 1/year  
Industry: Multiple times/year | Likely |
| C    | 10% - 33%              | Should occur at some time | Organisation/project: Once in the last year  
Industry: multiple times/year | Possible |
| D    | 3% - 10%               | Could occur at some time | Organisation/project: Has happened less than once a year  
Industry: More than 1/year | Unlikely |
| E    | 1% - 3%                | May occur only in exceptional circumstances | Organisation: Once or not at all  
Industry: Heard of, less than 1/year | Rare |
| F    | Less than 1%           | Possible but only in exceptional circumstances | Organisation: Never heard of  
Industry: Once or not at all | Nil or negligible |
## Consequence assessment

<table>
<thead>
<tr>
<th>Rank</th>
<th>People Impact</th>
<th>Downstream impact</th>
<th>Consequence Descriptor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Inconvenience or symptom. No injuries requiring any treatment.</td>
<td>Minimal or no impact downstream</td>
<td>Insignificant</td>
</tr>
<tr>
<td>2</td>
<td>First aid treatment, no lost time.</td>
<td>Increased flows downstream but within normal river flood regime. Moderate damage to dam and associated remedial costs</td>
<td>Minor</td>
</tr>
<tr>
<td>3</td>
<td>First aid or medical treatment required, some lost time (up to 2 days).</td>
<td>Moderate flooding and environmental damage, moderate economic losses for dam owner and downstream landowners.</td>
<td>Significant</td>
</tr>
<tr>
<td>4</td>
<td>Injuries resulting in medical treatment, significant lost time (more than 2 days), potential for fatalities</td>
<td>Significant flooding and environmental damage, moderate economic losses for dam owner and downstream landowners.</td>
<td>Major</td>
</tr>
<tr>
<td>5</td>
<td>Multiple fatalities likely.</td>
<td>Major flooding, severe environmental damage, severe economic losses for dam owner and downstream landowners. Significant damage to critical infrastructure.</td>
<td>Critical</td>
</tr>
</tbody>
</table>
## FMEA Summary Spreadsheet format

<table>
<thead>
<tr>
<th>Failure mode ID</th>
<th>Load case</th>
<th>Potential failure mode &amp; cause(s)</th>
<th>Credible [Y/N]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM01</td>
<td>Normal</td>
<td>Major defect in dam facing leading to sufficient flow through dam fill to cause internal erosion of embankment materials leading to dam failure</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Likelihood of failure</th>
<th>Comments on likelihood of failure</th>
<th>Consequence of failure</th>
<th>Comments on consequence of failure</th>
<th>Risk score</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Missing information / uncertainty / further work needed?</th>
<th>Surveillance requirements to monitor for this FM</th>
<th>Instrumentation requirements to monitor for this FM</th>
<th>Recommendation 1</th>
<th>Recommendation 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Background information

- Lee Valley Dam - Detailed Design report Stage 3, T+T July 2014
- Lee Valley Dam Dambreak Analysis and Hazard Assessment, T+T December 2009
- Seismic Hazard Assessment for the Proposed Waimea Dam, GNS, September 2017
- Lee Valley Dam Detailed Design Geotechnical Investigation Report, July 2014
## Waimea Dam characteristics

### Embankment characteristics

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment type</td>
<td>Concrete Face Rockfill Dam (CFRD)</td>
</tr>
<tr>
<td>Embankment volume (approximate)</td>
<td>435,000 m³</td>
</tr>
<tr>
<td>Nominal crest elevation (excluding camber)</td>
<td>201.23 mRL</td>
</tr>
<tr>
<td>Top of parapet wall (excluding camber)</td>
<td>202.83 mRL</td>
</tr>
<tr>
<td>Design Camber</td>
<td>0.3 m</td>
</tr>
<tr>
<td>Maximum dam height (from riverbed to dam crest on CL)</td>
<td>53 m</td>
</tr>
<tr>
<td>Crest length (approximately)</td>
<td>220 m</td>
</tr>
<tr>
<td>Crest width</td>
<td>6 m</td>
</tr>
</tbody>
</table>

### Hydrology, reservoir and flood routing characteristics

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Catchment area</td>
<td>77.5 km²</td>
</tr>
<tr>
<td>Normal top water level (NTWL)</td>
<td>197.2 mRL</td>
</tr>
<tr>
<td>Reservoir storage at NTWL</td>
<td>13,000,000 m³</td>
</tr>
<tr>
<td>Reservoir area at NTWL</td>
<td>630,000 m²</td>
</tr>
<tr>
<td>Maximum design flood level (MDFL)</td>
<td>202.53 mRL</td>
</tr>
<tr>
<td>Reservoir storage at MDFL</td>
<td>16,600,000 m³</td>
</tr>
<tr>
<td>Operational basis flood level (OBFL)</td>
<td>200.48 mRL</td>
</tr>
<tr>
<td>Reservoir storage at OBFL</td>
<td>15,200,000 m³</td>
</tr>
<tr>
<td>Reservoir storage at top of parapet wall (202.83 mRL)</td>
<td>16,800,000 m³</td>
</tr>
</tbody>
</table>
## Waimea Dam characteristics

### Spillway characteristics

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary spillway type</td>
<td>Ogee Weir</td>
</tr>
<tr>
<td>Ogee weir effective length (on arc)</td>
<td>41.89 m</td>
</tr>
<tr>
<td>Peak outflow — Mean Annual Flow (MAF)</td>
<td>179 m³/s</td>
</tr>
<tr>
<td>Peak outflow — Operational Basis Flow (OBF)</td>
<td>472 m³/s</td>
</tr>
<tr>
<td>Peak outflow — Maximum Design Flood (MDF)</td>
<td>1060 m³/s</td>
</tr>
<tr>
<td>Capacity outflow — Reservoir level at top of parapet wall</td>
<td>1152 m³/s</td>
</tr>
</tbody>
</table>

### Spillway and Energy dissipation characteristics

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chute length (plan – ogee crest to start of flip bucket)</td>
<td>124 m</td>
</tr>
<tr>
<td>Chute width, narrow section</td>
<td>20 m</td>
</tr>
<tr>
<td>Chute horizontal transition length</td>
<td>71 m</td>
</tr>
<tr>
<td>Chute vertical curve length</td>
<td>21 m</td>
</tr>
<tr>
<td>Chute minimum height of concrete lining</td>
<td>2.8 m</td>
</tr>
<tr>
<td>Dissipation type</td>
<td>Flip Bucket</td>
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</tbody>
</table>
### Waimea Dam characteristics

<table>
<thead>
<tr>
<th>Feature</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flip bucket radius</td>
<td>20 m</td>
</tr>
<tr>
<td>Bucket lip level</td>
<td>156.6 mRL</td>
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</tbody>
</table>

### Outlet characteristics

<table>
<thead>
<tr>
<th>Feature</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outlet type</td>
<td>Sloping outlet conduits on upstream face with removable screens and valve control.</td>
</tr>
<tr>
<td>Number of outlets</td>
<td>2</td>
</tr>
<tr>
<td>Outlet level – Upper (elevation of top of bellmouth)</td>
<td>181.5 mRL</td>
</tr>
<tr>
<td>Outlet level – Lower (elevation of top of bellmouth)</td>
<td>163.0 mRL</td>
</tr>
<tr>
<td>Control type</td>
<td>Twin 800mm Free Discharge Valves</td>
</tr>
</tbody>
</table>
| Maximum design discharge capacity
  (Valve manufacturer velocity limits applied)         | 15.1 m³/s                                   |
| Concrete conduit size under embankment
  (internal dimensions)                               | Twin 2.5 m Wide x 4.0 m High               |
### Waimea Dam characteristics

<table>
<thead>
<tr>
<th>River tailwater characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tailwater level MAF</td>
<td>150.85 mRL</td>
</tr>
<tr>
<td>Tailwater level OBF</td>
<td>153.46 mRL</td>
</tr>
<tr>
<td>Tailwater level MDF/PMF</td>
<td>156.54 mRL</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Irrigation and environmental flow release¹</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Irrigation release at dam toe (at minimum operating level and from either intake)</td>
<td>2.23 m³/s</td>
</tr>
<tr>
<td>Environmental residual flow (7 day Mean Annual Low Flow (MALF) at minimum operating level and from either intake)</td>
<td>0.51 m³/s</td>
</tr>
<tr>
<td>Environmental flushing flow (at minimum operating level and from either intake)</td>
<td>5.0 m³/s</td>
</tr>
</tbody>
</table>

*Note 1: The criterion design capacity of the outlet is the largest of the requirements of 5.0 cumecs and is not additive (i.e. It is not 2.23 + 5 + 0.51)
Morning tea
Waimea dam general arrangement
Waimea dam concrete face elevation
Embankment Cross Section

LEGEND
- Existing ground/top of Class 3 rock
- Inferred top of Class 2 rock
- Inferred top of Class 1 rock
- Embankment Zone Boundary

Refer to Doc. 17425-GEN-12 for stripping and excavation requirements.

Zone 3B

Refer to Doc. 17425-GEN-12 for stripping and excavation requirements.

Zone 3C

Access camp
Parapet wall
Failure modes assessment table

Microsoft Excel
Worksheet
Minutes

Meeting: Waimea Dam Permanent work Failure Modes and Effects Analysis
Venue: Brightwater
Job No: 27425.100
Date: 19 March 2018
Time: 9:30am
Present: Mark Taylor T+T
David Bouma T+T
John Grimston T+T
Luke Gallagher - WSP (Skype)
Andy Nelson - Waimea Water
Peter Wissel - FHTJV
Richard Frost - GHD (Part only) - Skype

Apologies: Richard Kirby - TDC
Ian Walsh - WSP
Ian Davison - Damwatch

Agenda Item

<table>
<thead>
<tr>
<th>Agenda Item</th>
<th>Owner</th>
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<tbody>
<tr>
<td>1</td>
<td>NA</td>
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</tbody>
</table>

A workshop was held on 19 March 2018 to document Failure Modes and subsequent Effects for the Waimea Dam.

The workshop concentrated only on the permanent works and did not consider the temporary works.

Action Record

<table>
<thead>
<tr>
<th>Action</th>
<th>Responsible</th>
<th>Due Date</th>
<th>Action required</th>
<th>Action taken</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Recommendations from attached Register are to be considered and adopted into the design, instrumentation or OM&amp;S.</td>
<td>T+T</td>
<td>Completion of detailed design</td>
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<tr>
<td>Should issues from the recommendations arise; then these should be documented and justification for non adoption provided.</td>
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6-Apr-18
p:\27425\27425.100\workingmaterial\36 failure modes effect analysis\fmea minutes.docx
<table>
<thead>
<tr>
<th>Failure mode ID</th>
<th>Level of concern</th>
<th>Potential failure scenario if due to flooding leading to insufficient flow through dam 01 to cause erosion of foundation material leading to dam failure.</th>
<th>Critical</th>
<th>Evidence</th>
<th>Consequence of failure</th>
<th>Comments on consequences of failure</th>
<th>Design consideration</th>
<th>Comments on consideration of failure</th>
<th>Recommendations 1</th>
<th>Recommendations 2</th>
<th>Recommendations 3</th>
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</thead>
<tbody>
<tr>
<td>FM02 Normal</td>
<td>Major defect in construction design: result in dam facing being subjected to insufficient flow through dam 01 to cause erosion of foundation material leading to dam failure.</td>
<td>Critical</td>
<td>Evidence</td>
<td>Consequence of failure</td>
<td>Comments on consequences of failure</td>
<td>Design consideration</td>
<td>Comments on consideration of failure</td>
<td>Recommendations 1</td>
<td>Recommendations 2</td>
<td>Recommendations 3</td>
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<td>FM03 Normal</td>
<td>Severe leakage through fault in foundation rock leading to internal erosion of dam embankment interior, leading to dam failure.</td>
<td>Critical</td>
<td>Evidence</td>
<td>Consequence of failure</td>
<td>Comments on consequences of failure</td>
<td>Design consideration</td>
<td>Comments on consideration of failure</td>
<td>Recommendations 1</td>
<td>Recommendations 2</td>
<td>Recommendations 3</td>
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<td>FM04 Normal</td>
<td>Failure of both isolation and flood control (FCD) valves on one of the outlet gate linings leading to uncontrolled release of reservoir water through pipe 01 - no damage to dam.</td>
<td>Low</td>
<td>Evidence</td>
<td>Consequence of failure</td>
<td>Comments on consequences of failure</td>
<td>Design consideration</td>
<td>Comments on consideration of failure</td>
<td>Recommendations 1</td>
<td>Recommendations 2</td>
<td>Recommendations 3</td>
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<td>FM05 Normal</td>
<td>Dam failure due to seepage such as occurrence of seepage to cause sufficient damage to result in uncontrolled release of contents and complete dam failure.</td>
<td>N</td>
<td>Evidence</td>
<td>Consequence of failure</td>
<td>Comments on consequences of failure</td>
<td>Design consideration</td>
<td>Comments on consideration of failure</td>
<td>Recommendations 1</td>
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<td>Recommendations 3</td>
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<td>FM06 Normal</td>
<td>Deformation of downstream face during earthquake that leads to insufficient flow through dam 01 to cause erosion of foundation material leading to dam failure.</td>
<td>N</td>
<td>Evidence</td>
<td>Consequence of failure</td>
<td>Comments on consequences of failure</td>
<td>Design consideration</td>
<td>Comments on consideration of failure</td>
<td>Recommendations 1</td>
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<td>Failure Mode and Effect Analysis</td>
<td>FM11</td>
<td>FM10</td>
<td>Commentary</td>
<td>Recommendation 1</td>
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<tr>
<td>Deformation of embankment face during earthquake that leads to cracking and embankment failure due to liquefaction of soil</td>
<td>Y</td>
<td>N</td>
<td>Critical</td>
<td>Major</td>
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<td>Overtopping of dam from earthquake that leads to failure of wave wall</td>
<td>Y</td>
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<tr>
<td>Displacement of the dam foundation or dam abutment due to major seismic path</td>
<td>Y</td>
<td>N</td>
<td>Critical</td>
<td>Major</td>
<td>Moderate</td>
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<td>Failure of water conduit through displacement and rupture of intake conduit due to major or minor earthquake loading</td>
<td>Y</td>
<td>N</td>
<td>Critical</td>
<td>Major</td>
<td>Moderate</td>
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<tr>
<td>Overtopping of the dam entrance due to flood wave generated waves - cause no failure of spillway but may cause erosion of downstream shoulders</td>
<td>Y</td>
<td>N</td>
<td>Critical</td>
<td>Major</td>
<td>Moderate</td>
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<tr>
<td>Overturning of the dam entrance due to flood wave generated waves - cause no failure of spillway but may cause erosion of downstream shoulders</td>
<td>Y</td>
<td>N</td>
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<tr>
<td>Failure mode</td>
<td>Load case</td>
<td>Potential failure mode &amp; cause(s)</td>
<td>Credible frequency of failure</td>
<td>Comments on consequence of failure</td>
<td>Indicative</td>
<td>Recommendations 1</td>
<td>Recommendation 2</td>
<td>Recommendation 3</td>
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<tr>
<td><strong>FM24</strong></td>
<td>Road</td>
<td>Overtopping of the dam - embankment due to</td>
<td>Y</td>
<td>Initially, no immediate risk.</td>
<td>Minor</td>
<td>Further review for acceptable factor of safety.</td>
<td>Instrumentation requirements in manual for this FM</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>flood in studied reservoir due to failure of the rock overlying the dam</td>
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<td></td>
<td></td>
<td>Foundation failure due to instability of the reservoir bedrock</td>
<td>N</td>
<td>Requires multiple low probability events to happen simultaneously (siltation, availability of flood water).</td>
<td>Significant</td>
<td>Check if underwater pipes can be monitored</td>
<td>Instrumentation requirements in manual for this FM</td>
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<td></td>
<td>Structural failure of the downstream pressure pipe due to undermining or settlement</td>
<td>Y</td>
<td>May lead to loss of life, but expensive repairs</td>
<td>Major</td>
<td>Further assessment for potential instability of downstream face</td>
<td>Instrumentation requirements in manual for this FM</td>
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<tr>
<td>Failure Mode</td>
<td>Load Case</td>
<td>Potential Failure mode &amp; cause(s)</td>
<td>Credible Likelihood of failure</td>
<td>Consequence of Failure</td>
<td>Comments on consequence of failure</td>
<td>Risk score</td>
<td>Surveillance requirements to monitor for this failure mode</td>
<td>Instrumentation requirements to monitor for this failure mode</td>
<td>Recommendations 1</td>
<td>Recommendations 2</td>
<td>Recommendations 3</td>
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<td>FM25 Flood</td>
<td></td>
<td>Flooding of conduit causing power to fail to actuator. Resulting in flow out of outlet pipes.</td>
<td>Y rare</td>
<td>Minor Loss of water from reservoir</td>
<td>Low</td>
<td>Check commissioning. Annual inspections and comprehensive inspections. Stroking valves as per recommendations.</td>
<td>Level indicator in conduit.</td>
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<td>FM26 Earthquake</td>
<td></td>
<td>Cracking at perimetric joint during an earthquake causing a concentrated leak through the dam face. This is a subset of FM01.</td>
<td>Y Rare</td>
<td>Minor Loss of water from reservoir and potential erosion of dam embankment</td>
<td>Low</td>
<td>Check ICOLD and literature review as to whether this has caused catastrophic damage. Not just for this failure mode</td>
<td>toe seepage drains flow monitoring system (electronic and telemetered).</td>
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Appendix E: Safety in Design

- Safety in Design Risk Register
Waimea Community Dam - ECI
Safety in Design workshop for permanent works only

**AGENDA**

<table>
<thead>
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<th>Attendees:</th>
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<tr>
<td>Fulton Hogan</td>
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<td>Taylors Contracting</td>
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<tr>
<td>Waimea Water</td>
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<tr>
<td>Tonkin &amp; Taylor</td>
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<tr>
<td>WSP</td>
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**Purpose of the Workshop:**
Go through main dam components and consider different stages in respect of safety:

**Scope**
Focus on SiD in respect of operational and maintenance of the dam
Less emphasis on temporary works (covered elsewhere); and

- Geotechnical aspects already covered in Geotechnical Risks workshop; and
- Diversion workshop
- Purpose and background – legal requirements **10 mins (Mark Taylor (T+T))**
- Talk through M&E design to **date 3 on 4 slides with key operational aspects** – focus on maintenance and operations (**20 mins Luke Gallagher (WSP) by Skype**) Then come back M&E items by item (1 hour)

a. Removal and maintenance of screens
b. Winch arrangements for screen maintenance and inclined intake pipework installation/removal
c. Installation and maintenance of the isolation valves
d. Inspect pipes for leaks
e. Long term corrosion/coating system
f. FCV’s – installation and maintenance, and outlining hazards
g. Conduit access – lighting and ventilation, and crawl space between conduits
h. Power supply aspects –
i. Diesel genset and fuel supply and storage
ii. Transformer – protection (if transmission line is available)
iii. Battery storage

- Instrumentation inspection/monitoring, calibration (Dominic Fletcher T+T)

Civil Design for Maintenance (Dominic Fletcher T+T – to describe general maintenance requirements for other aspects of dam 20 mins)

- Hazard/risk register development (Dominic 1.5 hr)
- Anchoring points
- Clearance of debris (intake screens)
- Spillway clearing
- Dam decommissioning
- Plunge pool monitoring and clearance
- Bridges – maintenance/corrosion protection
  - Jacking points for bridge bearing replacement

- Major of replacement of any items.

Other items or meeting float 0.5 hours

Outputs

- SiD Risk register developing risk and mitigations for the permanent works.

Boundaries/Constraints – assume current arrangements i.e. not a redesign, so the workshop will only focus on completion of design and any feedback that can be incorporated now.

Outcomes required:
An understanding of safety in design aspects for project permanent works components; a list of items and potential mitigation or follow up actions.

Preparation Required:
Draft ppt and spreadsheet have been prepared.

Draft ppt attached.

Agenda Items:

1. Introduction – Safety in Design & workshop objectives and target outcomes
2. Review initial list of components and risks and add any that are missing
3. Progress through each dam component and discuss for each key risks, potential mitigation and risk owner(s). Some risks might straddle structures. Document discussion.
4. Summary session – wrap up of key risks and mitigation and general final comments/discussion.
### SAFETY IN DESIGN RISK REGISTER

<table>
<thead>
<tr>
<th>ID</th>
<th>Project Lifecycle</th>
<th>Action/Work/Event</th>
<th>Hazard</th>
<th>Uncontrolled Harm/Consequence</th>
<th>Existing Controls</th>
<th>Risk Assessment</th>
<th>Mitigation options</th>
<th>Residual Risk</th>
<th>Considerations</th>
<th>Go?</th>
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<tbody>
<tr>
<td>1</td>
<td>Design</td>
<td>Spillway location</td>
<td>High cut (30 m) slope Instability</td>
<td>Rockfall, wedge movement, toppling blocks fall from rock face causing serious harm/death to people below</td>
<td>Design slope around rock mass properties and features, stabilise slopes progressively, construction controls.</td>
<td>E L Risk</td>
<td>Move spillway or change type</td>
<td>E L Risk</td>
<td>Different spillways have their own high risks. Location change may not reduce likelihood</td>
<td>No</td>
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<td>2</td>
<td>Design</td>
<td>Spillway type</td>
<td>Non gated spillway with no method of controlling excess persons working on chute and/or downstream to uncontrolled flows</td>
<td>People being swept away/drowned</td>
<td>Do not provide uncontrolled access points to spillway or downstream. Signage, Operator's procedures.</td>
<td>F Mod</td>
<td>Add mechanism for providing temporary upstream control</td>
<td>F Mod</td>
<td>Severe level control</td>
<td>No</td>
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<td>3</td>
<td>Design</td>
<td>Flip bucket location</td>
<td>Difficult to access</td>
<td>Slips, trips, falls to operator</td>
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<td>E</td>
<td>Provide specific safe access</td>
<td>E</td>
<td>Yes</td>
<td>Yes</td>
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<td>4</td>
<td>Construction</td>
<td>Spillway construction</td>
<td>Slope instability</td>
<td>Rockfall resulting in harm to construction workers</td>
<td>Slope stabilisation measures, better design.</td>
<td>F</td>
<td>Map excavation faces progressively and apply recommended protection</td>
<td>F</td>
<td>Severe</td>
<td>Yes</td>
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<td>5</td>
<td>Operation</td>
<td>Spillway operation</td>
<td>Discharge flows in excess of the spillway design capacity</td>
<td>Possible cut, damage to spillway chute, or 2</td>
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<td>E</td>
<td>No further mitigation</td>
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<td>6</td>
<td>Operation</td>
<td>Surveillance of plunge pool</td>
<td>Erosion to the plunge pool occurs over time requiring ongoing dive inspections and potentially remedial works</td>
<td>Harm to persons having to access the plunge pool to undertake routine inspections and remedial works</td>
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<td>F Mod</td>
<td>Consider using the plunge pool and/or pre-excavations during design to reduce monitoring frequency</td>
<td>F Mod</td>
<td>No</td>
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<td>7</td>
<td>Operation</td>
<td>Surveillance of spillway</td>
<td>Falling from heights</td>
<td>Operator staff falling onto spillway and being seriously harmed</td>
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<td>F</td>
<td>Ensure barrier to prevent access</td>
<td>F</td>
<td>T+T</td>
<td>Yes</td>
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<td>8</td>
<td>Operation</td>
<td>Spillway repair</td>
<td>Exposure of workers to flood risk during provisional repairs to spillway liner</td>
<td>People being swept away/drowned</td>
<td>Design spillway liner to reduce potential for spilling, chipping, abrasion resistance, strength, high strength, cabling.</td>
<td>F Mod</td>
<td>Change spillway type</td>
<td>F Mod</td>
<td>Cost of alternatives</td>
<td>No</td>
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<td>9</td>
<td>Decommissioning</td>
<td>Spillway decommissioning</td>
<td>Removal of spillway concrete</td>
<td>Construction work resulting in harm to workers</td>
<td>Demolition SMHMS</td>
<td>E</td>
<td>No further mitigation</td>
<td>E</td>
<td>No</td>
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<td>10</td>
<td>Design</td>
<td>Bridge bearing replacement</td>
<td>Jigging/lifting of bridge deck to replace bearings</td>
<td>Crushing, falls from heights, suspended heavy loads</td>
<td>Replacement of jacking points to enable bearing replacement</td>
<td>F Mod</td>
<td>No</td>
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<td>11</td>
<td>Operation</td>
<td>Common protection/irrigation</td>
<td>Falling from heights while applying protection</td>
<td>Construction workers falling from heights</td>
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<td>F Mod</td>
<td>No</td>
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<td>12</td>
<td>Design</td>
<td>Abutment stability</td>
<td>Possible stability problems on the left abutment</td>
<td>Rockfall resulting in harm to construction workers</td>
<td>Design excavation profile and protection measures based on site investigations</td>
<td>E</td>
<td>No</td>
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<td>13</td>
<td>Construction</td>
<td>Excavation</td>
<td>Stability movement (sloping surfaces)</td>
<td>Rock wedges resulting in harm to construction workers</td>
<td>Map excavated surfaces and confirm slope stabilisation/ protection measures</td>
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No further mitigation
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<tr>
<th>Design Stage</th>
<th>Operative Description</th>
<th>Risk Description</th>
<th>Management</th>
<th>Mitigation</th>
<th>Commentary</th>
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<tr>
<td>Design</td>
<td>Embankment</td>
<td>Damage to slope instability due to erosion and landslides</td>
<td>High</td>
<td>No further mitigation</td>
<td>Cost of uphill rock to reduce risk is not practical</td>
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<tr>
<td>Design</td>
<td>Intake Works</td>
<td>Damage to intake works due to erosion and landslides</td>
<td>High</td>
<td>No further mitigation</td>
<td>Cost of uphill rock to reduce risk is not practical</td>
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<tr>
<td>Design</td>
<td>Concrete Face</td>
<td>Damage to concrete face due to erosion and landslides</td>
<td>High</td>
<td>No further mitigation</td>
<td>Cost of uphill rock to reduce risk is not practical</td>
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</tbody>
</table>

### Design Stage: Detailed Design

#### Design Intake screens

**Operative Description:**

- Intake screens are used to control the flow of water into the dam.

**Risk Description:**

- Intake screen cleaning manual cleaning involves diver hazards
- Intake screen maintenance manual cleaning involves diver hazards
- Intake screen removal and maintenance remote work involves diver hazards
- Intake screen testing and inspection involves diver hazards

**Management:**

- Design with guide plates to facilitate installation

**Mitigation:**

- Use of compressed air for non diver cleaning

**Commentary:**

- Intake screen maintenance no risk.

---

<table>
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<tr>
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<td>30</td>
<td>Design Remixed pipework on concrete face</td>
<td>Submerged in reservoir requiring diver access for inspection and maintenance</td>
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**Note:** The risk register includes various hazards, their descriptions, and associated risks. Each entry details the severity, probability, and consequence, which together determine the risk score. The risk management plan outlines strategies to mitigate these risks and improve safety measures.
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<th>Project Name</th>
<th>Design Stage</th>
<th>Date</th>
<th>Risk Category</th>
<th>Risk Description</th>
<th>Mitigation Actions</th>
<th>Mitigation Ownership</th>
<th>Level</th>
<th>Action</th>
<th>Event</th>
<th>Cost</th>
<th>Schedule</th>
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<tr>
<td>57</td>
<td>Construction</td>
<td>Forestry debris</td>
<td>Management of the deforestation during construction and the impact this debris may have on the dam during filling</td>
<td>S E VH</td>
<td>Additional debris capture measures such as catch dams and debris booms upstream of the site for construction</td>
<td>S E VH</td>
<td>Contractor</td>
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<td>58</td>
<td>Construction</td>
<td>Fire Hazard</td>
<td>Forestry areas catch on fire trapping people on site</td>
<td>S F Mod</td>
<td>No further mitigation</td>
<td>S F Mod</td>
<td>Waimea Water</td>
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<td>59</td>
<td>Construction</td>
<td>Event of weather</td>
<td>High rainfall, ice causing access unsafe</td>
<td>S E VH</td>
<td>No further mitigation</td>
<td>S E VH</td>
<td>Waimea Water</td>
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<td>Construction</td>
<td>Control of water (river flows and floods)</td>
<td>Foundation of work site leading to overtopping of the works and dam breach flood sourced</td>
<td>S F Mod</td>
<td>No further mitigation</td>
<td>S F Mod</td>
<td>Contractor</td>
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<td>Operation</td>
<td>Forestry debris</td>
<td>Poor forestry operations resulting in large amounts of debris entering the reservoir and affecting intakes requiring cleaning/maintenance</td>
<td>S E VH</td>
<td>Change design to no embankment forms (i.e., tower)</td>
<td>S E VH</td>
<td>Waimea Water</td>
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<td>62</td>
<td>Operation</td>
<td>Debris management</td>
<td>Debris floating spillway open causing blockage and reduced capacity during flood passage</td>
<td>S F Mod</td>
<td>No further mitigation</td>
<td>S F Mod</td>
<td>Waimea Water</td>
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<td>63</td>
<td>Operation</td>
<td>Embedded items</td>
<td>Working around in water</td>
<td>S E VH</td>
<td>Change design to no embankment forms (i.e., tower)</td>
<td>S E VH</td>
<td>Waimea Water</td>
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<tr>
<td>64</td>
<td>Operation</td>
<td>Operator error</td>
<td>Opening closing valves inappropriately resulting in unintended release of water downstream and flood/loss of control</td>
<td>S F Mod</td>
<td>Multiple commands required to open valves, discharge alarms downstream</td>
<td>S F Mod</td>
<td>Waimea Water</td>
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<tr>
<td>65</td>
<td>Operation</td>
<td>Security</td>
<td>Unauthorised persons accessing the site and being harmed and/or harming others/equipment that results in harm to others</td>
<td>S E VH</td>
<td>Install security cameras</td>
<td>S E VH</td>
<td>Waimea Water</td>
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<td>Operation</td>
<td>Lightning strike</td>
<td>Lightning electrocution resulting in burns or deaths</td>
<td>None</td>
<td>Site is at the end of a private road with multiple locked gates, no further mitigation</td>
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<td>Waimea Water</td>
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<td>Operation</td>
<td>Landslide wave</td>
<td>Landslide wave impacting dam and reservoir areas</td>
<td>S F Mod</td>
<td>No further mitigation</td>
<td>S F Mod</td>
<td>Waimea Water</td>
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<tr>
<td>68</td>
<td>Operation</td>
<td>Maintenance of fish pass channel</td>
<td>Slips, trips and falls due to steep face</td>
<td>S D VH</td>
<td>Include access points on all dam access points</td>
<td>S D VH</td>
<td>Waimea Water</td>
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<td>Operation</td>
<td>Extreme flood event</td>
<td>Dam failure</td>
<td>S F Med</td>
<td>Warning system for downstream inhabitants</td>
<td>S F Med</td>
<td>Waimea Water</td>
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<tr>
<td>70</td>
<td>Operation</td>
<td>Extreme earthquake event</td>
<td>Dam failure</td>
<td>S F Med</td>
<td>Warning system for downstream inhabitants</td>
<td>S F Med</td>
<td>Waimea Water</td>
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<tr>
<td>71</td>
<td>Operation</td>
<td>Post event access to the site</td>
<td>During PMF the flood passage will direct flows across the access road. This will have a significant impact on access to the dam because of the erosion during this process</td>
<td>S F Med</td>
<td>Access to site after the rescue</td>
<td>S F Med</td>
<td>Waimea Water</td>
<td></td>
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<td>72</td>
<td>Operation</td>
<td>Public access to site</td>
<td>Public access to site after the emergency</td>
<td>S E VH</td>
<td>Install secured fence at access road to dam</td>
<td>S E VH</td>
<td>Waimea Water</td>
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<td></td>
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<td>73</td>
<td>Operation</td>
<td>Operator rescue from reservoir dam crest</td>
<td>Operator falls into reservoir</td>
<td>S F Med</td>
<td>H civic duties either side of lake. Life buoys and two operators</td>
<td>S F Med</td>
<td>Waimea Water</td>
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<td>A</td>
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<td>Ex</td>
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<tr>
<td>Likely</td>
<td>B</td>
<td>Mod</td>
<td>VH</td>
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<td>C</td>
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<td>Low</td>
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<td>Rare</td>
<td>D</td>
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<tr>
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### Consequence Assessment

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<th>Rank</th>
<th>People Impact</th>
<th>Consequence Descriptor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Inconvenience or symptom. No injuries requiring any treatment.</td>
<td>Insignificant</td>
</tr>
<tr>
<td>2</td>
<td>First aid treatment, no lost time.</td>
<td>Minor</td>
</tr>
<tr>
<td>3</td>
<td>First aid or medical treatment required, some lost time (up to 2 days).</td>
<td>Significant</td>
</tr>
<tr>
<td>4</td>
<td>Injuries resulting in medical treatment, significant lost time (more than 2 days), some permanent disability.</td>
<td>Major</td>
</tr>
<tr>
<td>5</td>
<td>Single or multiple fatalities or serious injury and serious permanent disability.</td>
<td>Critical</td>
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</table>

### Likelihood Assessment

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<th>Rank</th>
<th>Probability of occurrence during the project</th>
<th>The likelihood of the event occurring during the project</th>
<th>Experience Assessment</th>
<th>Likelihood Descriptor</th>
</tr>
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<tr>
<td>A</td>
<td>80% - 100% Expected to occur in most circumstances</td>
<td>Organisation/project: Several times in the last 12 months</td>
<td>Organisation/project: More than 1/year</td>
<td>Almost certain</td>
</tr>
<tr>
<td>B</td>
<td>33% - 80% Probably occur in most circumstances</td>
<td>Organisation/project: More than 1/year</td>
<td>Industry: Multiple times/year</td>
<td>Likely</td>
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<tr>
<td>C</td>
<td>10% - 33% Should occur at some time</td>
<td>Organisation/project: Once in the last year</td>
<td>Industry: multiple times/year</td>
<td>Possible</td>
</tr>
<tr>
<td>D</td>
<td>3% - 10% Could occur at some time</td>
<td>Organisation/project: Has happened less than once a year</td>
<td>Industry: More than 1/year</td>
<td>Unlikely</td>
</tr>
<tr>
<td>E</td>
<td>1% - 3% May occur only in exceptional circumstances</td>
<td>Organisation: Once or not at all</td>
<td>Industry: Heard of, less than 1/year</td>
<td>Rare</td>
</tr>
<tr>
<td>F</td>
<td>Less than 1% Possible but only in exceptional circumstances</td>
<td>Organisation: Never heard of</td>
<td>Industry: Once or not at all</td>
<td>Nil or negligible</td>
</tr>
</tbody>
</table>
Appendix F: Seismic hazard assessment

DISCLAIMER

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The data presented in this Report are available to GNS Science for other use from February 2011.

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EXECUTIVE SUMMARY

Spectra are presented to address the NZSOLD New Zealand Dam Safety Guidelines for Maximum Design Earthquake (MDE) motions and Operating Basis Earthquake (OBE) motions for high Potential Impact Category structures at the proposed Lee Valley Dam site at location 41.47° S, 173.16° E. Smoothed horizontal acceleration response spectra (5% damping) are provided for the site for NZS1170 ground conditions Weak Rock, Shallow Soil and Deep/Soft Soil (Tables ES-1 to ES-3 and Figures ES-1 to ES-3). The spectra are provided in equation form in Section 3.2. The spectra were calculated using the May 2010 update of GNS’s National Seismic Hazard Model (NSHM). Results are presented for return periods of 150, 500, 2500, 5000 and 10,000 years for periods up to 3 seconds.

An earlier version of the NSHM was used to derive the hazard spectra presented in the New Zealand Standard NZS1170.5:2004 Structural Design Actions. The smoothing procedures used in NZS1170.5 are generally conservative, in that they produce near upper-bound envelopes to the spectral shapes for all locations in New Zealand for periods beyond the plateau at the peak of the smoothed spectrum. In addition to updated fault and background seismicity models, the spectra presented for the proposed Lee Valley site use alternative smoothing procedures to more closely approximate the spectra derived directly for the location from the hazard studies. The hazard-derived Z value is 0.29, very similar to the NZS1170 value for Lee Valley of 0.30.

There are four active fault sources included in the 2010 NSHM that lie within 50 km of the proposed Lee Valley Dam. Details of these faults are listed in Section 3.3. The closest is the Waimea Fault (represented as two sources in the NSHM) located approximately 8 km north-west of the site with an estimated recurrence interval of about 10,000 years. To the south of the site there are two faults that are appreciably more active; the Wairau Fault and the Alpine Fault, located respectively about 20 and 40 km from the site. Both of these faults are considered capable of producing high magnitude 7 earthquakes with average recurrence intervals of a few thousand (Wairau Fault) to a few hundred years (Alpine Fault). With reference to GNS Science’s active fault database, there are no active faults mapped in the immediate vicinity of the proposed site. A review of aerial photographs also did not reveal any topographic evidence for the existence of active fault traces in the immediate vicinity of the proposed site. It appears, from available data, that the site is free of active fault displacement hazard. However, no site investigations have been undertaken to further substantiate this.

The smoothed 150-year motions listed in Tables ES-1 to ES-3 are recommended as the Operating Basis Earthquake (OBE) motions for the three site classes, consistent with the NZSOLD Dam Safety Guidelines.

The NZSOLD Guidelines allow adoption of a probabilistically-based 10,000-year spectrum or scenario spectra for the estimated motions from rupture of nearby faults to represent the Maximum Design Earthquake (MDE) motions. Consideration of deterministic spectra for fault-rupture scenarios suggest that the envelope of the 84-percentile spectra for a magnitude 7.0 earthquake on the Waimea South Fault at 8 km distance and a magnitude 7.8 earthquake on the Wairau Fault at 21 km distance are sufficient to represent the MDE motions, in lieu of the purely probabilistically-based 10,000-year spectra. The envelope of these deterministic spectra can be conveniently approximated by the smoothed 5000-year spectra of Tables ES-1 to ES-3.
### Table ES-1
Recommended smoothed horizontal acceleration spectra, weak rock.

<table>
<thead>
<tr>
<th>Period (T(s))</th>
<th>150yrs (pga)</th>
<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
<th>10,000yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (pga)</td>
<td>0.16</td>
<td>0.24</td>
<td>0.40</td>
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<td>0.15</td>
<td>0.41</td>
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</tr>
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<td>0.41</td>
<td>0.67</td>
<td>1.30</td>
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### Table ES-2
Recommended smoothed horizontal acceleration spectra, shallow soil.

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<th>Period (T(s))</th>
<th>150yrs (pga)</th>
<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
<th>10,000yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (pga)</td>
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<tr>
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<td>0.40</td>
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</table>
Table ES-3  Recommended smoothed horizontal acceleration spectra, Deep/Soft soil.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>150yrs</th>
<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
<th>10,000yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (pga)</td>
<td>0.18</td>
<td>0.27</td>
<td>0.41</td>
<td>0.48</td>
<td>0.56</td>
</tr>
<tr>
<td>0.075</td>
<td>0.31</td>
<td>0.50</td>
<td>0.74</td>
<td>0.88</td>
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<tr>
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<td>0.58</td>
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<tr>
<td>0.15</td>
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<td>0.31</td>
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</table>

Figure ES-1  Recommended 5% damped horizontal response spectra for Weak Rock. The peak ground acceleration (0s) values are plotted at 0.03s.
**Figure ES-2**  Recommended 5% damped horizontal response spectra for Shallow Soil. The peak ground acceleration (0s) values are plotted at 0.03s.

**Figure ES-3**  Recommended 5% damped horizontal response spectra for Deep/Soft soil. The peak ground acceleration (0s) values are plotted at 0.03s.
1.0 INTRODUCTION

1.1 Project Brief

The technical brief required GNS Science to calculate the elastic acceleration response spectra to satisfy the criteria of the NZSOLD New Zealand Dam Safety Guidelines (2000) for Maximum Design Earthquake (MDE) motions and Operating Basis Earthquake (OBE) motions for High Potential Impact Category Structures. The resulting 5% damped, horizontal spectra for the proposed site at location 41.47° S, 173.16° E are for return periods of 150, 500, 2500 and 10,000 years and NZS1170 sites classes corresponding to Weak Rock, Shallow Soil and Deep/Soft Soil. The spectra are provided for the set of periods from 0s up to 3s and are smoothed appropriately for their use as design spectra. A discussion of nearby active faults and their characteristics was also required. Spectra are also presented at the 50- and 84-percentile levels for several rupture scenarios of nearby faults, as potential deterministic candidates for the MDE spectra.

2.0 NZSOLD GUIDELINES FOR OBE AND MDE MOTIONS

Smoothed 5% damped acceleration response spectra are presented in this report for various return periods and for various fault-rupture scenarios to determine spectra that satisfy the criteria of the NZSOLD (2000) New Zealand Dam Safety Guidelines for Operating Basis Earthquake (OBE) motions and Maximum Design Earthquake (MDE) motions for High Potential Impact Category (PIC) structures. The performance requirement for OBE motions is either no damage, or minor repairable damage. In MDE motions, some damage is acceptable, but it must not result in catastrophic failure, and it is required that at least the impounding capacity of the dam be maintained.

The NZSOLD Guidelines specify that the return period for OBE Motions is 150 years. The MDE spectra are determined by considering both probabilistic spectra and scenario spectra for the estimated 50- and 84-percentile motions from rupture of nearby faults. The NZSOLD Guidelines specify the return period to be considered for MDE motions for High PIC dams as “a 1 in 10,000 AEP event if probabilistically derived” (AEP is Annual Exceedance Probability). The MDE may also be Maximum Credible Earthquake, described as the “largest reasonably conceivable earthquake that appears possible along a recognised fault or within a geographically defined tectonic province, under the presently known or interpreted tectonic framework”. According to the NZSOLD Guidelines, the probabilistic (i.e. return periods) and scenario percentile-level criteria are alternatives, with it not being necessary to satisfy both criteria.

3.0 THE 2010 NATIONAL SEISMIC HAZARD MODEL

The hazard calculations performed in this study used the May 2010 update of the NSHM fault model with the June 2006 model for distributed seismicity. The 2010 version has been significantly changed from the 2000 NSHM of Stirling et al. (2000, 2002), which was used to develop the hazard section of the New Zealand Standard NZS1170.5 for earthquake loads in New Zealand (Standards New Zealand, 2004).
The changes from the 2000 to 2010 NSHM affected both the grid of point sources, with parameters derived from the historical seismicity catalogue since 1840, and the fault sources, with parameters based largely on geological information. The updates from the 2000 to the 2010 NSHM are discussed below.

3.1 Distributed Seismicity Sources

3.1.1 Modifications to the Modelling from the 2000 NSHM

Both the input data and methodology for characterizing the distributed seismicity sources have been significantly updated since 2000. The same overall approach is used, with the b-value of the Gutenberg-Richter distribution \( \log N = a - bM \) (\( N \)= number of events \( \geq \) magnitude \( M \)) calculated for each seismotectonic region, and the a-value calculated at each grid point, with these values then smoothed using a Gaussian weighting function with distance. There are fewer seismotectonic zones than in the 2000 model, and the NSHM now uses seismicity data past the previous 1997 cut-off up to the end of 2005. In calculating the a- and b-values, events are now assigned to the depth layer corresponding to their catalogue depth, while in the 2000 model events with constrained depths up to 33 km were randomly distributed between 0 and 33 km depth. The final a-value for each grid cell remains a maximum-likelihood estimate based on the various sub-catalogues identified in the New Zealand earthquake catalogue, a sub-catalogue being a space-time subset of the catalogue with a complete record above a specific magnitude threshold.

3.2 Fault Sources

The second component of the seismicity model in the NSHM represents the fault sources. In the main, the fault sources model earthquakes that are associated with geologically-identified surface traces. The NSHM fault sources consist of planar segments, having perhaps several end-to-end planar surfaces for each source to model changes in strike or dip along a fault. This approach inevitably results in a simplified representation of the fault sources compared with the identified traces. Because of this, the localised differences between the NSHM fault sources and observed traces can affect the site-to-fault distances by approximately 1-2 km in some cases. Each of these sources is assigned a characteristic magnitude and average recurrence interval, and is modelled as producing earthquakes of only its characteristic magnitude. Some long faults, such as the Alpine and Wellington Faults, are separated into several independent segments, each with its own characteristic magnitude and average recurrence interval.

The 2000 NSHM used a hierarchy of methods to assign magnitudes and average recurrence intervals. The 2007 and subsequent versions of the NSHM used a single method to estimate the characteristic magnitude and recurrence interval for each fault source. Newly developed regression equations of moment magnitude \( M_w \) on fault area were used for New Zealand earthquakes (Villamor et al. 2001; Berryman et al. 2002), and an internationally-based regression for plate boundary strike-slip faults (Hanks & Bakun 2002) for the Alpine Fault.
### 3.3 Faults affecting the Lee Valley site

There are four active fault sources included in the 2010 NSHM that lie within 50 km of the proposed Lee Valley Dam (Figure 1). Details of these faults are listed in Table 1. The closest is the Waimea Fault (represented as two sources in the NSHM) located approximately 8 km north-west of the site with an estimated recurrence interval of about 10,000 years. To the south of the site, there are two faults that are appreciably more active; the Wairau Fault and the Alpine Fault, located respectively about 20 and 40 km from the site. Both of these faults are considered capable of producing high magnitude 7 earthquakes at average recurrence intervals of a few thousand (Wairau Fault) to a few hundred years (Alpine Fault).

More detailed information on the fault systems and issues relating to avoidance of fault displacement and deformations are discussed in Section 5.0.

Table 1: Active faults in the vicinity of the Lee Valley site.

<table>
<thead>
<tr>
<th>Name</th>
<th>Distance to site (km)</th>
<th>Magnitude</th>
<th>Recurrence Interval (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waimea North</td>
<td>8</td>
<td>7.4</td>
<td>9600</td>
</tr>
<tr>
<td>Waimea South</td>
<td>12</td>
<td>7.0</td>
<td>5600</td>
</tr>
<tr>
<td>Wairau</td>
<td>21</td>
<td>7.8</td>
<td>2500</td>
</tr>
<tr>
<td>Alpine</td>
<td>43</td>
<td>7.7</td>
<td>620</td>
</tr>
</tbody>
</table>
Figure 1  Faults in the vicinity of the Lee Valley site. The proposed dam location is shown by the yellow star. Fault parameters are listed in Table 1.
3.4 The attenuation model

The attenuation model used is the New Zealand model of McVerry et al. (2000, 2006) for 5% damped acceleration response spectra. This model was used in the hazard studies defining the New Zealand seismic hazard maps and spectral shapes in the New Zealand Standard NZS1170.5. The McVerry et al. attenuation model accounts for the three different tectonic regimes which apply in New Zealand (i.e. crustal, subduction interface, and intraslab earthquakes in the dipping slab). Crustal earthquakes provide the main contributions to the hazard for the region around the Lee Valley site. The attenuation relationships for crustal earthquakes have further subdivisions, through mechanism terms, for different types of fault rupture (strike-slip, normal, oblique/reverse and reverse). They also cater for several site conditions that are defined in terms of Classes A/B, C and D of NZS1170:2004

The attenuation expressions were developed mainly from New Zealand strong-motion earthquake records, supplemented by data from elsewhere to obtain near-source constraint. This was achieved through introducing additional records at distances of less than 10 km, a distance range for which there were no New Zealand data. The crustal model was modified from the Abrahamson & Silva (1997) model which was derived from mainly western US data, while the subduction zone expression was modified from the Youngs et al. (1997) expression derived from subduction zone earthquakes around the world.

4.0 HAZARD ESTIMATES

4.1 Unsmoothed spectra

Elastic acceleration response spectra for 5% of critical damping with magnitude-weighting have been estimated for five return periods, 150, 500, 2500, 5000 and 10,000 years, corresponding to the client’s specifications. The unsmoothed spectra as produced by the 2010 NSHM fault model combined with the June 2006 distributed-seismicity model for the NZS1170.5 Class B Weak Rock, Class C Shallow Soil and Class D Deep/Soft Soil Site conditions are shown in Figure 2, Figure 3 and Figure 4, with the pga value plotted at a period of 0.03s. The spectral values are listed in Table 2, Table 3 and Table 4.

The hazard studies conducted for the development of the NZS1170.5 spectra used magnitude-weighting of the spectra for periods up to 0.5s. The magnitude-weighting method scales the expected accelerations for any event according to earthquake magnitude M, by a factor \( (M/7.5)^{1.285} \) (Idriss, 1985), while the unweighted estimates have no scaling of the expected accelerations. Full magnitude-weighting has been used for periods up to and including 0.5s, tapering to no magnitude-weighting at 0.75s.

Magnitude-weighting addresses a criticism of uniform-hazard spectra that they tend to be dominated by contributions from moderate-magnitude earthquakes, and do not reflect the effect of duration in causing structural damage. The magnitude-weighting method scales the expected spectra for any event according to earthquake magnitude, to reflect duration effects which affect the damage potential of motions for a given peak response. The magnitude-weighting factor is intended to produce estimates that are equivalent to magnitude 7.5 values in terms of damage-potential. As a result, at short spectral periods magnitude-weighted spectral accelerations are usually less than those from uniform hazard analysis. For
example, the magnitude-weighting factor for magnitude 6 is 0.75. For spectral periods longer than 0.5s, small-to-moderate magnitude earthquakes produce significantly weaker motions than larger magnitude events, making scaling unnecessary.

Table 2  The unsmoothed magnitude-weighted horizontal spectra for Weak Rock for the Lee Valley Dam site.

<table>
<thead>
<tr>
<th>Period (T(s))</th>
<th>150yrs</th>
<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
<th>10,000yrs</th>
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<tbody>
<tr>
<td>0 (pga)</td>
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<tr>
<td>0.075</td>
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<tr>
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<tr>
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<td>0.71</td>
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Note1: See Table 8 for recommended smoothed spectra
Table 3  

The unsmoothed magnitude-weighted horizontal spectra for Shallow Soil for the Lee Valley Dam site.

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<th>Period (T(s))</th>
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</table>

Note1: See Table 9 for recommended smoothed spectra
Table 4  The unsmoothed magnitude-weighted horizontal spectra for Deep/Soft Soil for the Lee Valley Dam site.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>150yrs</th>
<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
<th>10,000yrs</th>
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</thead>
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Note1: See Table 10 for recommended smoothed spectra

In NZS1170.5, the Z-factor corresponds to half the 500-year value of the 0.5s spectral ordinate for Shallow Soil. The Z-value from this study is 0.29, close to its corresponding NZS1170 value of 0.3.

Figure 2  Unsmoothed magnitude-weighted Weak Rock spectra for the Lee Valley site.
4.2 Smoothing of the spectra

Smoothed design envelopes were developed to largely envelope the raw unsmoothed spectra from the hazard analyses for the requested return periods of 150, 500, 2500, 10,000 years and an additional return period of 5,000 years. The construction of these envelopes followed procedures similar to those used in developing code spectra, although different from the specific procedures used for NZS1170.5. Each smoothed spectrum comprises a segment rising linearly with period $T$ from the 0s value to period $T=0.1s$, a constant spectral acceleration plateau at the peak of the smoothed spectrum to a corner period $T_c$ and
descending branches in which the spectral acceleration reduces with increasing spectral period $T$. The smoothing procedure involves defining an appropriate amplitude and period band for the constant acceleration plateau, and approximating the descending branches by segments proportional to $T^{-\gamma}$, where the exponent $\gamma$ takes values such as $2/3$, $3/4$, 1 or 2 in various segments. The equations for the smoothed magnitude-weighted spectra are given in Table 5, with parameters in Tables 6 and 7.

**Table 5** Equations for the smoothed 150, 500, 2500, 5000 and 10,000 year horizontal spectra.

<table>
<thead>
<tr>
<th>Value and Range</th>
<th>Equation to obtain value</th>
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</thead>
<tbody>
<tr>
<td>SSA($T=0$s)</td>
<td>RSA(0$s$)</td>
</tr>
<tr>
<td>SSA($0$s&lt;$T&lt;T0$)</td>
<td>RSA(0$s$) + ($T/T0$)*($SA_{max}$-RSA($0$s))</td>
</tr>
<tr>
<td>SSA($T0$≤$T$≤$Tc$)</td>
<td>$SA_{max}$</td>
</tr>
<tr>
<td>SSA($Tc$&lt;$T&lt;Tv$)</td>
<td>RSA($Tref$) * ($Tref/T$)$^{0.75}$</td>
</tr>
<tr>
<td>SSA($Tv$&lt;$T&lt;Td$)</td>
<td>SSA($Tv$) * ($Tv/T$)</td>
</tr>
<tr>
<td>SSA($T&gt;Td$)</td>
<td>SSA($Td$)*($Td/T$)$^{2}$</td>
</tr>
<tr>
<td>$SA_{max}$</td>
<td>RSA($Tref$) * ($Tref/Tc$)$^{0.75}$</td>
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<td>SSA($Tv$)</td>
<td>RSA($Tref$) * ($Tref/Tv$)$^{0.75}$</td>
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**Table 6** Parameter values for the equations in Table 5.

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<th>$Tv$</th>
<th>$Td$</th>
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<td>0.75</td>
<td>3</td>
</tr>
<tr>
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Table 7  
Parameter values for the equations in Table 5 (cont).

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<th>Return period (yrs)</th>
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<th>RSA (Tref)(g)</th>
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Figure 5, Figure 6 and Figure 7 show comparisons of the raw spectra to the smoothed spectra. The smoothed spectral values for the requested periods are shown in Table 8, Table 9 and Table 10 and Table ES-1, Table ES-2 and Table ES-3.

**MAGNITUDE-WEIGHTED, SMOOTHED WEAK ROCK SPECTRA, LEE VALLEY DAM**

![Graph showing magnitude-weighted, smoothed weak rock spectra](image)

**Figure 5**  
Horizontal spectra for Weak Rock for the Lee Valley site showing smoothed and unsmoothed spectra.
Figure 6  Horizontal spectra for Shallow Soil for the Lee Valley site showing smoothed and unsmoothed spectra.

Figure 7  Horizontal spectra for Deep/Soft Soil for the Lee Valley site showing smoothed and unsmoothed spectra.
Table 8  Smoothed magnitude-weighted Weak Rock hazard spectra.

<table>
<thead>
<tr>
<th>5% Damped Acceleration Response Spectra SA(T) (g)</th>
<th>Period</th>
<th>150yrs</th>
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<th>2500yrs</th>
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Table 9  Smoothed magnitude-weighted Shallow Soil hazard spectra.

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<th>Period</th>
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<th>500yrs</th>
<th>2500yrs</th>
<th>5000yrs</th>
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<td>T(s)</td>
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Table 10

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<td>1.52</td>
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<tr>
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<td>1.28</td>
<td>1.52</td>
</tr>
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<td>0.73</td>
<td>1.06</td>
<td>1.28</td>
<td>1.52</td>
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<td>0.45</td>
<td>0.73</td>
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<td>1.28</td>
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<td>1.28</td>
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<tr>
<td>0.5</td>
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<td>1.06</td>
<td>1.28</td>
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<td>1.28</td>
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<td>1.28</td>
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<td>0.22</td>
<td>0.37</td>
<td>0.63</td>
<td>0.76</td>
<td>0.90</td>
</tr>
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<td>2</td>
<td>0.17</td>
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<td>0.47</td>
<td>0.57</td>
<td>0.68</td>
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<tr>
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<td>0.13</td>
<td>0.22</td>
<td>0.38</td>
<td>0.46</td>
<td>0.54</td>
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<td>0.15</td>
<td>0.31</td>
<td>0.38</td>
<td>0.45</td>
</tr>
</tbody>
</table>

4.3 Deaggregation of the Hazard

Table 11 provides a breakdown of the contributions to the exceedance rates of magnitude-weighted peak ground accelerations by magnitude. Figure 8 shows a typical magnitude-weighted pga deaggregation by magnitude and distance, for a return period of 2500 years. The prominent peak in the magnitude ranges centred on magnitude 7.8 corresponds, in the most part, to the Wairau Fault’s component of the hazard with a much smaller contribution from the Alpine Fault at about 43 km distance from the site. Despite its prominence in the plot, the Wairau Fault peak corresponds to only about 16% of the exceedances of the 2500-year pga. Other contributions come from the Waimea North Fault which contributes about 11% of the hazard in the peak centred on magnitude 7.4 and the Waimea South Fault which produces about 9% of the hazard in the peak centred on magnitude 7.0. Most of the rest of the contribution to the hazard rate comes from distributed background seismicity, shown on the chart in the magnitude range 5.0 – 6.9. The mean magnitude of the contributions to the pga hazard ranges from about 6.3 to 6.5 for return periods from 150 years to 10,000 years (Table 11).
Table 11  
Percentage contributions to exceedance rates of peak ground accelerations based on magnitude-weighted spectra.

<table>
<thead>
<tr>
<th>Magnitude range</th>
<th>150yr pga</th>
<th>500yr pga</th>
<th>2500yr pga</th>
<th>10,000yr pga</th>
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</thead>
<tbody>
<tr>
<td>4.9-5.1</td>
<td>8.5</td>
<td>7.9</td>
<td>7.0</td>
<td>6.4</td>
</tr>
<tr>
<td>5.1-5.3</td>
<td>14.1</td>
<td>13.1</td>
<td>11.6</td>
<td>10.5</td>
</tr>
<tr>
<td>5.3-5.5</td>
<td>11.0</td>
<td>10.4</td>
<td>9.2</td>
<td>8.1</td>
</tr>
<tr>
<td>5.5-5.7</td>
<td>8.6</td>
<td>8.2</td>
<td>7.2</td>
<td>6.4</td>
</tr>
<tr>
<td>5.7-5.9</td>
<td>6.8</td>
<td>6.5</td>
<td>5.8</td>
<td>5.2</td>
</tr>
<tr>
<td>5.9-6.1</td>
<td>5.4</td>
<td>5.2</td>
<td>4.7</td>
<td>4.2</td>
</tr>
<tr>
<td>6.1-6.3</td>
<td>4.3</td>
<td>4.1</td>
<td>3.8</td>
<td>3.4</td>
</tr>
<tr>
<td>6.3-6.5</td>
<td>3.5</td>
<td>3.3</td>
<td>3.1</td>
<td>2.8</td>
</tr>
<tr>
<td>6.5-6.7</td>
<td>2.9</td>
<td>2.7</td>
<td>2.5</td>
<td>2.3</td>
</tr>
<tr>
<td>6.7-6.9</td>
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<td>2.3</td>
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<tr>
<td>6.9-7.1</td>
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<td>6.0</td>
<td>9.0</td>
<td>9.5</td>
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<td>7.1-7.3</td>
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<td>0.1</td>
<td>0.0</td>
<td>0.0</td>
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<tr>
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<td>2.2</td>
<td>4.4</td>
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<tr>
<td>7.5-7.7</td>
<td>5.7</td>
<td>2.2</td>
<td>0.2</td>
<td>0.0</td>
</tr>
<tr>
<td>7.7-7.9</td>
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<td>22.5</td>
<td>22.0</td>
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</tr>
<tr>
<td>7.9-8.1</td>
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<td>0.0</td>
<td>0.0</td>
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</tr>
<tr>
<td>8.1-8.3</td>
<td>2.0</td>
<td>0.9</td>
<td>0.0</td>
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<tr>
<td>8.3-8.5</td>
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<td>0.0</td>
<td>0.0</td>
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<tr>
<td>8.5-8.7</td>
<td>0.4</td>
<td>0.1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Average magnitude 6.29 6.35 6.45 6.54

Figure 8  
2500-year peak ground acceleration deaggregation plot for the Lee Valley site. The horizontal axes are magnitude and source-site distance (km).
4.4 Near-Fault Factors

The possible need for near-fault factors, usually applied (when required) only to hazard spectra for return periods of 500 years and above, was considered as part of this study. Near-fault factors are used to allow for added directivity effects caused by faults capable of producing large earthquakes very close to the site being considered. The method used to determine the level of near-fault effects at the Lee Valley site was to take the Wairau Fault, Waimea North Fault and the Waimea South Fault as bases. The Wairau Fault causes most of the hazard at the site, but the Waimea North Fault has the potential to cause the most severe acceleration values and is also the closest of the faults included in the 2010 NSHM. The near-fault factors were calculated by considering five possible rupture initiation points at equally spaced locations along the fault (at the ends and the three quarter-points along the fault for strike-slip faults, at equal intervals on a vertical profile from the base to the top for dip-slip faults). The factors for each of these scenarios are calculated using the method of Somerville (Somerville et al., 1997), which has different models for strike-slip and dip-slip faults. The strike-slip model, which is appropriate for the Wairau Fault, was used in developing the NZS1170 near-fault factors. The dip-slip model is appropriate for the Waimea North Fault and Waimea South Fault. Figure 9 and Figure 10 show the calculated near-fault factors for the rupture of the Waimea North and South Faults and the Wairau Fault respectively.

The dip-slip model for the Waimea South Fault results in maximum (worst case) factors that marginally exceed 1. The Waimea North Fault results in a maximum (worst case) factor of about 1.2 at 3 seconds spectral period. However, given the long recurrence interval (9600 years) of this fault it is not considered appropriate to apply this factor to the hazard spectra. The maximum factors are shown in Figure 9.

For the Wairau Fault, the average near-fault factor never exceeds 1 (Figure 10).

Based on the values resulting from this part of the study for average directivity effects from these faults, we recommend that the near-fault factor be taken as 1.

![Near-fault factors for Lee Valley](image)

**Figure 9** Worst case near-fault factors for Lee Valley, rupture of the Waimea North and South Faults.
Near-fault factors for Lee Valley for rupture of the Wairau Fault

Figure 10  Near-fault factors for Lee Valley, rupture of the Wairau Fault.

4.5 Scenario spectra

Figure 11 shows comparisons of the smoothed 2500-, and 10,000-year 5% damped spectra, together with an additional 5000-year smoothed hazard spectrum that was not requested in the original brief and scenario spectra considered appropriate for comparison. The scenario spectra shown relate to the Waimea South and the Wairau Fault modelled at the 84-percentile level. Neither of these spectra approach the 10,000-year hazard spectra, however they both exceed the 2500-year hazard spectra for varying amounts of the spectral period window. Both scenario spectra fall below the additional 5000-year hazard spectra for most spectral periods. Although Figure 11 shows only the scenario spectra comparison for Shallow Soil ground conditions, the other ground classes show similar results. Other faults were considered but are not shown in Figure 11 for the sake of clarity. The Waimea North Fault was considered at a 50-percentile level, because of its long recurrence interval, and the resulting spectrum fell below the 2500-year hazard spectra for most spectral periods. The 84-percentile scenario spectrum for the closest segment of the Alpine Fault to the site lies part way between the 500-year and 2500-year hazard spectra.

These results indicate that the scenario spectra corresponding to 84-percentile motions on the Wairau and Waimea South Faults are alternative candidates for the MDE motions, in lieu of the 10,000-year spectra required by the NZSOLD Guidelines for probabilistically-derived MDE motions. The Waimea South Fault gives stronger motions up to 0.2s period and the Wairau Fault for longer spectral periods. The envelope of these two scenario spectra can be conveniently represented by the 5000-year hazard spectra.
Figure 11  Comparison of the smoothed hazard spectra and scenario spectra for the Waimea South and Wairau Faults for the proposed Lee Valley site.

4.6  Comparison with NZS1170 spectra

Figure 12 shows a comparison of the NZS1170 code spectra for $Z = 0.30$, the value for the Lee Valley site in NZS1170, and the recommended smoothed hazard spectra for Shallow Soil for return periods up to 2500 years, the maximum covered by NZS1170 (high PIC dams required consideration of return periods up to 10,000 years). With the exception of the 150-year spectrum, the NZS1170 curves lie above the equivalent hazard spectra. This is partially due to the slight decrease in $Z$-value for the site in the latest version of the NSHM. The shapes of the hazard curves are generally similar to the equivalent code spectrum. For the shallow soil class shown in Figure 13, the 150- and 500-year hazard curves have a steeper slope between 0.75 seconds and 1.5 seconds spectral period, while the 2500- and 10,000-year hazard curves have a shallower slope beyond between 1.5 seconds and 2 seconds. The comparisons for the other site classes are similar, with differences in slopes between the smoothed hazard spectra and the NZS1170 spectra occurring only over limited spectral period ranges that depend on the site class and return period.
5.0 ACTIVE FAULTING IN THE VICINITY OF THE LEE VALLEY SITE

5.1 Waimea-Flaxmore fault system

The Waimea-Flaxmore fault system is the closest known active fault, or fault system, to the proposed Lee Valley site. The Waimea-Flaxmore fault system has an approximate length of ca 150 km, and extends from near St Arnaud in the southwest (where it intersects the Alpine Fault) to near D’Urville Island in the northeast. At its closest, it passes within about 8 to 9 km northwest from the proposed site. The Waimea-Flaxmore fault system encompasses a number of active folds and faults (e.g. Bishopdale, Eighty Eight, Flaxmore, Waimea) within a zone up to several kilometres wide (e.g. Fraser et al. 2006, Johnston 1982, Rattenbury et al. 1998). Faults within the Waimea-Flaxmore system typically have moderate to steep dips to the southeast, and predominantly a reverse sense of displacement (with a subordinate component of dextral strike-slip).

The Waimea-Flaxmore fault system has not ruptured the ground surface and generated a large magnitude earthquake within historic times. The paleoearthquake investigations of Fraser et al. (2006), south of Nelson city, indicate that this portion of the Waimea-Flaxmore fault system last ruptured about 6200 years ago, and has an average recurrence interval of surface fault rupture earthquakes of about 6000 years (based on the timing of three surface fault rupture earthquakes which are presumed to be the three most recent ones). This southern portion of the Waimea-Flaxmore fault system (termed Waimea South in the National Seismic Hazard Model) is considered capable of generating earthquakes in the order of M 7, based on fault length and single-event displacement size considerations.

The paleoseismicity of the northern portion of the Waimea-Flaxmore fault system (termed
Waimea North in the National Seismic Hazard Model) is not nearly as well studied as the south. It presumably has a longer rupture length which would imply a larger single-event displacement size (coseismic rupture displacement scales with rupture length). If the northern and southern portions of the Waimea-Flaxmore fault system have the same slip rate (and there is currently no reason to suggest that they don’t) then this would suggest that Waimea North would have a longer recurrence interval than Waimea South (recurrence interval, in this case, being approximated by dividing single-event displacement size by slip rate). In the National Seismic Hazard Model, Waimea North in considered capable of generating M 7.4 earthquakes with an average recurrence interval of about 9600 years. The larger earthquake size, and longer recurrence interval of Waimea North, compared to Waimea South, are consistent with its inferred longer rupture length and implied larger single-event displacement size.

5.2 Wairau Fault

The Wairau Fault is as little as 21 to 22 km south-southeast from the site. The Wairau Fault is the northern section of the Alpine Fault, and extends from the Nelson Lakes area in the west-southwest to offshore Cook Strait in the east-northeast. Like the Waimea-Flaxmore fault system, the Wairau Fault has not ruptured in a large earthquake in historic times. Paleoearthquake investigations on the Wairau Fault, both on-shore and off, indicate that the fault most recently ruptured the ground surface about 2000 years ago, and that it has a recurrence interval of surface fault rupture earthquakes in the order of 2000 to 3000 years (Barnes & Pondard 2010, Zachariasen et al. 2006). Single-event surface rupture displacements of about 6 m, or more, have been documented on the fault, and this, along with its anticipated surface rupture length are consistent with the fault being capable of generating moderate to high magnitude 7 earthquakes. In the current National Seismic Hazard Model the Wairau Fault is modelled as a M 7.8 earthquake source with a recurrence interval of 2500 years, these parameters differ from those used in some previous versions of the model (M 7.6 earthquakes with a recurrence of 1600 years). However, the effect of the different magnitude and recurrence interval on the level of hazard at the Lee valley site is negligible.

5.3 Assessment of active faulting in the immediate vicinity of the proposed site

To assess the potential for active fault displacement through the proposed Lee Valley site, a review was undertaken of existing geological maps (Johnston 1982, Rattenbury et al. 1998), the GNS Active Fault Database (http://data.gns.cri.nz/af/), and several different scales of stereo vertical aerial photography (photos: 4035, 11-14; 4279, 8-16; 1210; 40-44; 1211, 42-48; 1212, 41-46). There are no active fault traces shown on existing geological maps, nor in the GNS Active Fault Database that are near the immediate vicinity of the proposed site. As mentioned above the closest known active fault is the Waimea-Flaxmore fault system about 8 to 9 km distance from the site. Review of the above aerial photographs also did not reveal any topographic evidence for the existence of active fault traces in the immediate vicinity of the proposed site. It appears, from available data, that the site is free of active fault displacement hazard.
As is the case with all investigations of this sort, it is impossible to categorically rule-out any possibility of fault displacement at the site. If past displacements were small and/or occurred sufficiently long ago then evidence of these displacements in the landscape could have been eroded, and may go undetected. Accordingly, we recommend that if, and when, the proposed site gets cleaned-down, rock defects at the site, if present, be examined for possible evidence of geologically recent displacement (e.g. the presence of soft clay gouge).

6.0 DISCUSSION

Site-specific horizontal spectra have been developed for a seismic review of the proposed Lee Valley Dam site. Smoothed spectra for return periods ranging from 150 years to 10,000 years are presented in Table ES-1, Table ES-2 and Table ES-3 and Figure ES-1, Figure ES-2 and Figure ES-3. Features of the estimated site-specific earthquake hazard are:

- The results are provided for NZS1170 Class B Weak Rock, Class C Shallow Soil and Class D Deep/Soft Soil conditions;
- The main contribution to the estimated hazard is provided by magnitude 7.8 earthquakes on the closest segment of the Wairau Fault, at a closest distance of about 21 km from the substation and with a recurrence interval of 2500 years. The Waimea North Fault lies about 8 km from the site and is capable of producing magnitude 7.4 earthquakes; however, this contributes less to the overall hazard because of its far greater recurrence interval, estimated to be 9600 years. The Waimea North Fault does, however, have the potential to produce the largest single-event acceleration values at the site;
- Near-fault factors for rupture-scenarios of the Wairau Fault are less than 1.0 on average for the Lee valley site. Dip-slip rupture-scenarios of the Waimea North Fault result in values of 1.2 or less and given it's long recurrence interval of about 9600 years it is recommended that the near-fault factor be taken as 1.0;
- The estimated hazard-derived Z value is 0.29 compared with the NZS1170 value of 0.30;
- The NZSOLD Dam Safety Guidelines specify that the return period for Operational Basis Earthquake (OBE) motions is 150 years, and allow adoption of a probabilistically-based 10,000-year spectrum or scenario spectra for the estimated motions from rupture of nearby faults to represent the Maximum Design Earthquake (MDE) motions;
- Accordingly, the smoothed 150-year motions listed in Tables ES-1 to ES-3 are recommended as the OBE motions for the three site classes;
- The results presented here suggest that the envelope of the 84-percentile spectra for a magnitude 7.0 earthquake on the Waimea South Fault at 8 km distance and a magnitude 7.8 earthquake on the Wairau Fault at 21 km distance, which can be conveniently approximated by the smoothed 5000-year spectra of Tables ES-1 to ES-3, are sufficient to represent the MDE motions, in lieu of the purely probabilistically-based 10,000-year spectra;
- A review of existing geological data shows that there is no evidence of active fault traces in the immediate vicinity of the proposed site. However, we recommend that if, and when, the proposed site gets cleaned-down then significant rock defects at the site, if present, be examined for possible evidence of geologically recent displacement.
7.0 ACKNOWLEDGEMENTS

Rob Langridge and Angelique Zajec are acknowledged for providing fault maps. This report has been reviewed by Dr. Jim Cousins and Dr. Rob Langridge of GNS Science.

8.0 REFERENCES


APPENDIX 1 ACCELEROGRAMS FOR LEE VALLEY

This Appendix presents tables and plots of the $k_1$-scaling factors (as defined in NZS1170) required for each selected accelerogram to best match the recommended smoothed horizontal spectra for Lee Valley. Accelerograms have been selected to represent the “seismic signature” of the Lee Valley site as closely as possible i.e. providing a close match of the hazard spectra while reflecting the magnitude, distance, earthquake type and site conditions appropriate for Lee Valley. Scale factors were calculated using the procedures of NZS1170.5, matching the 5000-yr Rock spectra at the Lee Valley site.

As shown in a previous section, the principal contributions to the earthquake hazard (i.e. the rate of exceedance of the response spectral acceleration values) affecting Lee Valley for a return period of 5000 years are provided by one reverse and two strike-slip faults, namely the Waimea North and Wairau Faults and the closest section of the Alpine Fault which can contribute at longer periods. The faults are distances of 8 – 43 km from the site, with magnitudes in the range 7.0 to 7.8 (Table 1). The target parameters sought in selecting accelerograms are those from earthquakes of a similar magnitude range recorded within a similar distance of the source, with spectral shapes that provide good matches to the hazard spectral shapes. The recommended records, their GNS identifier, associated magnitudes, source-to-site distances, mechanisms and site descriptions are summarised in Table A1.

The El Centro record from the magnitude 7.0 strike-slip Imperial Valley earthquake of 1940 is included as a reference record, because of its long history as a design accelerogram. The characteristic of the El Centro record of nearly constant spectral velocity over a broad period range often makes it a good spectral match to design spectra. It often provides more demanding motions than those of other records scaled to the same target spectrum.

### Table A1 Records Selected as representative of Rock spectra for Lee Valley

<table>
<thead>
<tr>
<th>Accelerogram</th>
<th>$M_w$</th>
<th>Distance (km)</th>
<th>Mechanism</th>
<th>Site Description</th>
<th>Primary component</th>
<th>Secondary component</th>
</tr>
</thead>
<tbody>
<tr>
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<td>7.0</td>
<td>10</td>
<td>Strike-slip</td>
<td>Rock</td>
<td>N90W</td>
<td>S00E</td>
</tr>
<tr>
<td>Imperial Valley 1940</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abbar Iran F9016331</td>
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<td>Strike-slip</td>
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<td>N68W</td>
<td>S22W</td>
</tr>
<tr>
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<td>N90E</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td>16 September 1978</td>
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</tr>
</tbody>
</table>

The scale factors $k_1$ for each record as a function of matching period $T_{\text{match}}$ for the range $T_{\text{match}}=0.4s$ to $T_{\text{match}}=6s$ are presented (Figures A1, A3, A5 and A7) together with plots indicating the goodness-of-fit of the records to the target spectra (Figures A2, A4, A6 and A8).
k₁, horizontal scaling factors

For each accelerogram, the upper half of each page in the following figures outlines the details of the parameters used to calculate the scale factor required to match each horizontal component of the chosen accelerogram to the 5000-year hazard spectrum. The only information of relevance for applying the accelerograms are the scaling factors $K_{\text{first}}$ and $K_{\text{second}}$ for the period of interest together with their associated errors RMS1 and RMS2 as a function of structural period $T_{\text{fit}}$. Also associated with each value of $T_{\text{fit}}$ is the period band $T_{\text{min}}$ to $T_{\text{max}}$, corresponding to $0.4T_{\text{fit}}$ to $1.3T_{\text{fit}}$, over which the matching was performed. These bands correspond to those used in the accelerogram scaling procedures given in NZS1170.5. The results are presented both in tabular and graphical form, giving $K(T_{\text{fit}})$ against period $T_{\text{fit}}$. The other parameters relate to the type of matching performed. The second figure on each page is a representative plot demonstrating how the scaled values of the two components compare with target spectrum for $T_{\text{fit}}=1.0$ seconds. The values of $K_{\text{first}}$ and $K_{\text{second}}$ associated with the $T_{\text{fit}}=1$ second values in the upper table therefore also appear in the text associated with the lower graph. Similar plots are available for a selection of periods (0.4s, 0.5s, 1s, 1.5s, 2s and 2.5s). The scaling factors should be selected based on the estimated period $T$ of the structure. The listed scaling factors may be linearly interpolated for intermediate periods. RMS1 and RMS2 give the root mean square error over the period band between the logarithm to base 10 of the target spectrum and the spectrum of the scaled accelerogram corresponding to the best fit for each component. These values correspond to factors given by $10^{\text{RMS1}}$ and $10^{\text{RMS2}}$ for the spectra themselves. Values of $10^{\text{RMS}}$ of less than 1.2 correspond to excellent matches, values between 1.2 and 1.4 are good matches, values of 1.4 to 1.5 are marginal matches, while higher values indicate poor fits and indicate that the accelerogram is not appropriate for that period range.

The $k₁$ horizontal scaling factor required for each particular accelerogram is the smaller of the values listed for the fundamental period $T$ of interest, in the columns $K_{\text{first}}$ and $K_{\text{second}}$. The smaller of these values is used to determine the stronger, or principal, horizontal component in the associated period band $T_{\text{min}} (=0.4 T_{\text{fit}})$ to $T_{\text{max}} (=1.3 T_{\text{fit}})$. For some records, the principal component changes with period. Components 1 and 2 correspond to their order of listing in the accelerogram time-history and response spectra files, and are noted in Table A1.

The smaller of the scaling factors $K_{\text{first}}$ and $K_{\text{second}}$ is equivalent to the record scale factor $k₁$ in Section 5.5.2 of Standard NZS1170.5 for earthquake actions in New Zealand, taking the structural performance factor $S_p$ as 1.0 (values for other $S_p$ factors can be obtained by multiplying by $(1+ S_p)/2$). This is the factor that produces a least-squares match of the log of the accelerogram spectrum to the log of the target spectrum over the period band 0.4T to 1.3T, consistent with the requirements of NZS1170.5. For each record:

Total scaling factor = $k₁k₂((1+ S_p)/2)$

$k₁$ is the smaller of $k_{\text{first}}$ or $k_{\text{second}}$ for the period range of interest and $S_p$ is the adopted structural performance factor. The scale factors are reported for matching the 5000-yr hazard spectra.

$k₂$ is a family scaling factor that may be required in some cases to ensure that every point on the target spectrum in the target period is exceeded by at least one of the spectra in the family of scaled principal component accelerograms. The complete family is required to be
multiplied by this second factor $k_2$ if it is greater than one. Usually this is determined for specific periods of interest. Table A2 lists $k_2$ values for a range of periods from $T = 0.4$ to $T = 6$ seconds.

**Table A2  $k_2$ factors for a range of periods**

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<thead>
<tr>
<th>Period T(s)</th>
<th>$k_2$</th>
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**Rock accelerograms**

**El Centro record, Imperial Valley earthquake 1940**

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<th>Rrup (km)</th>
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</table>

| R= | 1.00 | SAref= | 0.41 | CF= | 1.30 |
| IMSF= | 0 | Magnitude= | 7.00 | MSF= | 1.00 |

| IFIT= | 3 | ISRSS= | 0 | IG= | 0 | ILARGE= | 1 |

Matching= Primary Match  
Scaling factors RMS1= RMS2=  
$T(fit)$ $T_{min}$ $T_{max}$ $N_{periods}$ $K_{first}$ $K_{second}$ $\text{rms log(error1)}$ $10^*\text{RMS1}$ $\text{rms log(error2)}$ $10^*\text{RMS2}$  
0.4 0.16 0.52 31 2.47 1.70 0.140 1.38 0.116 1.31  
0.5 0.20 0.65 32 2.06 1.41 0.141 1.38 0.135 1.37  
1.0 0.40 1.30 23 1.69 1.21 0.058 1.14 0.095 1.24  
1.5 0.60 1.95 21 1.93 1.58 0.085 1.22 0.131 1.35  
2.0 0.80 2.60 21 1.88 1.67 0.077 1.19 0.099 1.26  
2.5 1.00 3.25 23 1.83 1.82 0.073 1.18 0.082 1.21  
3.0 1.20 3.90 24 1.76 2.02 0.075 1.19 0.091 1.23  
4.0 1.60 5.20 26 1.62 2.33 0.116 1.31 0.116 1.31  
6.0 2.40 7.80 20 1.31 2.47 0.088 1.22 0.095 1.24  

**Figure A1  Scaling factors and RMS errors for best matches to 5000-yr Rock spectrum.**
Figure A2  
Best matches of scaled El Centro 1940 components to 5000-yr Rock spectrum at 1 second. El Centro 1940 is a standard reference accelerogram from a strike-slip earthquake.

**Abbar Iran accelerogram**

<table>
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Figure A3  
Scaling factors and RMS errors for best matches to 5000-yr Rock spectrum.
Figure A4  
Best matches of scaled Abbar, Iran 1990 accelerogram components to 5000-yr Rock spectrum at 1 second.

Izmit accelerogram

<table>
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<th>Location</th>
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IZMITF99606Z2 matching Lee Valley Rock

R=1 SAre=0.412

Primary Match

Figure A5  
Scaling factors and RMS errors for best matches to 5000-yr Rock spectrum.
Figure A6  Best matches of scaled Izmit 1999 accelerogram components to 5000-yr Rock spectrum at 1 second.

Tabas accelerogram

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Figure A7  Scaling factors and RMS errors for best matches to 5000-yr Rock spectrum.
Figure A8  Best matches of scaled Tabas 1978 accelerogram components to 5000-yr Rock spectrum at 1 second.
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<td>Wairakei Research Centre</td>
</tr>
<tr>
<td>Avalon</td>
<td>114 Karetoto Road</td>
</tr>
<tr>
<td>PO Box 30368</td>
<td>Wairakei</td>
</tr>
<tr>
<td>Lower Hutt</td>
<td>Private Bag 2000, Taupo</td>
</tr>
<tr>
<td>New Zealand</td>
<td>New Zealand</td>
</tr>
<tr>
<td>T +64-4-570 1444</td>
<td>T +64-7-374 8211</td>
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<tr>
<td>F +64-4-570 4600</td>
<td>F +64-7-374 8199</td>
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Seismic Hazard Assessment for the Proposed Waimea Dam

GH McVerry        RJ Van Dissen        E. R. Abbott

GNS Science Consultancy Report 2017/150
September 2017
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Use of Data:

Date that GNS Science can use associated data: April 2017

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EXECUTIVE SUMMARY

Acceleration response spectra for 5% damping have been estimated for Waimea Dam for NZS1170.5 Site Class B Rock site conditions, with an assumed average shear-wave velocity Vs30 over the top 30 metres of 800m/s, as assigned for this site class by Bradley (2013). The study differs from the earlier study of Buxton et al. (2011) by incorporating an updated seismicity model, including modelling of the Waimea Fault as three rather than two source segments, and by using the weighted combination of five ground-motion prediction equations (GMPEs) rather than the one used in 2011.

The five ground-motion prediction equations used are: the New Zealand models of McVerry et al. (2006) and Bradley (2013), and three models from the NGA-West 2014 GMPE study, namely Abrahamson, Silva & Kamai (ASK, 2014); Boore, Stewart, Seyhan and Atkinson (BSSA, 2014); and Campbell & Bozorgnia (CB, 2014). The weights for each of the models were: ASK 1/6; BSSA 1/6; CB 1/6; Bradley 3/10 and McVerry 2/10. The McVerry model characterises site conditions through the NZS1170 site classes, while the other models use Vs30.

Probabilistic spectra have been estimated for return periods of 150, 500, 2500 and 10,000 years. Deterministic spectra for various rupture scenarios have also been produced, including considering multi-fault ruptures (combined Waimea Central and South fault segments, combined Waimea South and Alpine Kaniere-Tophouse source, and combined Wairau and Alpine Kaniere-Tophouse source).

Tables ES1 and ES2 respectively list the probabilistic mean and 84-percentile estimates of the 5% damped acceleration response spectra for the four return periods. The results are given both unweighted (UW) and with magnitude-weighting (MW) up to periods of 0.5s. The weighting for magnitude M is \((M/7.5)^{1.285}\), as used in developing the spectra of NZS1170.5:2004. The values are for RotD50 (very similar to the geometric mean) versions of the GMPEs. Hanging wall factors have been incorporated in all the GMPEs. The NZSOLD Large Dam Guidelines require the mean estimate of the 10,000-year spectrum for the Safety Evaluation Earthquake (SEE) motions, if they are determined probabilistically.

Figures ES1 and ES2 show the mean unweighted spectra on linear and log-log plots. These figures also show the mean spectra for the case where the average recurrence interval of the southern segment of the Waimea Fault has been reduced from 5600 years to 4000 years, in recognition of the possibility that the slip rate of the Waimea Fault increases towards the south as it becomes closer to the higher strain-rate Wairau and Alpine faults. The effect of this change on the hazard estimates is slight, a maximum of less than 2% at the peak of the 10,000-year spectrum, and generally much less than that.

Figures ES3 and ES4 indicate the variation of results between the GMPEs by showing the probabilistic 50- and 84-percentile unweighted spectra across the GMPEs, as well as the mean spectra shown in Figures ES1 and ES2. The 50-percentile spectra are very similar to the mean spectra listed in Table ES1, and are virtually indistinguishable from them in the plots, except at long spectral periods and return periods.

Figures ES5 and ES6 compare the unweighted and magnitude-weighted spectra, on linear and log scales. Magnitude-weighting generally has only minor effects on these spectra, with the largest effects at the peaks of the spectra, which are reduced by about amounts ranging from about 15% for the 150-year spectrum down to about 4% for the 10,000-year spectrum.
Spectra for three multi-segment fault-rupture scenarios have been considered as alternatives to the mean 10,000-year spectrum for the Safety Evaluation Earthquake (SEE) motions. The scenarios considered were: combined rupture of the central and southern segments of the Waimea Fault in a magnitude 7.5 earthquake at a shortest distance of about 8 km from the dam site; combined rupture of the Waimea South and Alpine sources in a magnitude 7.8 earthquake at a shortest distance of about 12 km from the dam site; and combined rupture of the Wairau and Alpine Faults in a magnitude 8.3 earthquake at about 21 km shortest distance. The mean 50th- and 84th-percentile estimates of these scenario spectra are shown in Figures ES7 and ES8, in linear and log plots. The spectra are the weighted combination of the 5 ground-motion prediction equations considered. Also shown are the mean uniform hazard spectra for return periods of 150, 500, 2500 and 10,000 years. None of these spectra are magnitude-weighted. The 84th-percentile scenario spectra range from around the 2500-year motions to stronger than the 10,000-year motions. The strongest 84th-percentile scenario estimates, for the combined rupture of the central and south segments of the Waimea Fault, exceed the mean 10,000-year spectrum, so need not be considered for the SEE motions according to the NZSOLD (2015) Guidelines. The recommended SEE spectrum is the mean 10,000-year spectrum (Table ES3). The mean estimate of the 84th-percentile motions for the combined Alpine-Waimea South sources is very similar to the 10,000-year probabilistically-based SEE spectrum. The probabilistic spectra estimated in the current study (Figure ES9) are reduced from those of the 2011 study for all return periods for spectral periods of 0.25s and longer. The change appears to result mainly from the seismicity model rather than the use of a combinations of GMPEs in place of the single one used in 2011.

The PGA values for the SEE motions are enhanced by about one-third from the 2011 magnitude-weighted value of 0.48g, to 0.62g magnitude-weighted or 0.64g unweighted. However, for all periods of 0.25s or longer, the recommended SEE spectrum of the current study falls below the MDE spectrum of the 2011 study, despite being associated with a longer return period of 10,000 years rather than 5000 years.

The main contribution (about 60% of the total) to the exceedance rate of the 10,000-year spectrum is from the central and south segments of the Waimea Fault, modelled as producing magnitude 7.1 earthquakes at distances of 8 km and 12 km, respectively, from the dam site, with average recurrence intervals of rupture of about 6000 years for both sources. The contribution-averaged magnitude for the 10,000-year peak ground accelerations is 7.2, because of the contributions of larger magnitude sources in addition to those of the Waimea Fault.

The recommended aftershock spectrum (Table ES4 and Figure ES10) corresponds to the 84th-percentile spectrum for a magnitude 6.8 earthquake at 12 km distance from the dam site, following a magnitude 7.8 main-shock corresponding to a combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments. This is consistent with the 84th-percentile main-shock spectrum being similar to the probabilistic 10,000-year SEE spectrum.
Table ES 1  Summary of mean estimates of 5% damped unweighted (UW) and magnitude-weighted (MW) acceleration response spectra for Waimea dam for preferred fault source parameters.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>Mean 5% damped acceleration response spectra SA(T) (g)</th>
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<td>Return Period</td>
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<tr>
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Table ES 2  Summary of 84-percentile estimates of 5% damped unweighted (UW) and magnitude-weighted (MW) acceleration response spectra for Waimea dam for preferred fault source parameters.

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<th>Period T(s)</th>
<th>84-percentile 5% damped acceleration response spectra SA(T) (g)</th>
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Table ES 3  Summary of mean estimates of 5% damped unweighted acceleration response spectra for Waimea dam for a return period of 10,000 years and for the 84th-percentile spectra for three multi-segment fault-rupture scenarios. The 10,000-year spectrum is recommended from these candidates for the SEE spectrum.

<table>
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<th>Period T(s)</th>
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Table ES 4  Recommended aftershock spectrum for a magnitude M6.8 Waimea South and Alpine Fault event, consistent with the associated magnitude 7.8 main-shock spectrum being similar to the 10,000-year SEE spectrum.

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Figure ES 1  Waimea Dam mean 5% damped acceleration response spectra for return periods of 150, 500, 2500 and 10,000 years for preferred fault source model and model with shorter recurrence interval of 4000 years rather than 5600 years for the Waimea South fault source. There is no magnitude-weighting.

Figure ES 2  Spectra of Figure 1 on log-log plot.
**Figure ES 3** Mean spectra of Figure 1 for the preferred fault source model with the addition of the 50- and 84-percentile spectra for the weighted combination of all the GMPEs.

**Figure ES 4** Spectra of Figure 3 on log-log plot.
Figure ES 5  Waimea Dam mean spectra and mean magnitude-weighted spectra for return periods of 150, 500, 2500 and 10,000 years for the preferred fault source model.

Figure ES 6  Spectra of Figure 5 on log-log plot.
Figure ES 7  Comparison of mean uniform hazard spectra of Figure 1 with the mean estimates (over the 5 GMPEs) of the 50th - and 84th -percentile spectra for three multi-segment rupture scenarios. Magnitude-weighting is not included for any of the spectra. The mean 10,000-year spectrum is very similar to the mean estimate of the 84th-percentile motions for the combined Alpine-Waimea South sources. The strongest 84th-percentile scenario estimates, for the combined rupture of the central and south segments of the Waimea Fault, exceed the mean 10,000-year spectrum, so need not be considered for the SEE motions according to the NZSOLD (2015) Guidelines.

Figure ES 8  Spectra of Figure 7 on a log-log plot.
Figure ES 9  Comparison of the probabilistic spectra from the current study (solid curves) with those from the 2011 study (dashed curves).

Figure ES 10 Recommended aftershock spectrum (dash-dot curves), for a magnitude 6.8 event following a magnitude 7.8 earthquake associated with combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments, compared to the probabilistic spectra.
1.0 INTRODUCTION

1.1 TECHNICAL BRIEF

GNS Science was requested by Tonkin & Taylor Ltd on behalf of their client Tasman District Council to prepare an update of a site-specific hazard assessment for the Waimea (previously Lee Valley) Dam (GNS Science Consultancy Report 2011/26) to address the following issues:
1) Aftershock motions; 2) Findings from the Kaikoura and Canterbury earthquakes; 3) Outcomes from the resource consent process; and 4) Update due to the new NZSOLD guidelines.

The proposal stated:

‘Both magnitude-weighted and unweighted 5% damped acceleration response spectra and peak ground accelerations will be developed for horizontal motions for NZS1170.5 Class B Rock at the site of the proposed Waimea Dam, to satisfy the requirements of the 2015 NZSOLD New Zealand Dam Safety Guidelines (NZSOLD, 2015). Spectra will be produced for periods up to 3s. Results will be provided for return periods of 150, 500, 2500 and 10,000 years, together with deaggregations for the 10,000-year motions. The 150-year return period is appropriate for Operating Basis Earthquake (OBE) motions and the 10,000-year return period for the Safety Evaluation Earthquake (SEE) motions, if determined probabilistically, according to the NZSOLD Guidelines. The 500- and 2500-year motions satisfy the Ultimate Limit State requirements for Importance Level 2 (IL2) and IL4 structures on the site governed by the New Zealand structural design standard NZS1170.5:2004. Additionally, the 10,000-year return period motions will be compared with the strongest 84-percentile scenario motions for the controlling maximum earthquake (CME) (likely to be one of the segments of the Waimea Fault) and, if applicable, other key faults in the region. A simple deterministic approach will be used to estimate aftershock spectra, based on events one magnitude unit lower than the CME.

The results will be a major update of those provided in the GNS Science Consultancy report 2011/26 (Buxton, McVerry and Van Dissen, 2011). The need for the major update results from a change between the 2000 and 2015 NZSOLD Guidelines, namely the requirement that ‘Epistemic uncertainties associated with earthquake sources and ground motion prediction equations should be considered.’ In discussions with Tonkin & Taylor and their advisors, Ian Walsh of Opus and Trevor Matuschka of Engineering Geology Ltd, it was decided that the uncertainties will be considered through sensitivity studies rather than full logic tree analysis. Even with a sensitivity-analysis approach, the consideration of epistemic uncertainties represents a large increase in the calculations required compared to the 2000 Guidelines that were addressed in the 2011 study, requiring multiple representations of the main fault sources affecting the seismic hazard at the site, and the combination of results from multiple ground-motion prediction equations (GMPEs).

The 2011 study did not consider the estimation of vertical motions, and these have not been requested for the update.

The starting point for the modelling of the faults will be the 2010 National Seismic Hazard Model (NSHM), as published in Stirling et al. (2012), with modification from a two- to a three-segment representation of the Waimea Fault, as developed in the course of the resource consent process (e-mail 3 December 2014 from G. McVerry of GNS Science to M. Foley of Tonkin & Taylor). Uncertainties in fault parameters will be addressed by considering increased
and reduced values of the recurrence intervals and magnitudes of the Waimea North, Central and South fault segments, as well as preferred values, based on studies of the Waimea-Flaxmore fault system through to the present. In addition, consideration will be given to whether any additional earthquake sources are required to represent the Waimea-Flaxmore fault system, including taking into account information provided in Fraser et al. (2006) and Johnston and Nicol (2013).

One of the lessons from the Kaikoura earthquake was the possibility of ruptures extending along multiple faults, either as a single large source or one source triggering ruptures of neighbouring faults in the course of its propagation. To address this possibility, the sensitivity studies will include the estimation scenario motions for combined ruptures of two or three segments of the Waimea Fault, or of the Alpine Fault in conjunction with the Wairau or Waimea Faults.

Ground-motion uncertainty will be addressed by considering the two GMPEs most commonly used in New Zealand, namely McVerry et al. (2006) and Bradley (2013), together with one of the GMPEs from NGA-West project (Gregor et al. 2014) commonly used in California. The final selection of GMPEs and their weightings will not be pre-ordained; rather, the basis of their selection will be reviewed by Trevor Matuschka (Engineering Geology Ltd) when that stage of the work is reached.

In addition, it was agreed that the calculations are to be for the geometric-mean component, or the 50th-percentile orientation, which is close to the geometric mean, for the NGA models.

1.2 2015 NEW ZEALAND DAM SAFETY GUIDELINES

Since the preparation of the 2011 report (Buxton et al, 2011), the New Zealand Dam Safety Guidelines (NZSOLD, 2000) have been updated (NZSOLD, 2015).

The proposed Waimea Dam has a high Potential Impact Classification (PIC) according to the briefing information supplied by Tonkin & Taylor Limited. For high PIC Dams, the 2015 Guidelines allow the Safety Evaluation Earthquake (SEE) motions to be either the probabilistically-derived mean 1 in 10,000 Annual Exceedance Probability (AEP) ground motions or the deterministic scenario motions at the 84th-percentile level for the Controlling Maximum Earthquake (CME). The scenario motions need not exceed those derived by the probabilistic approach. The CME is defined as ‘the maximum earthquake on a seismic source that is capable of inducing the largest seismic demand on a dam.’ The 2015 Guidelines also require that ‘epistemic uncertainties associated with earthquake sources and ground motion prediction equations should be considered’.

The SEE requirements of the 2015 Guidelines for high PIC dams are similar to the 2000 requirements, but are more onerous in two ways. The less important change is that the 2015 Guidelines specify the 84th-percentile level for the scenario motions, where the percentile level was previously undefined, although the 84th-percentile level was recommended in the Mejia et al. (2001) paper that was often used to interpret the 2000 Guidelines. The effects of this change are limited by the retention of the maximum requirement in terms of the 1 in 10,000 AEP motions. Of more consequence is the new requirement to explicitly consider ‘epistemic uncertainties’, needing consideration of multiple GMPEs and multiple representations of the earthquake sources.

Although ‘epistemic uncertainties’ aren’t defined in the Guidelines, they correspond to one of two items discussed in the description for uncertainty in the Glossary to the Guidelines:
'Uncertainty' – Result of imperfect knowledge concerning the present or future state of a system, event, situation or population under consideration. The level of uncertainty governs the confidence in predictions, inferences or conclusions. In the context of dam safety, uncertainty can be attributed to (i) inherent variability in natural properties and events, and (ii) incomplete knowledge of parameters and the relationships between input and output values.

The first type of uncertainty above is often referred to as 'aleatory', and is accounted for in GMPEs by defining motions in terms of probabilistic distributions (usually log-normal distributions for PGAs or response spectral accelerations, defined in terms of their median values and the standard deviation of the logarithm of the acceleration). The second type of uncertainty is referred to as 'epistemic'.

In this study, the requirements for considering epistemic uncertainties in GMPEs are addressed by the use of GMPE logic trees. Epistemic uncertainties in the fault locations, segmentation, parameters and the possibility of multi-segment ruptures are considered through sensitivity analyses and estimation of deterministic spectra for various fault-rupture scenarios as alternatives to the probabilistic hazard spectra. These two approaches were discussed in the proposal for this study and agreed to in the contract.
2.0 MODELLING OF EARTHQUAKE SOURCES

The starting point for the modelling of the earthquake sources in this study is the 2010 National Seismic Hazard Model (NSHM), as published in Stirling et al. (2012). The NSHM has two seismicity components: a ‘distributed seismicity’ component consisting of a three-dimensional grid of point sources that are not associated with specific faults, derived from the historical seismicity catalogue, and a geologically-based fault source component. The distributed seismicity component is unchanged from the 2010 NSHM. As specified in the proposal, the modelling of the fault sources is largely that of the 2010 NSHM, apart from modification of the representation of the Waimea Fault.

2.1 ACTIVE FAULT EARTHQUAKE SOURCES IN THE VICINITY OF THE WAIMEA DAM SITE

The modelling of the Waimea Fault has been modified from the two-segment representation of Stirling et al. (2012), as used in the 2011 hazard study for the site (Buxton et al., 2011), to a three-segment representation. The three-segment model was originally developed in the course of the 2014 resource consent process (e-mail 3 December 2014 from G. McVerry of GNS Science to M. Foley of Tonkin & Taylor). Slight changes to the original three-segment model of the Waimea Fault have been made in this study, with the dip of all three segments modified from the previous 90° to 70°, consistent with the value given by Johnston (1983), Fraser et al. (2006) and Johnston & Nicol (2013). This in turn has a small effect on the area of the rupture surface, and hence the estimated magnitudes and recurrence intervals. The parameters of the three segments of the Waimea Fault used in this study are listed in Table 2.1, together with those of the two other NSHM active fault earthquake sources most relevant to the Waimea Dam site, namely the Kaniere-Tophouse segment of the Alpine Fault (AlpineK2T) and the Wairau Fault. These parameters correspond to the current NSHM as updated since 2010. These and other nearby active fault sources of the NSHM are shown in Figure 2.1. The table also lists three combined sources. These are considered for generating deterministic spectra for multi-segment rupture scenarios (section 2.5), but not in the probabilistic hazard estimates.

2.2 WAIMEA-FLAXMORE FAULT SYSTEM

The closest known active fault, or fault system, to the Waimea Dam site is the Waimea-Flaxmore Fault System (e.g. Langridge et al. 2016). The Waimea-Flaxmore Fault System has an approximate length of 110-130 km, and extends from near St Arnaud in the southwest (where it intersects the Alpine Fault) to near D’Urville Island in the northeast. At its closest, it passes within about 8 to 9 km northwest from the Waimea Dam site. The Waimea-Flaxmore Fault System encompasses a number of active folds and faults (e.g. Waimea, Eighty-eight, Bishopdale, Flaxmore, Whangamoa faults) within a zone up to several kilometres wide (e.g., Johnston 1982, Rattenbury et al. 1998, Fraser et al. 2006, Johnson and Nicol 2013, Nicol et al. 2014). The multiple fault traces of this zone are modelled by a single through-going fault strand, with three lengthwise segments (Figure 2.1). The Waimea-Flaxmore Fault System spans active traces across 2 or 3 old terrane boundaries within the bedrock of the Waimea-Richmond Ranges. None of these terrane boundaries has a continuously mappable active fault trace along it, and, they are very closely spaced faults across strike. Therefore, it is assumed that collectively, the Waimea-Flaxmore Fault System could be represented by continuous rupture sources that involve multiple faults, or pieces of faults. Faults within the Waimea-Flaxmore Fault system typically have moderate to steep dips to the southeast, and predominantly a reverse sense of displacement with a subordinate, often minor, component of dextral strike-slip (e.g. Litchfield et al. 2014).
The Waimea-Flaxmore Fault System has not ruptured the ground surface and generated a large magnitude earthquake within written historic times. The timing of the most recent known rupture of the fault system, 400 to 1000 years ago, comes from an investigation trench site located about 20-25 km north from its intersection with the Alpine / Wairau Fault (Nicol et al. 2014). This southwestern portion of the fault system is termed WaimeaS (Waimea South) in the current National Seismic Hazard Model maintained by GNS Science. The next section of the Waimea-Flaxmore Fault System to the north is termed WaimeaC (Waimea Central), and it is the closest section to both Nelson City, and the Waimea Dam site. The paleoearthquake investigations of Fraser et al. (2006), south of Nelson city, indicate that this portion of the Waimea-Flaxmore Fault System last ruptured about 6,200 years ago, and has an average recurrence interval of surface fault rupture earthquakes of about 6,000 years (based on the timing of three surface fault rupture earthquakes over the last ~18,000 years which are presumed to be the three most recent ones). Comparatively less is known about the earthquake activity of the northeastern portion of the fault system, termed WaimeaN (Waimea North) but, for several reasons outlined in Johnston and Nicol (2013) and Nicol et al. (2014), its activity is presumed to be less than sections of the fault system further to the southwest that are closer to the higher strain-rate Alpine Fault and Marlborough Fault System.

All three sections of the Waimea-Flaxmore Fault System, as portrayed in the current National Seismic Hazard Model (WaimeaS, WaimeaC, and WaimeaN), are considered capable of generating earthquakes in the order on M 7 with average recurrence intervals of about 6000 years, based on considerations related to fault length and single-event displacement sizes of about 2.5 – 3.2 m of ground surface displacement per event. These values are consistent with those for the three-segment representation of the Waimea Fault developed in December 2014 during the resource consent process, but contrast with the magnitude of 7.4 and average recurrence interval of 9600 years for the Waimea North source in the Buxton et al. (2011) report. The representation and parameters of the Waimea South source are similar to those of Buxton e al. (2011).
Figure 2.1 Characterisation of fault sources in the vicinity of the proposed Waimea Dam in the National Seismic Hazard Model (Stirling et al., 2010), with updating of the segmentation of the Waimea Fault.
Table 2.1  Earthquake parameters for active fault earthquake sources closest to the Waimea Dam site.

<table>
<thead>
<tr>
<th>Active fault earthquake source</th>
<th>Type</th>
<th>Type Index</th>
<th>Length (km)</th>
<th>Dip (°)</th>
<th>Dip dir (°)</th>
<th>Depth (km)</th>
<th>SR (mm/yr)</th>
<th>Mw</th>
<th>SED (m)</th>
<th>RI (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WaimeaS</td>
<td>rs</td>
<td>3</td>
<td>40</td>
<td>70</td>
<td>110</td>
<td>12</td>
<td>0.5</td>
<td>7.1</td>
<td>2.8</td>
<td>5600</td>
</tr>
<tr>
<td>WaimeaC</td>
<td>rs</td>
<td>3</td>
<td>42</td>
<td>70</td>
<td>145</td>
<td>12</td>
<td>0.5</td>
<td>7.1</td>
<td>2.9</td>
<td>5800</td>
</tr>
<tr>
<td>WaimeaN</td>
<td>rs</td>
<td>3</td>
<td>40</td>
<td>70</td>
<td>130</td>
<td>12</td>
<td>0.5</td>
<td>7.1</td>
<td>2.8</td>
<td>5600</td>
</tr>
<tr>
<td>Wairau</td>
<td>ss</td>
<td>3</td>
<td>143</td>
<td>80</td>
<td>160</td>
<td>12</td>
<td>4</td>
<td>7.8</td>
<td>10.0</td>
<td>2500</td>
</tr>
<tr>
<td>AlpineKT</td>
<td>ss</td>
<td>1</td>
<td>194</td>
<td>60</td>
<td>145</td>
<td>12</td>
<td>7</td>
<td>7.7</td>
<td>4.3</td>
<td>620</td>
</tr>
<tr>
<td>WaimeaCS</td>
<td>rs</td>
<td>3</td>
<td>82</td>
<td>70</td>
<td>128</td>
<td>12</td>
<td>-</td>
<td>7.5</td>
<td>5.7</td>
<td>-</td>
</tr>
<tr>
<td>AlpineKT-WaimeaS</td>
<td>sr</td>
<td>1</td>
<td>234</td>
<td>60</td>
<td>110</td>
<td>12</td>
<td>-</td>
<td>7.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>AlpineKT-Wairau H&amp;B scaling</td>
<td>ss</td>
<td>1</td>
<td>337</td>
<td>80</td>
<td>160</td>
<td>12</td>
<td>-</td>
<td>7.9</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Type: rs = predominantly reverse fault with strike-slip component; ss = strike-slip fault; sr = predominantly strike-slip with reverse component.

Type Index: fault source empirical earthquake magnitude code for New Zealand crustal faults, Equations 1 and 3 in Stirling et al. (2012).

Length, Dip and Dip direction are average values calculated from mapped fault traces.

SR: estimates of the late Quaternary slip rate.

SED: single-event displacement, calculated from Equation 5 in Stirling et al. (2012).

RI: recurrence interval, calculated from Equation 4 in Stirling et al. (2012).

2.3  WAIKAU FAULT

The closest, short (<2500 yr) recurrence interval fault to the Waimea Dam site is the Wairau Fault, about 21 to 22 km south-southeast from the site, at its closest. The Wairau Fault is the northeastern section of the Alpine Fault, and extends from the Nelson Lakes area in the west-southwest to offshore Cook Strait in the east-northeast. Like the Waimea-Flaxmore Fault System, the Wairau Fault has not ruptured in a large earthquake within historic times. Paleoearthquake investigations on the Wairau Fault indicate that the fault has a recurrence interval of surface fault rupture earthquakes in the order of 2000 to 3000 years (e.g., Barnes & Pondard 2010, Zachariasen et al. 2006). Single-event surface rupture displacements of up to about 6 m have been documented on the fault, and this, along with its anticipated surface rupture length are consistent with the fault being capable of generating moderate to high magnitude 7 earthquakes. In the National Seismic Hazard Model, the Wairau Fault is modelled as a 143 km long, Mw 7.8 earthquake source with a recurrence interval of approximately 2500 years (Stirling et al. 2012).

2.4  ALPINE FAULT

The Alpine Fault is the longest and has the highest slip-rate of all on-land faults in New Zealand. The extent of the North Westland section of the Alpine Fault (Alpine K2T source) is defined by intersections with major faults: to the southeast by its intersection with the high slip-rate Hope Fault near Lake Kaniere, and to the northeast by its intersection with the Waimea-Flaxmore Fault System near Tophouse (e.g. Stirling et al. 2012, Howarth et al. 2014). In the National Seismic Hazard Model, the North Westland section of the Alpine Fault is modelled as a 194 km long, M 7.7 earthquake source with a recurrence interval of approximately 600 years, and it is called the Alpine Kaniere - Tophouse active fault earthquake source (AlpineK2T in
Figure 2.1) (Stirling et al. 2012). At its closest, the Alpine Kaniere - Tophouse source is about 40 - 45 km southwest from the Waimea Dam site.

2.5 **SENSITIVITY STUDIES FOR FAULT MODELLING**

In characterising active fault earthquake sources for the National Seismic Hazard Model, considerable effort goes into making sure modelled source parameters are consistent with known paleoearthquake data for the active faults those sources represent. However, high-quality paleoearthquake data are not available for all active faults. For example, much more is known about the activity of the southern and central sections of the Waimea-Flaxmore Fault System than the northern section. As a consequence, it is inevitable that assumptions have been made regarding the parameterisation of active fault earthquake sources in the National Seismic Hazard Model. In addition, recent large earthquakes in New Zealand, such as the 2010 Darfield earthquake and the 2016 Kaikoura earthquake, have shown that a specific earthquake can result from the rupture of multiple sections of the same fault, and can also be the result of rupture of multiple faults from differing tectonic provinces and with differing slip rates, and recurrence intervals (e.g. Hamling et al. 2017, Stirling et al. 2017).

With regards to the Waimea Dam site, it is important to understand what, if any, impact potential uncertainties in active fault earthquake source parameterisation may have on the evaluation of earthquake ground motions at the site. In this current study, we explore this topic through a series of four sensitivity scenarios (Figures 2.2 to 2.5). The specific uncertainties that are encompassed by these four scenarios are as follows:

1. The southwestern part of the Waimea-Flaxmore Fault System may be more active than the northeastern part.

2. A large earthquake impacting the Waimea Dam site may be the result of rupture of multiple sections of the same fault.

3. A large earthquake impacting the Waimea Dam site may involve rupture of multiple faults from differing tectonic provinces and with different slip rates and recurrence intervals.

2.5.1 **Scenario 1: reduced recurrence interval for WaimeaS**

In sensitivity scenario 1, the recurrence interval of the WaimeaS active fault earthquake source is arbitrarily reduced by a third (Figure 2.2). This scenario has the WaimeaS source rupturing with a recurrence interval of 4000 years (c.f. ~6000 years), and emulates the possibility that the southwestern part of the Waimea-Flaxmore Fault System is more active than the northeastern part, and has experienced 1 - 2 additional earthquakes over the last ~18,000 years compared to the northeastern part of the fault system (see item 1 above). This scenario is considered as an alternative in a sensitivity analysis for the probabilistic hazard estimates.
The bold lines indicate the Waimea South fault segment, for which the average recurrence interval of rupture was reduced from 5600 to 4000 years for re-estimation of the probabilistic spectra in the first of the sensitivity studies.

2.5.2 Scenario 2: combined rupture of WaimeaS and WaimeaC

Sensitivity scenario 2 involves the combined rupture of the WaimeaS and WaimeaC sources (Figure 2.3). This scenario implies an earthquake with a rupture length of about 82 km, a magnitude of M 7.5 (cf M 7.1 for the two individual segments), and a source to site distance of
8 – 9 km. This scenario provides insight into the potential impact on ground motion evaluation at the Waimea Dam site of uncertainty item 2 above. It is considered in producing 50th- and 84th-percentile deterministic estimates of ground-motions at the proposed dam site, but not in probabilistic hazard estimates.

Figure 2.3 The bold lines indicate the combined Waimea South and Central fault segments, for which 50th- and 84th- percentile deterministic scenario spectra were estimated in the second of the sensitivity studies.
2.5.3 Scenario 3: combined rupture of AlpineK2T and WaimeaS

Sensitivity scenario 3 involves the combined rupture of the Alpine Kaniere-Tophouse active fault earthquake source (AlpineK2T) and the WaimeaS source (Figure 2.4). This scenario implies an earthquake with a rupture length of about 235 km, a magnitude of M 7.8, and a source to site distance of about 12 km. This scenario provides insight into the potential impact on ground motion evaluation at the Waimea Dam site of uncertainty items 1 and 3 above. It is considered to produce deterministic estimates of ground-motions at the proposed dam site, but not in probabilistic hazard estimates. It represents a considerably increased magnitude from the Mw 7.1 for the WaimeaS source on its own at this distance from the dam site.
2.5.4 Scenario 4: combined rupture of AlpineK2T and Wairau

Sensitivity scenario 4 involves the combined rupture of the AlpineK2T source and the Wairau source (Figure 2.5). This scenario implies an earthquake with a rupture length of about 340 km, a magnitude of M 7.9, and a source to site distance of about 22 – 23 km. This scenario provides insight into the potential impact on ground motion evaluation at the Waimea Dam site of uncertainty item 2 above. It is considered to produce deterministic estimates of ground-motions at the proposed dam site, but not in probabilistic hazard estimates.
Figure 2.5 The bold lines indicate the combined Alpine Kaniere-Tophouse and Wairau fault segments, for which 50th- and 84th-percentile deterministic scenario spectra were estimated in the fourth of the sensitivity studies.

See Sections 4.2 and 4.4 of this report for detailed discussion regarding the potential impacts these four sensitivity scenarios have on the evaluation of probabilistic and deterministic scenario estimates of earthquake ground motions at the Waimea Dam site.
3.0 RECOMMENDATION OF GMPES FOR THE WAIMEA DAM STUDY

The contract for the Waimea Dam seismic hazard estimates calls for the use of three GMPEs, the McVerry et al. (2006) model, the Bradley (2013) model, and one of the models from the 2014 NGA-West-2 project (Gregor et al., 2014). The NGA-West-2 project produced five GMPEs, all summarised in the August 2014 issue of Earthquake Spectra (Volume 30, Number 3), namely: Abrahamson, Silva & Kamai (ASK); Boore, Stewart, Seyhan and Atkinson (BSSA); Campbell & Bozorgnia (CB); Chiou & Youngs (CY); and Idriss (2014). The Bradley (2013) model was derived from an earlier but similar version of the CY model (Chiou at al., 2010).

The various GMPEs were compared for several scenarios relevant to the Waimea Dam hazard study, based on the position of the dam relative to known faults and results of the 2011 analysis. These included a magnitude 7 oblique reverse mechanism earthquake at a shortest distance D of the dam from the Waimea Central fault segment of 8.3 km; a magnitude 7.8 oblique-reverse mechanism earthquake at distance D=12 km from the combined Waimea South/Alpine Kaniere-Tophouse sources; and magnitude 5.5 reverse and strike-slip mechanism earthquakes at a distance of 15 km to represent moderate magnitude local earthquakes not associated with known faults. These scenarios were chosen as representative of classes of events, or because they produce stronger motions than similar events at greater distances. For example, the Waimea North and Waimea South fault segments are associated with the same magnitude earthquakes as the Waimea Central segment, but at greater distances; the Waimea South/Alpine Kaniere-Tophouse source is associated with a larger magnitude event than the Waimea South segment at the same distance, or with a similar magnitude at shorter distance than the Wairau and Alpine Kaniere-Tophouse sources.

Observations from these scenario analyses are:

1. The NGA-West 2 GMPEs do not have separate spectra for reverse-oblique events. Predicted spectra for these models for this type of event are the same as for reverse mechanism events.

2. The Idriss spectra for the magnitude 7 and 7.8 scenarios exhibit shoulders in the period range starting at about 2s period (Figures 3.1 and 3.2), which are likely to be unrealistic; additional scenarios show that these shoulders occur for both strike-slip and reverse mechanisms; the incipient appearance of this feature starts at about magnitude 6.5, becoming more pronounced for larger magnitudes.

3. The Bradley and CY model from which it was derived provide very similar spectra (Figures 3.1 and 3.2).

4. The NGA West spectra often lie within quite narrow bands when plotted as a function of period, but exhibit different shapes.

In addition, the southern end of the Hikurangi subduction interface and the underlying slab produce earthquakes that may affect the dam site. Logic trees were also provided to consider subduction slab and interface GMPEs (see Section 3.2).

As part of the project discussions, it was agreed between Tonkin & Taylor and GNS Science that results are to be calculated for the geometric-mean horizontal component, or, for the NGA models, the 50th-percentile orientation, which is close to the geometric mean. The NGA-West 2 GMPEs were formulated in terms of these components. The McVerry et al. (2006) GMPEs contain expressions for both the larger and geometric-mean horizontal components.
3.1 HANGING WALL EFFECTS

The proposed Waimea Dam site is on the hanging wall of the Waimea Fault, i.e., on the side that lies above the dipping fault plane. This affects the strength of ground motions expected as a function of distance from the fault. A site on the hanging-wall side, particularly when it lies over the fault plane, will generally experience stronger motions from rupture of the fault than a site on the foot wall (the opposite side to the hanging wall) at the same shortest distance. This results from the hanging-wall site having a shorter average distance to the fault plane than the foot-wall site and from amplification effects as the wedge of material between the fault plane and surface tapers as the dipping fault approaches the surface. There is no tapering effect on the foot-wall side.

The NGA West-2 GMPEs, apart from the Idriss model, and the Bradley GMPE account for hanging-wall effects either through the choice of distance measure or through explicit hanging-wall factors. For the McVerry et al. (2006) GMPE, hanging-wall effects are accounted for in this study by adding the hanging-wall terms from the Abrahamson et al. (2014) GMPE, which in turn made use of simulation results of Donahue and Abrahamson (2014) and empirical fitting of data.

3.2 WEIGHTINGS OF THE INDIVIDUAL CRUSTAL GMPE MODELS

On the basis of these observations, it is recommended that the Idriss model be omitted because of poor behaviour at longer periods. The Bradley and CY spectra are largely duplicates of each other, so including the CY model together with the Bradley model is essentially including the same model twice. There appears no good reason for preferring any one of the ASK, BSSA or CB model over the other two. It is therefore recommended that they be given equal weighting.

The Expert Elicitation (EE) panel assembled by GNS Science after the Christchurch earthquake recommended a 60:40 weighting of the Bradley and McVerry GMPEs for magnitudes of 5.5 and greater (Gerstenberger et al., 2014). It is recommended that this relative weighting of these two models be retained in this study.

Finally, it is recommended that there be a 50:50 weighting of the combined NGA to the New Zealand models.

This leads to the recommended weights:

ASK 1/6; BSSA 1/6; CB 1/6; Bradley 3/10; and McVerry 2/10.

This selection of crustal GMPEs and weights was agreed to by reviewer Dr Trevor Matuschka (e-mail Trevor Matuschka of Engineering Geology Ltd to Graeme McVerry of GNS Science and Mark Taylor of Tonkin & Taylor Ltd, 27 June 2017) and Tonkin &Taylor (e-mail Mark Taylor to Graeme McVerry, 30 June 2017).
Figure 3.1 Median scenario spectra for rupture of the Waimea Central fault segment. Note the shoulders on the Idriss spectrum (green) at about 2-3s period, and at 4-5s period, and the general similarity of the CY (yellow) and Bradley (dashed brown) spectra.

Figure 3.2 Median scenario spectra for rupture of the combined Waimea south and Alpine K-T fault segments. Note the shoulders on the Idriss spectrum beyond about 2s period, and the general similarity of the CY (yellow) and Bradley (dashed brown) spectra.
3.3 **SUBDUCTION ZONE MODELS**

The selection of subduction zone models is less important for the seismic hazard at the proposed Waimea Dam than selection of the crustal models, given the presence of the nearby surface faults and the relatively large distance between the site and the Hikurangi Subduction Interface dipping under Marlborough from offshore of Cape Campbell. For these hazard calculations, the subduction zone models of Abrahamson et al. (2016), Atkinson and Boore (2003; 2008), McVerry et al. (2006), and Zhao et al. (2006) are selected. These four models were recommended by Van Houtte (2017) for use in New Zealand PSHA. These models are applied with equal weights.
## 4.0 HAZARD CALCULATIONS

Given the changes in 2015 to the New Zealand Dam Safety Guidelines (NZSOLD, 2015) since the previous report was completed in 2011, a full update of the hazard estimates for Waimea dam has been undertaken. The update includes consideration of epistemic uncertainties in both the ground-motion prediction equations (GMPEs) and fault modelling. Epistemic uncertainties are those resulting from insufficient knowledge or simplification in the models, as opposed to random variability. An important change affecting the determination of the Safety Evaluation Earthquake (SEE) motions for this study is that the 2015 Guidelines require deterministic (or ‘scenario’) spectra to be considered at the 84th-percentile level.

To address the uncertainties in ground-motion predictions required by the 2015 NZSOLD Guidelines, the hazard estimates were performed using a GMPE logic tree in the OpenQuake engine, an open source software developed by Global Earthquake Model (GEM) Foundation as a best-practice engine for hazard and risk calculation and modelling (GEM, 2017). The selection of GMPEs used and their weightings are discussed in Section 3.

The brief calls for peak ground accelerations and 5% damped acceleration response spectra to be developed both with and without magnitude weighting (or scaling). In magnitude-weighting for structural applications, response spectrum values for magnitude M are scaled by the Idriss (1985) factor of \((M/7.5)^{1.285}\) for periods between 0s and 0.5s, as used in the New Zealand Standard NZS170.5:2004 (Standards New Zealand, 2004), while the unweighted estimates have no scaling of the expected accelerations. This factor is intended to produce estimates that are equivalent to magnitude 7.5 values in terms of damage-potential. Magnitude-weighting addresses the criticism that uniform-hazard spectra tend to be dominated by contributions from moderate-magnitude earthquakes, and do not reflect the effect of duration in causing structural damage. Duration depends strongly on magnitude. The Idriss (1985) factor was originally developed for assessing liquefaction potential. Idriss references a study by Kennedy et al. (1984) for the US Nuclear Regulatory Commission that shows that the magnitude-weighting factors developed for liquefaction studies are also relevant to the response of ductile structures.

For liquefaction analyses, the Idriss (1985) expression has been replaced by more modern relations with stronger dependence on magnitude.

As discussed in Section 3.0, the results are presented for the geometric mean of the horizontal components. Vertical PGAs and spectra are outside the scope of this study.

### 4.1 OpenQuake Software/PSHA Software

The hazard calculations for this assessment were calculated using the March 2017 Version 2.3 of the OpenQuake Engine. OpenQuake (OQ) is a suite of open-source software developed by Global Earthquake Model (GEM) Foundation to promote consistent use of data and facilitate best practices in seismic hazard and risk calculation (GEM, 2017).

We utilise an updated version of the 2010 National Seismic Hazard Model (NSHM) (Stirling et al. 2012). The most significant change to the fault modelling from the 2010 NSHM model is the updating of the modelling of the Waimea Fault (see Section 2). This is combined with the use of multiple GMPEs rather than the single GMPE (McVerry et al. 2006) used in the earlier report (Buxton et al., 2011) for the dam site.
The OQ implementation of the GMPE logic tree (Section 3.2) is used to produce hazard curves and response spectra, one for each branch of the logic tree. The hazard curves for each of the logic tree branches are combined according to the associated weights to produce a single hazard curve and response spectrum. Spectra for the 16th, 50th and 84th percentiles along with the mean are reported.

In addition to the comprehensive treatment of epistemic uncertainty represented in the GMPE logic trees, the PSH calculations also consider the aleatory variability in ground motions from the GMPEs. All of the GMPEs have published standard deviations, and the PSH calculations consider the variability in predicted ground motions up to the 3-standard deviation level. This is frequently-used practice in PSHA globally.

The OQ software is also used to produce 50th- and 84th-percentile estimates of spectra for several fault-rupture scenarios, including the combined rupture of several fault segments that are treated as independent sources in the probabilistic estimates. These include the same weighted combinations of crustal GMPEs used for the probabilistic calculations, and use the same fault geometries (apart from linking together some fault segments) to ensure consistent calculations of distances and hanging-wall factors with the probabilistic calculations.

4.2 Probabilistic Hazard Spectra

Tables 4.1 and 4.2 respectively list the probabilistic mean and 84-percentile estimates of the 5% damped acceleration response spectra for return periods of 150 years, 500, 2500 and 10,000 years, for the preferred fault source parameters. The results are given both unweighted (UW) and with magnitude-weighting (MW) up to periods of 0.5s. The weighting for magnitude M is $(M/7.5)^{1.285}$, as used in developing the spectra of NZS1170.5:2004. The values are for RotD50 (very similar to the geometric mean) versions of the GMPEs. Hanging wall factors have been incorporated in all the GMPEs.

The mean unweighted spectra are plotted in Figures 4.1 and 4.2, on linear and log-log plots. These figures also show the mean spectra for the case where the average recurrence interval of the southern segment of the Waimea Fault has been reduced from 5600 years to 4000 years, in recognition of the possibility that the fault’s slip rate increases towards the south as it becomes closer to the higher strain rate Wairau and Alpine faults. The effect of this change on the hazard estimates is slight, a maximum of less than 2% at the peak of the 10,000-year spectrum, and generally much less than that.

Figures 4.3 and 4.4 show the probabilistic 50th- and 84th-percentile unweighted spectra for the preferred fault source model for the combination of all the GMPEs, as well as the mean spectra shown in Figures 4.1 and 4.2. The 50th-percentile spectra are very similar to the mean spectra listed in Table 4.1, and are virtually indistinguishable from them in the plots, except at long spectral periods and return periods.

Figures 4.5 and 4.6 compare the unweighted and magnitude-weighted spectra, on linear and log scales. Magnitude-weighting generally has only minor effects on these spectra, with the largest effects at the peaks of the spectra, which are reduced by about amounts ranging from about 15% for the 150-year spectrum down to about 4% for the 10,000-year spectrum.
Table 4.1  Summary of mean estimates of 5% damped unweighted (UW) and magnitude-weighted (MW) acceleration response spectra for Waimea dam for preferred fault source parameters.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>Mean 5% damped acceleration response spectra SA(T) (g)</th>
<th>Return Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>150 years</td>
<td>500 years</td>
</tr>
<tr>
<td></td>
<td>UW</td>
<td>MW</td>
</tr>
<tr>
<td>0</td>
<td>0.15</td>
<td>0.25</td>
</tr>
<tr>
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<td>0.28</td>
</tr>
<tr>
<td>0.1</td>
<td>0.34</td>
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<td>0.26</td>
<td>0.42</td>
</tr>
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<td>0.5</td>
<td>0.21</td>
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<tr>
<td>0.75</td>
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Figure 4.1  Waimea Dam mean 5% damped acceleration response spectra for return periods of 150, 500, 2500 and 10,000 years for preferred fault source model and model with shorter recurrence interval of 4000 years rather than 5600 years for the Waimea South fault source. There is no magnitude-weighting.
### Table 4.2

Summary of 84-percentile estimates of 5% damped unweighted (UW) and magnitude-weighted (MW) acceleration response spectra for Waimea dam for preferred fault source parameters.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>84-percentile 5% damped acceleration response spectra SA(T) (g)</th>
<th>Return Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>150 years</td>
<td>500 years</td>
</tr>
<tr>
<td></td>
<td>UW</td>
<td>MW</td>
</tr>
<tr>
<td>0</td>
<td>0.17</td>
<td>0.14</td>
</tr>
<tr>
<td>0.075</td>
<td>0.32</td>
<td>0.26</td>
</tr>
<tr>
<td>0.1</td>
<td>0.38</td>
<td>0.32</td>
</tr>
<tr>
<td>0.15</td>
<td>0.41</td>
<td>0.34</td>
</tr>
<tr>
<td>0.2</td>
<td>0.42</td>
<td>0.36</td>
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<td>0.25</td>
<td>0.36</td>
<td>0.32</td>
</tr>
<tr>
<td>0.3</td>
<td>0.32</td>
<td>0.29</td>
</tr>
<tr>
<td>0.35</td>
<td>0.29</td>
<td>0.26</td>
</tr>
<tr>
<td>0.4</td>
<td>0.26</td>
<td>0.24</td>
</tr>
<tr>
<td>0.5</td>
<td>0.23</td>
<td>0.21</td>
</tr>
<tr>
<td>0.75</td>
<td></td>
<td>0.17</td>
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<td>3</td>
<td>0.059</td>
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</tbody>
</table>

**Figure 4.2**

Spectra of Figure 4.1 on log-log plot.
Figure 4.3  Mean spectra from Figure 4.1 for the preferred fault source model with the addition of the 50- and 84-percentile spectra for the weighted combination of all the GMPEs.

Figure 4.4  Spectra of Figure 4.3 on log-log plot.
Figure 4.5 Waimea Dam mean spectra and mean magnitude-weighted spectra for return periods of 150, 500, 2500 and 10,000 years for the preferred fault source model.

Figure 4.6 Spectra of Figure 4.5 on log-log plot.

4.3 **Deterministic Scenario Spectra**

For high Potential Impact Classification (PIC) dams, such as the proposed Waimea Dam, the New Zealand Dam Safety Guidelines (NZSOLD, 2015) allow deterministic estimates of
scenario motions as alternatives to the mean 10,000-year spectrum. The deterministic motions are required to be the 84th-percentile motions associated with the SEE earthquake at the 84th-percentile level for the Controlling Maximum Earthquake (CME). The SEE is the earthquake that would result in the most severe ground motion which a dam structure must be able to endure without uncontrolled release of the reservoir. The CME is the earthquake capable of inducing the largest seismic demand on a dam.

Spectra for three multi-segment and one single-segment fault-rupture scenarios have been considered as alternatives to the mean 10,000-year spectrum for the Safety Evaluation Earthquake (SEE) motions (Table 4.3). The scenarios considered were: combined rupture of the central and southern segments of the Waimea Fault in a magnitude 7.5 earthquake at a shortest distance of about 8 km from the dam site; a single-segment rupture of the central segment of the Waimea Fault in a magnitude 7.1 earthquake at a shortest distance of about 8 km; combined rupture of the Waimea South and Alpine sources in a magnitude 7.8 earthquake at a shortest distance of about 12 km from the dam site; and combined rupture of the Wairau and Alpine Faults in a magnitude 8.3 earthquake at about 21 km shortest distance. Table 4.3 also lists the recommended SEE spectrum from the 2011 study (referred to as the “MDE spectrum” in that study), as discussed in Section 4.4.

The mean 50th- and 84th-percentile estimates of these scenario spectra are shown in Figures 4.7 and 4.8, in linear and log plots. These spectra are found by taking the 50th- and 84th-percentile estimates (median and one standard deviation above the median) for each GMPE, and then determining the weighted-average for the 5 GMPEs considered. Also shown are the mean uniform hazard spectra for return periods of 150, 500, 2500 and 10,000 years. None of these spectra are magnitude-weighted. The 84th-percentile scenario spectra range from around the mean 2500-year motions to stronger than the mean 10,000-year motions. The 50th-percentile scenario spectra generally lie between the 500- and 2500-year probabilistic spectra.

Determination of the SEE motions involves evaluation of the mean 10,000-year hazard spectrum and the 84th-percentile scenario spectra. The two strongest 84th-percentile scenario spectra, both involving rupture of the central segment of the Waimea Fault, one scenario with rupture in combination with the south segment and the other for rupture on its own, exceed the mean 10,000-year spectrum, so need not be considered for the SEE motions according to the NZSOLD (2015) Guidelines. The mean of the 84th-percentile spectra for the combined Alpine-Waimea South sources is very close to the mean 10,000-year spectrum, although slightly exceeding it in the period range 0.15s to 1s. Thus, the mean 10,000-year spectrum should be taken as the SEE motions, but the 84th-percentile motions for combined rupture of the Alpine-Waimea Sources in a magnitude 7.8 earthquake at a distance of about 12 km from the dam site can be taken as one approximate realisation of this spectrum.
Table 4.3  Summary of candidates for the SEE motions, namely mean estimates of 5% damped unweighted acceleration response spectra for Waimea dam for a return period of 10,000 years and for the 84th-percentile spectra for three multi-segment and one single-segment fault-rupture scenarios. Also listed is the recommended MDE spectrum of the 2011 study for rock site conditions.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>Mean 5% damped acceleration response spectra SA(T) (g)</th>
<th>Return Period or Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10,000yrs Waimea Central and South 84th-percentile</td>
<td>Waimea Central 84th-percentile</td>
</tr>
<tr>
<td></td>
<td>0.64</td>
<td>0.74</td>
</tr>
<tr>
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<td>1.26</td>
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</tr>
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<td>1.52</td>
<td>1.76</td>
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<td>0.15</td>
<td>1.65</td>
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<tr>
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<td>0.25</td>
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<td>0.35</td>
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</tr>
<tr>
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</tr>
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<td>0.28</td>
</tr>
<tr>
<td>3</td>
<td>0.19</td>
<td>0.17</td>
</tr>
</tbody>
</table>
Figure 4.7  Comparison of mean uniform hazard spectra of Figure 4.1 (solid lines) with the mean estimates (over the 5 GMPEs) of the 50th - and 84th -percentile spectra for three multi-segment rupture scenarios. Figure 4.8 The mean 10,000-year spectrum is very similar to the mean estimate of the 84th-percentile motions for the combined Alpine-Waimea South sources. The two strongest 84th-percentile scenario estimates, for the combined rupture of the central and south segments of the Waimea Fault and for rupture of the central segment of the Waimea Fault on its own, exceed the mean 10,000-year spectrum, so need not be considered for the SEE motions according to the NZSOLD (2015) Guidelines.

Figure 4.8  Spectra of Figure 4.7 on a log-log plot.
4.4 Comparison with 2011 Spectra

The recommended MDE (Maximum Design Earthquake) spectrum in the 2011 study (i.e., SEE spectrum) was based on the envelope of two 84-percentile deterministic scenario spectra produced for that study: 1) for a magnitude $M_w$ 7.0 earthquake at 8 km from the dam site on the Waimea South source of that study, and 2) for a magnitude $M_w$ 7.8 earthquake at 21 km distance on the Wairau Fault. The envelope was approximated by the smoothed magnitude-weighted 5000-year hazard spectrum for rock site conditions (Table 4.3, black dashed curve on Figure 4.9). In contrast, the recommended SEE spectrum in this study is probabilistically-derived rather than scenario-based, corresponding to the mean 10,000-year spectrum for rock site conditions (solid red curve).

The PGA values for the recommended SEE motions are enhanced by about one-third from the magnitude-weighted value of 0.48g recommended in 2011, to 0.62g magnitude-weighted or 0.64g unweighted. In contrast to the increase in PGA values, the recommended SEE spectrum of the current study for all periods of 0.25s or longer falls below the MDE spectrum of the 2011 study, despite being associated with a longer return period of 10,000 years rather than 5000 years.

It appears that most of the change is caused by differences in the seismicity models, as results from the current study that use only the McVerry et al. (2006) GMPE (dotted curves) lie below the 2011 results, for which only the McVerry GMPE was used. The combination of GMPEs has an effect on the shape of the spectra, in that for the current spectra those produced by combining all GMPEs (solid curves) cross over those produced using only the McVerry GMPE.

The uniform hazard spectra of this study are also compared with the spectra from the 2011 study (Buxton et al., 2011) in Figure 4.9. The spectra from the current study (solid curves) produced using the combination of all GMPEs Section 3) are considerably reduced from those of the 2011 study for the same return period (large dashed curves) for all periods of 0.25s and longer.
4.5 Deaggregation of 1 in 10,000 AEP Hazard

Deaggregation of the percentage contributions by magnitude and distance to the exceedance rate of the 10,000-year PGA are provided in Figure 4.10, by magnitude cells of 0.2 units width and distance cells of 20 km width. The main contributions totalling nearly 60% come from magnitude 7.1 earthquakes on the central and southern segments on the Waimea Fault, at shortest distances to the proposed dam site of about 8 km and 12 km respectively. These events have average recurrence intervals of 5800 and 5600 years, respectively (Table 2.1).

Table 4.4 lists the percentage contributions by magnitude, together with the percentage cumulative contributions. Only the cell for magnitude 7.0-7.2 and distance 0-20 km, corresponding to the Waimea central and south segments, produces contributions exceeding 10%. The average magnitude for the contributions to this PGA level is 7.2, boosted from the magnitude of 7.1 associated with the Waimea Fault by small contributions at larger magnitudes from the Alpine Fault (magnitude 7.7 at 32 km distance) and Hikurangi interface sources (magnitudes 8.1 to 8.9 at distances of about 100 to 120 km).

The OQ software amalgamates the contributions of the sources by magnitude and distance cells, and does not provide the contributions of individual faults. However, the contribution of the central segment of the Waimea Fault must be larger than that of the south segment, whose contributions are combined in the cell for magnitude range 7.0-7.2 and distance range 0-20 km distance, because it is at a shorter distance of about 8 km compared to about 12 km from the dam site, and these two sources have the same magnitude of 7.1, and similar average recurrence intervals of rupture of 5800 and 5600 years.
Figure 4.10  Percentage contribution by magnitude and distance to exceedance rate of 1/10,000 AEP PGA. Nearly 60% of the contribution is from the Central and South segments of the Waimea Fault producing magnitude 7.1 earthquakes at shortest distances of about 8 and 12 km, respectively, from the proposed dam site.

Table 4.4  Percentage contribution by magnitude to 1/10,000 AEP PGA.

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<th>Magnitude</th>
<th>% Contribution</th>
<th>Cumulative</th>
</tr>
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<td>0.80</td>
</tr>
<tr>
<td>5.5</td>
<td>1.93</td>
<td>2.73</td>
</tr>
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<td>96.79</td>
</tr>
<tr>
<td>8.9</td>
<td>3.21</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Average magnitude by contribution = 7.2
4.6 AFTERSHOCK MOTIONS

The 2015 NZSOLD Guidelines require consideration of shaking in aftershock motions for high PIC dams, because ‘SEE shaking may lead to cracking, increased seepage and reduced strength. … The information will enable the determination of dam stability following an aftershock.’ The requirements are that ‘For the purposes of dam safety assessments at least one aftershock of one magnitude less than the CME should be anticipated within one day of the SEE.’ The Guidelines also discuss multiple aftershocks in days to months after the mainshock, with a need to consider dam stability over the period until repairs can be completed.

The Guidelines define the CME as ‘The maximum earthquake on a seismic source that is capable of inducing the largest seismic demand on the dam’. However, they do not discuss whether the CME motions exclude those that need not be considered as scenario motions for the SEE motions, because they exceed the mean 10,000-year probabilistic motions. The Guidelines also do not state the percentile level that should be considered for the aftershock motions.

There are several candidates for the CME motions for the proposed Waimea Dam, as it is not clear whether these need be taken as stronger than the SEE motions. For Waimea Dam, it was recommended that the SEE motions be taken as the mean probabilistic 10,000-year motions. The largest contribution to the exceedance rate of the 10,000-year motions is from magnitude 7.1 earthquakes on the central segment of the Waimea Fault, at a shortest distance of about 8 km from the dam site. The 10,000-year spectrum was very similar to the 84th-percentile spectrum for a scenario earthquake involving combined rupture of the Waimea South and Alpine Kaniere-Tophouse fault segments, in a magnitude 7.8 earthquake at a shortest distance of 12 km. Two stronger scenario spectra were not required to be considered in determining scenario candidates for the SEE motions, because they exceed the 10,000-year probabilistic spectrum. The excluded scenarios are for rupture of the central segment of the Waimea Fault (the largest contributor to the probabilistically-determined SEE spectrum), and for combined rupture of the Waimea Central and South segments, in a magnitude 7.5 earthquake at 8 km distance. Although not required to be considered for the SEE spectrum, it is not clear whether stronger of these (for the combined rupture of the two segments) need to be considered as contributing the CME motions.

This leads to three candidates for aftershock motions:

i. A magnitude 6.1 earthquake on the central segment of the Waimea fault at a distance of about 8 km (aftershock of largest contributor to the probabilistically-determined SEE motions, and a disallowed deterministic contender for the SEE motions);

ii. A magnitude 6.5 earthquake on the central segment of the Waimea Fault at a shortest distance of 8 km (aftershock of the disallowed Waimea Central-South deterministic contender for the SEE motions);

iii. A magnitude 6.8 earthquake on the south segment of the Waimea Fault at a distance of about 12 km (aftershock of the combined rupture of the Waimea South and Alpine Kaniere-Tophouse fault segments).

Spectra for these three aftershock scenarios are plotted in Figure 4.11, at the 50th- and 84th-percentile levels, and compared to the uniform hazard spectra for the dam site. The 84th-percentile spectra for the two aftershocks involving the Waimea Central fault segment lie closer to the 10,000-year than to the 2500-year spectrum at short periods. The two associated main
shock spectra exceed the 10,000-year spectrum, so were not required to be considered as SEE spectra. The 84th-percentile spectrum for a magnitude 6.8 aftershock of the combined rupture of Alpine Kaniere-Tophouse and Waimea South fault segments lies close to the 2500-year spectrum, exceeding it at short spectral periods and falling below it for periods of about 0.75s and longer. This spectrum appears to be at a level more appropriate for consideration as an aftershock spectrum, given that it is significantly reduced from the SEE spectrum. This is consistent with the associated main shock spectrum lying very close to the 10,000-year SEE spectrum.

Figure 4.11 Spectra for three candidate aftershock events, plotted at the 84th-percentile (dash-dot curves) and 50th-percentile (small dashes) and compared with the uniform hazard curves (solid curves). The three events are a magnitude 6.5 aftershock following combined rupture of the Waimea Central and South fault segments (WaimeaCS, black curves), a magnitude 6.1 aftershock of rupture of the Waimea Central fault segment on its own (WaimeaC, grey curves), and a magnitude 6.8 aftershock of the combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments (Alpine K2T_WaimeaS, purple curves).
Table 4.5  Three candidate aftershock scenario spectra. Selection of the M6.8 Waimea South and Alpine event as the aftershock is consistent with the associate aftershock spectrum being similar to the 10,00-year SEE spectrum.

<table>
<thead>
<tr>
<th>Period T(s)</th>
<th>84th-percentile aftershock spectra (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M6.1 Waimea Central aftershock</td>
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</tr>
<tr>
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</tr>
<tr>
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<tr>
<td>0.75</td>
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</tr>
<tr>
<td>1</td>
<td>0.27</td>
</tr>
<tr>
<td>1.5</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>0.099</td>
</tr>
<tr>
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<td>0.049</td>
</tr>
</tbody>
</table>
5.0 CONCLUSIONS AND RECOMMENDATIONS

Acceleration response spectra for 5% damping have been estimated for Waimea Dam, updating the earlier study of Buxton et al. (2011) by incorporating an updated seismicity model, including modelling of the Waimea Fault as three rather than two source segments, and by using the weighted combination of five ground-motion prediction equations (GMPEs) rather than the one used in 2011.

- Spectra have been estimated for NZS1170.5 Site Class B Rock site conditions, with an assumed average shear-wave velocity $V_{s30}$ over the top 30 metres of 800 m/s.
- The five GMPEs used are McVerry et al. (2006); Bradley (2013); Abrahamson, Silva & Kamai (ASK, 2014); Boore, Stewart, Seyhan and Atkinson (BSSA, 2014); and Campbell & Bozorgnia (CB, 2014), with weights of ASK 1/6; BSSA 1/6; CB 1/6; Bradley 3/10 and McVerry 2/10.
- Probabilistic mean and 84th-percentile spectra have been estimated for return periods of 150, 500, 2500 and 10,000 years, with and without magnitude-weighting. The values are for RotD50 (very similar to the geometric mean) versions of the GMPEs. Hanging wall factors have been incorporated in all the GMPEs.
- Magnitude-weighting generally has minor effects on the probabilistic hazard spectra for this study.
- The 50-percentile spectra are very similar to the mean spectra, except at long spectral periods and return periods.
- The effect on the hazard estimates of reducing the average recurrence interval of the southern segment of the Waimea Fault from 5600 years to 4000 years is slight, a maximum of less than 2% at the peak of the 10,000-year spectrum.
- Deterministic spectra for various rupture scenarios have also been produced, including considering multi-fault ruptures (combined Waimea Central and South fault segments, combined Waimea South and Alpine Kaniere-Tophouse source, and combined Wairau and Alpine Kaniere-Tophouse source).
- The combined Waimea Central and South rupture scenario is associated with a magnitude of 7.5 for the modelled fault source closest to the dam (at about 8 km distance), rather than Mw 7.1 for rupture of the Waimea Central segment on its own.
- The combined rupture of the Waimea South and Alpine Kaniere-Tophouse fault segments is associated with a magnitude of Mw 7.8 for the event at 12 km from the dam, considerably increase from Mw 7.1 for rupture of the Waimea South source on its own.
- The strongest 84th-percentile scenario estimates, for the combined rupture of the central and south segments of the Waimea Fault, exceed the mean 10,000-year spectrum, so need not be considered for the SEE motions according to the NZSOLD (2015) Guidelines.
- The Safety Evaluation Earthquake (SEE) motions have been recommended as the probabilistically-determined mean 10,000-year spectrum.
- The mean estimate of the 84th-percentile motions for the combined Alpine-Waimea South sources is very similar to the 10,000-year probabilistically-based SEE spectrum.
- For all periods of 0.25s and longer, the probabilistic spectra estimated in the current study are reduced from those of the 2011 study for the same return period, with the change appearing to result mainly from the seismicity model rather than the use of a combinations of GMPEs in place of the single one used in 2011.
• The PGA values for the SEE motions are enhanced by about one-third from the 2011 magnitude-weighted value of 0.48g, to 0.62g magnitude-weighted or 0.64g unweighted.

• In contrast to the PGA values, the recommended SEE spectrum of the current study falls below the MDE spectrum of the 2011 study, despite being associated with a longer return period of 10,000 years rather than 5000 years.

• The main contribution (about 60% of the total) to the exceedance rate of the 10,000-year spectrum is from the central and south segments of the Waimea Fault, modelled as producing magnitude 7.1 earthquakes at distances of 8 km and 12 km, respectively, from the dam site.

• The contribution-averaged magnitude for the 10,000-year peak ground accelerations is 7.2, because of the contributions of larger magnitude sources in addition to those of the Waimea Fault.

• The recommended aftershock spectrum corresponds to the 84th-percentile spectrum for a magnitude 6.8 earthquake at 12 km distance from the dam site, following a magnitude 7.8 main-shock corresponding to a combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments.

6.0 ACKNOWLEDGEMENTS

Dr Robert Langridge and Dr Matt Gerstenberger are thanked for their reviews of this report.
7.0 REFERENCES


### Principal Location

1 Fairway Drive  
Avalon  
PO Box 30368  
Lower Hutt  
New Zealand  
T +64-4-570 1444  
F +64-4-570 4600

### Other Locations

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<th>Town</th>
<th>Region</th>
<th>Contact Details</th>
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</table>
| Dunedin Research Centre         | 764 Cumberland Street            | Dunedin  | New Zealand | T +64-3-477 4050  
                                | Private Bag 1930                 |          |                      |
| Wairakei Research Centre        | 114 Karetoto Road                | Wairakei | New Zealand | T +64-7-374 8211  
                                | Private Bag 2000, Taupo          |          |                      |
| National Isotope Centre         | 30 Gracefield Road               | Lower Hutt | New Zealand | T +64-4-570 1444  
                                | PO Box 31312                     |          |                      |
Dear Russell

Your query regarding briefing Councillors on the seismic risk aspects of this development refers. Firstly in dam engineering terms, I can say that concrete faced rockfill dams of the type proposed for the Lee River site provide very high levels of resilience to seismic loading.

Relative to other dam types, the reservoir water pressure acting on the upstream sloping membrane pushes perpendicular to the dam face in a direction that intersects the dam foundation within the rockfill footprint. This water pressure force adds to the weight of the rockfill embankment, increasing the frictional resistance of the dam to any tendency to slide. In this way, the dam is intrinsically very stable.

Another favourable aspect of this dam type is the drained nature of the rockfill, due to the waterproof membrane separating it from the reservoir. Earth core / granular shoulder embankment dams do not have this fully drained condition, as the upstream shoulder and core are exposed to saturation from the impounded water body. This absence of water within the embankment enables the full strength of the rockfill to be utilised, without any reduction associated with the presence of saturation and water pressure within the aggregate. A simple analogy might be to think of the performance of a road basecourse aggregate in dry versus saturated conditions.

A final favourable characteristic of this dam type is its free draining characteristics. Even in situations where the upstream membrane joints might become damaged and result in extensive leakage, the compacted rockfill can convey this leakage safely without the risk of significant erosion or deterioration that might apply to other embankment dam types. Furthermore, the concrete faced rockfill embankment is constructed with several internal zones that have differing specified particle sizes, such that finer zones are located upstream of coarser zones. This arrangement still provides effective relief of leakage where it is needed, but it also leads to control of the maximum possible rate of leakage in the event of membrane damage.

Behaviour of embankment dams under seismic loading is a function of the ground motion that can occur at the site, along with the associated dynamic response of the embankment to that ground motion.

The design process looks at the behaviour under smaller more frequent earthquake events, as well as the very large extreme events that might plausibly occur. The smaller scale events relate to earthquakes with a probability of occurrence of 1 in 150 years or 0.67% per annum. Under this...
level of shaking the dam is required to remain essentially in serviceable condition. The extreme design event relates to much larger earthquakes with a probability of up to 1 in 10,000 years or 0.01% per annum. Under this level of shaking the dam is required to prevent the sudden uncontrolled release of the impounded reservoir, but it may require extensive repair or even lead to demolition. By way of comparison, conventional dwellings are designed to not collapse in earthquakes with a probability of occurrence of 1 in 500 years or 0.2% per annum. The degree of ground motion is not linearly related to the earthquake probability; i.e. the 10,000 year event does not generate 20 times greater acceleration than the 500 year event. Determination of the actual expected ground motions for the respective design events is the field of seismology. Seismologists use probability methods as introduced above, as well as deterministic methods related to the predicted motion on specific faults. They also use wave propagation and attenuation models to assess the actual site effects caused by faulting some distance from the dam.

A key aspect of design is the determination of the expected degree of deformation that might be experienced in the embankment. That is the resultant displacement of portions of the dam that would be evident immediately after the earthquake. Often the crest deformation is a key consideration. Rockfill embankments can experience crest settlement associated with densification of the rockfill during the earthquake. Slope instability of the embankment shoulder may also be experienced in cases where the shear strength of the rockfill is exceeded. If the embankment deforms beyond a given limit, the membrane joints will rupture and leakage will occur. Significant settlement and/or instability might result in loss of reservoir freeboard and possibly overtopping. These adverse effects are addressed through careful control of rockfill quality and attention to effective compaction, as well as selection of appropriate batter slopes and freeboard allowance.

The recent seismicity review undertaken by GNS has updated the ground motion expectations for the site in light of the current state of knowledge. I will not seek to reproduce these findings, as their draft report and has already been provided. However, I have included the Tonkin and Taylor summary below along with some general observations.

"Changes in site seismicity

The design of the dam to date was based on a site specific hazard assessment prepared by GNS in early 2011 based on the requirements of NZSOLD 2000. New NZSOLD Dam Safety Guidelines (2015) have since been introduced requiring consideration of different scenarios in the evaluation of seismic hazard spectra for the dam design. The understanding of fault mechanisms in New Zealand has also developed due to the recent Canterbury Earthquake Sequence, [and] the Seddon and the Kaikoura earthquakes in the intervening years. T+T on behalf of the Principal has therefore commissioned GNS to review and update the site specific seismic study to be consistent with the requirements of DSG (2015). A final draft report has been completed and will be finalised shortly. The final draft report suggests an increase in the Peak Ground Acceleration (PGA) of the Maximum Design Earthquake for the dam site from 0.48g to between approximately 0.6 to 0.7g. The PGA for the Operating Basis Earthquake (OBE) has also increased, although only slightly.

Implications for design and construction due to the change in seismicity

Quantification (analysis and design) of the seismic changes has not yet been undertaken. The analysis and redesign by T+T is intended to be undertaken at an early stage of the ECI phase with a view to gaining benefit on constructability of and components (e.g. to refine reinforcing arrangements). Whilst the analysis and design has not been undertaken the following changes to the dam may be required to address the increased seismicity:
Changes in the structural steel and concrete dam components such as:

- The parapet wall dimensions and reinforcing;
- The spillway walls dimensions and reinforcing
- The spillway flip bucket dimensions and reinforcing
- The spillway bridges dimensions and reinforcing
- Culvert concrete dimensions and reinforcing
- Concrete starter dam dimensions and reinforcing
- Spillway cut slope stabilisation and support

Other components that will be investigated but may not require physical amendments include depending on the analysis:

- Requirements for additional drainage zones within the dam embankment to accommodate post-earthquake drainage
- Increased seismic induced dam embankment settlements requiring increased freeboard (either by increasing the dam height or the parapet wall)
- Geogrid reinforcing of the dam crest to reduce seismic deformation

I have no further comments on the above thoughts, other than to highlight the focus on the secondary structural elements rather than the key embankment form. This is not unexpected as the increased peak ground acceleration value does not translate directly into embankment deformation outcomes. The energy distribution in the seismic motion as described in the “spectral distribution” is also important, as is the magnitude of the events that represent the duration of shaking or the number of load cycles. Embankment deformation occurs typically at the peak acceleration point of each cycle in the weakest movement direction (i.e. downslope) at the crest, so the adverse effect is sensitive to the dynamic response of the embankment rather than simply the peak ground acceleration, and to the number of significant cycles.

Overall, completion of detailed design will be required to quantify the net effect of this new knowledge, but the result is unlikely to significantly change the seismic resilience or risk exposure associated with this development. There may be some commercial cost implications of course, but relative to the overall development investment involved, the adjustment of such factors as reinforcement content or added local geogrid is not expected be such as to change the commitment to the development.

Yours sincerely

Ian G Walsh CPEng(46343), FIPENZ
Independent Peer Reviewer
Dear Mark,

RE: WAIMEA DAM – SEISMIC HAZARD UPDATES (March 2018)  
PEER REVIEW

1.0 INTRODUCTION

This letter summarises the results of a peer review of the GNS 2017 updated estimates of seismic hazard for the project and subsequent work undertaken by Tonkin and Taylor Ltd (T+T) associated with estimates of seismic hazard for the Waimea Dam.

2.0 GNS 2017 ESTIMATES OF SEISMIC HAZARD

Estimates of seismic hazard for the site in terms of horizontal acceleration spectra were originally provided by GNS in 2009. They were updated in 2011 to account for new knowledge of potential sources of earthquakes using the May 2010 update of GNS’s National Seismic Hazard Model (NHSM). In 2017 GNS were requested to provide further updates to estimates of seismic hazard that included consideration of:

1. Aftershocks
2. Findings from the Kaikoura and Canterbury earthquakes

The NZSOLD Dam Safety Guidelines were updated in 2015. They include specific requirements for seismic hazard studies including consideration of aftershock events when assessing dam safety. For High PIC dams they recommend that “Epistemic uncertainties associated with earthquake sources and ground motion prediction equations should be considered”. The 2017 GNS estimates of seismic hazard have considered epistemic uncertainty though sensitivity studies rather than full logic true analyses. The study incorporated an updated seismicity model, included modelling of the Waimea fault as three rather than two source segments and used the weighted combination of five ground-motion prediction equations. We consider the approach adopted by GNS is satisfactory for the Waimea Dam project and fulfils the intent of the NZSOLD Dam Safety Guidelines with regards to epistemic uncertainty associated with earthquake sources and ground motion prediction equations.
One of the findings from the Kaikoura earthquake was the possibility of ruptures extending along multiple faults. GNS has allowed for this epistemic uncertainty by considering the possibility of rupture along different segments of the Waimea and Alpine Faults.

The 2017 GNS study provided probabilistic estimates of spectra for return periods of 150, 500, 2,500 and 10,000 years and deterministic spectra (84th percentile) for various fault rupture scenarios. The worst case deterministic scenario is a magnitude 7.5 earthquake that could occur at 8km due to combined rupture of the central and southern segments of the Waimea Fault. The ground motion associated with this event exceeds that of the 10,000-year spectrum, and so in accordance with the NZSOLD Dam Safety Guidelines GNS recommend the 10,000-year spectrum is adopted as the Safety Evaluation Earthquake (SEE). We agree with this interpretation. We note that it is very similar to the 84th percentile spectrum associated with combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments (magnitude 7.8 earthquake at 12km from the dam site).

The 2017 GNS study recommended that the design aftershock event be taken as the 84th percentile spectrum associated with a magnitude 6.8 earthquake at 12 km distance from the dam site. This is one magnitude unit less than the maximum earthquake that could occur due to combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments (magnitude 7.8). The 2015 NZSOLD Dam Safety Guidelines recommends for dam safety assessments at least one aftershock of one magnitude less than the Controlling Maximum Earthquake (CME). The CME for the site is a magnitude 7.5 earthquake at 8km due to combined rupture of the central and southern segments of the Waimea Fault. GNS argue that since the SEE is based on the 10,000-year spectrum and is very close to the spectrum associated with combined rupture of the Alpine Kaniere-Tophouse and Waimea South fault segments it is reasonable to base the aftershock spectrum on this event. However, the aftershock spectrum is less than that associated with an aftershock associated with combined rupture of the central and southern segments of the Waimea Fault (i.e. magnitude of 6.5 at 8km). The aftershock spectrum recommended by GNS is up to about 30% lower than an aftershock associated with combined rupture of the central and southern segments of the Waimea Fault. We recommend that the sensitivity of the dam design to the stronger aftershock event associated with combined central and southern segments of the Waimea Fault be considered.

3.0 SEISMIC ANALYSES BY TONKIN AND TAYLOR

Design analyses for the dam have been undertaken by T+T. The seismic analyses required, in addition to estimates of horizontal acceleration spectra, the following:

1. Derivation of vertical response spectra;
2. Selection of ground motions (acceleration time-histories) for dynamic analyses; and
3. Derivation of scaling factors to be applied to the selected acceleration time-histories to be representative of the 2017 spectra.

T+T undertook the above tasks as part of their design work and their recommendations are documented in a letter dated 19 March 2018. We have undertaken review of the recommendations. Initial review comments and a request for clarification of some aspects were emailed to T+T on 18 June 2018. T+T provided further comments and clarification in a letter dated 21 June 2018. This letter included a summary of the dynamic analyses was used in the embankment design.

A summary of our review of the recommendations by T+T follows:
1. **Derivation of vertical response spectra.** Vertical response spectra have been determined using the method of Bozorgnia and Campbell (2004) and assuming a vertical to horizontal ratio (V/H) of 0.9. We consider the method adopted is reasonable for estimating vertical acceleration spectra for design of the dam. It is adopted in the 2016 amendments to NZS1170.5.

2. **Selection of ground motions (acceleration time-histories) for dynamic analyses.** T+T undertook review of the four ground motions selected in the 2011 GNS seismic hazard report (El Centro, Abbar Iran, Izmit Turkey and Tabas Iran). This was because estimates of seismic hazard were updated by GNS in 2017 and since 2011 there are also other ground motion records available. We note that the 2017 estimates of seismic hazard were lower than the 2011. T+T concluded that the four accelerograms selected by GNS in 2011 are still suitable for design. We note that the earthquake source that contributes most to seismic hazard at the site for the SEE is the Waimea-Flaxmore Fault system. Faults in this system are predominantly reverse with a minor component of dextral strike-slip. The Alpine Fault also contributes to seismic hazard, particularly at shorter return periods because ruptures on this fault occur more often than on the Waimea-Flaxmore Fault system. The Alpine fault is a strike-slip fault. Three of the four selected accelerograms are from strike-slip earthquakes. The Tabas Iran accelerogram is referred to by GNS as a thrust earthquake, which is a type of reverse fault. As the Waimea-Flaxmore Fault system is more significant for the SEE we recommend that greater weight be given to the Tabas Iran accelerogram than the others when analysing embankment response for the SEE.

3. **Derivation of scaling factors to be applied to the selected acceleration time-histories to be representative of the 2017 spectra.** The selected time histories require scaling to match the design spectra. There are different methods available for scaling. T+T has adopted the method in NZS1170.5 for scaling the time histories. This method requires an understanding of the fundamental natural period of the structure being analysed. T+T propose to adopt the same scaling factors for horizontal and vertical components. We believe the use of the NZS1170.5 scaling method and the adoption of the same scaling factors for vertical ground motions is reasonable.

Yours faithfully

**ENGINEERING GEOLOGY LTD**

[Signature]

T Matuschka, CPEng
Appendix G: Dam Safety Management System documentation

- Draft Operation, Maintenance and Surveillance manual
- Draft Emergency Action Plan
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**Draft 4**

30 January 2019

**Distribution:**

- Waimea Water e-copy
- Tonkin & Taylor Ltd (FILE) e-copy
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Appendix B: Inspection Checklists
Appendix C: Monitoring Data Sheets
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Appendix F: Staff Proficiency Requirements
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1 General

1.1 Scope and Purpose

The dam safety management for Large Dams such as the Waimea Dam is implemented via the structured framework of the dam safety management system (DSMS) as per the New Zealand Dam Safety Guidelines 2015. The DSMS outlines the requirements for undertaking dam safety activities (such as surveillance and monitoring as outlined in this document), and facilitates dam safety decisions including addressing identified dam safety deficiencies.

This manual outlines the operations, maintenance and surveillance requirements and procedures for the Waimea Dam and forms part of the overall dam safety management system (DSMS) for the Waimea Dam. It covers procedures for correct operation according to design parameters, routine maintenance requirements for asset management and procedures related to surveillance and dam safety. This manual also forms part of the Operational Management Plan (OMP) required by the Resource Consents.

This manual is intended to provide an indication of the envisaged content for the Operation, Maintenance and Surveillance Manual(s) for the completed dam in its operational phase. This document may require updates to match the final physical and organisational arrangements following construction and commissioning.

This particular document does not cover the electromechanical plant and equipment for the outlet works and/or provisional hydro generation. Separate companion documents must be referred to for these parts of the total facility.

Specific details on emergency action procedures are covered in the separate Emergency Action Plan, which should be considered in conjunction with this manual.

The manual is intended to provide guidance to the staff that operate, maintain and carry out regular inspections of Waimea Dam as well as Tasman District Council staff and Consulting Engineers who may be involved in the evaluation of surveillance records and carrying out routine inspections of the dam.

Careful regular surveillance in accordance with this manual is of prime importance to safeguard the integrity of the works as well as to highlight any specific maintenance and operational problems. Effective surveillance is reliant upon the rigorous collection of observation and monitoring data followed by prompt evaluation and any necessary action.

1.2 Basis

This manual was prepared in general accordance and with consideration of the following:

- Tasman District Council Resource consents RM140540, and RM140556 to RM140559.
- The Stage 4 detailed design including aspects covered by safety in design and failure modes effects analysis.
- International practice for a dam of this type, size and PIC (Potential Impact Category).

1.3 Scheme description

The Waimea Dam is a 53 m high concrete face rockfill dam (CFRD) located on the Lee River, Tasman District. The dam’s purpose is water augmentation for irrigation and community water supply to provide drought security to the Waimea Plains. The dam is intended to supplement the Lee River’s natural flows to provide a constant residual flow as well as an irrigation flow.
The dam features an ungated ogee weir controlled chute spillway with a flip bucket for passing flood flows, and valve controlled pipework for the release of environmental, operational flushing flows (the outlet works). A 4 m high parapet wall is located on the crest of the dam to contain the reservoir during large floods and reduce the potential for waves splashing on the crest. The dam crest is access by a bridge over the spillway.

The dam is classified as a high PIC (Potential Impact Classification) dam in accordance with New Zealand Society on Large Dams New Zealand Dam Safety Guidelines (NZSOLD DSG 2015). The dam is was therefore designed to the highest standards applicable in New Zealand for dams at the time.

Further details on the design basis and general arrangements for the dam are presented in the Stage 4 Detailed Design Report and the Drawings. Select Drawings as attached in Appendix A.
2 Key Aspects Relating to Safety

Waimea Dam and appurtenant works, is a High Potential Impact Category (PIC) structure and is required to be operated and maintained in accordance with the NZSOLD Dam Safety Guidelines 2015 and the resource consents.

The main embankment of this dam is constructed from locally sourced rock and gravels. The upstream concrete face prevents excessive seepage (to avoid loss of storage and the potential for unravelling of the downstream face of the dam). The spillway is founded on rock at the left abutment.

Key aspects relating to dam safety are:

- Safe passage of floodwaters, which puts emphasis on spillway operation and the avoidance of any spillway blockage from accumulated debris.
- Performance in a large earthquake, which is essentially dealt with under emergency action procedures (refer separate EAP document).
- Operation of the outlet works intake screens, pipework and valves including maintenance (refer outlet works OMS manual).
- Safe collection and transfer of internal seepage so that general slope instability of the rockfill embankment is avoided, which puts focus on continuing surveillance and emergency procedures.
- The management structure and level of training of those involved in operating, observing and maintaining the facility, and the specialist advice available.
- Road access to the site via the Lee Valley Road and forestry roads.
3 Management Structure and Personnel

3.1 General

Overall management of the facility should be by a senior person who meets the proficiency requirements recommended by the NZSOLD Dam Safety Guidelines (2015) (refer Module 5, Table 2.1), who is fully familiar with the details of the facility, and the contents of this and all related documents.

Management should have access to a specialist consultant or consultants who can give appropriate advice on any proposed changes or repairs to structures, equipment or systems as well as advice in emergency situations or when equipment alerts occur. One or more Dam Safety Engineers should be readily available to provide advice in a timely manner in the event of an unusual occurrence or emergency situation.

Personnel undertaking routine operations and surveillance require a suitable level of education and background training in their areas of input. Staff undertaking routine operations and surveillance activities should meet the proficiency requirements recommended by the NZSOLD Dam Safety Guidelines (2015) (Module 5, Table 2.1 and reproduced in Appendix F).

External contractors would normally be employed for routine maintenance and repairs with their specific instructions being based on this document and other relevant supporting documents, with an appropriate level of management overview.

The management structure for the Waimea Dam is to comprise a Dam Owner and/or Dam Operator, and Dam Safety Consultant. The Waimea Dam is to be operated in the following way:

The Dam Operator is responsible for the day to day management of the structure and will give all operational directives. All instrumentation readings and surveillance reports will be forwarded to them for reporting to the Dam Owner and to the Tasman District Council. Any reading falling into the Alert Level or Trigger Level zone, or observation of any unusual or unsatisfactory behaviour is to be immediately forwarded to the Dam Owner and the Dam Safety Consultant. The Dam Operator will ensure that a suitable replacement is available at any time during which their individual representative is unavailable.

Where the Dam Owner is also the Dam Operator they shall assume the same responsibilities are outlined above. In addition to any other responsibilities, the Dam Owner is responsible for the overall safe operation and management of the Waimea Dam.

The Dam Safety Consultant is responsible for the examination of the instrumentation readings and consideration of any unusual or unsatisfactory behaviour. They will prepare the surveillance reports, undertake dam safety inspections and prepare summary reports as required. The Dam Safety Consultant will ensure that a suitable replacement is available at any time during which their individual representative(s) are unavailable.

3.2 Summary of roles and responsibilities

Table 3.1 below summarises the roles and responsibilities of key parties in the operational phase of the dam.
## Table 3.1  Operational roles and responsibilities

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<thead>
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<th>Role / Responsibilities</th>
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<tr>
<td>Dam Operator</td>
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<td>Dam Safety Consultant</td>
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<td>Provide ongoing dam safety advice and support to the Dam Owner/Operator.</td>
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<tr>
<td>Designer</td>
<td>Tonkin &amp; Taylor Ltd</td>
<td>Design of the dam and associated design documentation.</td>
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<td>Tasman District Council</td>
<td>Resource consent compliance monitoring.</td>
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4 Operations and Maintenance Requirements

4.1 Introduction

This section describes how the dam and its appurtenant structures are to be operated, what items need to be maintained, and the standards of maintenance to retain functional safety. The requirements are subdivided into the various structures, with a brief description of the item and then operational requirements described before maintenance.

The full suite of operation and maintenance requirements will only be available after completion of detailed design for the outlet works, at which time these requirements will be split into a separate manual together with a stand-alone Surveillance Manual.

4.2 Access roads

4.2.1 Description

The access road to the dam include the unsealed forestry roads to the dam, dam site roads, and sealed crest and toe berm roads.

4.2.2 Operation

Use of the access roads for operation shall be in accordance with the Waimea Dam access traffic management plan. All vehicles travelling along the shared forestry road need to be 4WD vehicles equipped with the appropriate radios and be familiar with the operations of the forestry operator.

4.2.3 Maintenance

The unsealed site access roads should be maintained by grading and application of new aggregate when rutting or pot-holing is evident. The sealed site access roads (including the dam crest road) should be maintained by patching, edge-break and pothole repairs.

4.3 Reservoir

4.3.1 Description

At the Normal Top Water Level (NTWL) of 197.2 m RL the reservoir has a storage capacity of approximately 13 Mm³. The design storage elevation curve is presented in Figure 4.1 below. Further details on the reservoir can be found in the Stage 4 detailed design report (T+T, 2018).
4.3.2 Operation

Operation of the reservoir shall be in accordance with the separate reservoir operating plan.

4.3.3 Maintenance

Reservoir maintenance substantially comprises regular removal of debris and remedying any significant abutment instability adjacent to the embankment. All debris that impairs free discharge at the spillway and intakes should be removed as soon as evident.

Any sign of abutment instability arising from wave action undercutting slopes, intense local rainfall, or earthquake, which might lead to blockage of any dam component, should immediately be assessed and remedial action taken as necessary. Any other instability or erosion of the reservoir rim not directly affecting the abutments should be noted and photographed and then discussed with an experienced dam engineer.

4.4 Debris boom

4.4.1 Description

The Debris Boom shall be a TUFFBOOM waterway barrier and includes debris screens and a mooring buoy with anchor and shoreline anchor connection chains. The anchor connection chains shall be attached to anchor blocks on the shoreline. Further details on the debris boom can be found in the Stage 4 detailed design report (T+T, 2018).

4.4.2 Operation

The debris boom has no operational requirements other than surveillance, which is discussed in Section 5.
4.4.3 Maintenance

During periods of significant flood it is possible that debris (in particular logs and other forestry debris) will become mobilised and float down the reservoir. The debris boom will trap the majority of this debris and this will have to be removed regularly to prevent excessive build up and stress forming on the boom.

The anchor locations, boom linkages and the individual booms will need to be inspected for signs of damage 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.

4.5 Dam embankment

4.5.1 Description

The embankment is a Concrete Faced Rockfill Dam (CFRD) with a maximum dam height of 53 m and a crest length of 220 m. Further details on the embankment can be found in the Stage 4 Design Report (T+T, 2018).

4.5.2 Operation

The embankment dam is a static structure and has no operational requirements other than surveillance, which is discussed in Section 5.

4.5.3 Maintenance

Components which may require maintenance from time to time include exposed upstream faces of the embankment, the crest parapet wall, the crest and other access roads, the downstream face of the dam, and the drainage outlet structures.

Maintaining the condition of the upstream concrete face of the dam is critical to dam performance and the prevention of seepage. During periods of draw down the concrete face should be inspected for cracks due to shrinkage of the concrete and uneven settlement of the underlying embankment. Any major cracking or spalling which causes significant seepage should be repaired as soon as possible.

The downstream face of the dam should be inspected for any signs of seepage, movement or other disturbance.

The drainage outlet structures include the v-notch weir, foundation drains and toe drains. From time to time these may become blocked due to debris accumulation or damaged due to weathering. Once observed debris shall be cleared and any damage remedied to ensure correct operation.

4.6 Spillway

4.6.1 Description

The spillway is comprised of a 40 m wide uncontrolled ogee weir contracting to a 20 m wide chute and terminating in a 20 m radius flip bucket. Further details on the spillway can be found in the Stage 4 detailed design report (T+T, 2018).

Access bridges cross the flip bucket and the crest ogee weir.
4.6.2 Operation

Operation of the spillway is automatic when the reservoir rises above the NTWL (due to it being an uncontrolled oggee crest), provided that the spillway system is in operating order. No manual or mechanical intervention is required for operation of the spillway weir, chute or flip bucket.

Under design operation, the spillway underdrains should not be producing flowrates in excess of 10 l/sec.

4.6.3 Maintenance

The concrete structures of the spillway weir, chute and flip bucket shall be inspected for cracking, spalling or uneven settlement. Any displacement, spalling or cracking that is likely to affect performance shall be repaired as soon as possible.

The drainage system under the spillway needs regular inspection and periodic clearance and/or jetting/flushing of any accumulated sediment. The transverse drains under the spillway daylight above the concrete lining of the chute walls to enable flushing. End caps on these drains should be properly secured and maintained to prevent ingress of debris from the surface.

The downstream toe of the flip bucket should be inspected for damage and any sign of erosion or undercutting. Any significant damage to the concrete liner protecting the downstream toe of the flip bucket should be repaired to prevent erosion or undercutting of the flip bucket foundations.

The flip bucket should be regularly inspected and cleared of any accumulated debris or rocks. Under low flow conditions rocks in the bottom of the flip bucket may circulate in the flow and abrade and erode the concrete lining. Further, the flip bucket incorporates a low level drain to allow water to drain from the bucket in times of low or no flow. This drain should be kept clear of any blockages and flushed if necessary.

The plunge pool should be monitored for ongoing erosion and any accumulated gravel in the river or plunge pool should be removed or regraded as required. Ongoing erosion of the plunge pool is expected in the longer term. Undermining of the downstream toe slab should be monitored and assessed to decide if preventative works are required.

Cut slope batters and stability works shall be inspected routinely to ensure that the slope stability and performance is not impaired.

All exposed structural steel should be inspected for corrosion. Areas of corrosion or damaged paint should be repaired to prevent failure and shortened service life.

4.7 Bridges

4.7.1 Description

There are two bridges across the spillway channel to enable access to the dam:

- Upper bridge to the dam crest from the crest access road. This bridge is to provide access to the crest of the dam, instrumentation, and the intakes.
- Lower bridge to the toe access road. This bridge is to provide access to the outlet works and provisional future power station.

The bridge type is a steel beam substructure with composite concrete bridge deck.

Upstand kerbs are provided to prevent vehicles from falling off the bridge. These kerbs are 300 mm wide by 300 mm high and include cast in 50 mm diameter PVC pipe drain holes at 500 centres. The pipes will require regular clearance of silt and debris.
Side mounted handrails (CSP Pacific Nu-Guard PVB or equivalent) with galvanised steel barriers (CSP Pacific Bridge Flexi-Rail W-beam barrier or equivalent) are provided on the bridges.

Galvanised steel crash barriers (CSP Pacific Highway Flexi-Rail W-beam barrier or equivalent) are also provided at the bridge approaches. The barriers flare out at the terminations to a standard curved trailing terminal installation, except for the true right abutment of the upper bridge where the crash barriers continue to the crest ramp wall (upstream) and extend along the entire length of the dam crest.

4.7.2 Operation

The bridges over the spillways have no specific operational requirements, other than limits on their load carrying capacity.

The design vehicle for the bridges is a 6 wheel, 3 axle, 11 m long truck with an 8.2 tonne design axle. The bridges are also considered to have adequate capacity to carry a single HN (maximum legal weight limit vehicle) vehicle on any given span of the bridge. Should heavy vehicles be required to access the dam, sufficient temporary propping will be required and this will also require specific design.

4.7.3 Maintenance

Maintenance of the bridges should generally be in accordance with the Transit New Zealand 2001 Bridge Inspection and Maintenance Manual SP/M/016. Jacking points are provided to facilitate major maintenance of the bridges which will require specialist input to prepare the required procedures (typically by the Contractor undertaking the maintenance work).

The bridges are comprised of a steel beam sub structure with composite concrete deck. Attention should be paid to:

- Any cracking or damage to the concrete deck.
- Ensuring the expansion joint cover plates remain in place and in good condition.
- Failure of protective coatings and corrosion damage, including at cross brace and fastener locations.
- Bearings are maintained in good condition and free of accumulation of debris. Bearings will likely require replacement during the design life of the bridge.
- Loose or defective fastenings.

The drain holes in the upstand kerbs will require regular clearance of silt and debris.

4.8 Outlet Facility

4.8.1 Intake Structure

4.8.1.1 Description

The proposed intake structure includes an intake screen, a bellmouth entry, a screen outlet bend and associated supporting structures. There are two sets of intakes; an upper intake and a lower intake. Further details on the intake structure can be found in the Mechanical Design Report.

4.8.1.2 Operation

The intakes are designed so that the level they are set at can be lowered if necessary by removal of sections of the inclined intake pipework. It is not envisioned that this will be required for normal operation.
Each of the intakes includes a pressure sensor/transducer on the conduit pipeline to monitor the maximum pressure differential across the intake screens. This is to monitor screen blockage to reduce the risk of screen collapse.

A programmable logic controller is to be used to compare the pressure transducer measurement to a transmitter monitoring the reservoir level, in order to calculate the pressure across each screen at the prevailing pipe discharge. If the pressure differential exceeds a set differential at that pipe discharge an alarm is to activate and the flow through that intake stopped until the cause of the screen blockage is rectified.

4.8.1.3 Maintenance

Regular screen cleaning is required for correct operation of the intake structure. Screen cleaning is to be a manual process and should only be undertaken with a safety management plan and when it is safe to do so.

Cleaning and maintenance of the intake screens will be required at regular intervals over the operating life of the structure. The design includes a winch and rail system to enable a diver to attach the winch cable to the intake screen structure and a winch on the dam crest to haul the intake screen structure up to the crest via rails fastened to the concrete face.

Temporary works such pads should be considered by the operator when using mobile cranes on the crest to remove or access the intakes, pipework or any other materials/equipment. The man access ladders and crest barrier may need to be temporarily removed to enable a mobile crane to set up adjacent to the intake pipes.

This is not intended to be a common occurrence, in most instances cleaning and other maintenance performed on the screens will be undertaken by divers on an annual basis, as part of the IDSR.

For operator safety and to allow for easier removal of debris off the screen, the flow through the screen shall be stopped by closing the downstream valves while the screen is being cleaned. If the screen is in an accessible depth of water minor debris may be removed by an appropriately trained diver. If the screen is significantly blocked the screen may be unbolted and removed by winching it to the dam crest and then it can be cleaned in a readily accessible area.

4.8.2 Inclined pipework

4.8.2.1 Description

The inclined intake pipework on the upstream face of the dam transfers the water from the intake screens to the large radius bends at the upstream end of the through dam conduits. Further details on the inclined pipework are presented in the Stage 4 Detailed Design report (T+T, 2018).

The arrangement consists of two pipe supports welded to each length of pipework. Each pipe support has two guides attached to the underside of the supports which run on rails that are anchored to the face of the dam. The guides provide alignment when the pipework is being installed or removed and also provide pipework restraint.

To accommodate movement of the concrete face as a result of deformation of the dam, the inclined pipework incorporates non thrust type dismantling joint couplings between each length of pipe. These couplings accommodate regular expansion and contraction pipework movement, and angular deflection between the two adjoining pipe lengths. The guide rail tracks are to be installed with gaps between each length of rail, to allow for thermal expansion and contraction, and any movement of the concrete face as a result of deformation of the dam face.
4.8.2.2 Operation
The intake pipework is designed so that sections can be removed, by use of divers and winching from the dam crest, to adjust the level the intakes are set at but it is envisaged that this will rarely be required. Due to the complex nature of the joint arrangements it may be that the pipework will actually be required to be floated to the reservoir surface by divers instead of sliding.

4.8.2.3 Maintenance
The intake pipework should be periodically inspected for damage and corrosion and repaired as necessary. This exercise may require divers and could be carried out at the same time as screen cleaning activities. Should the reservoir level be drawn down and expose sections of the pipework, these should be inspected at such times.

The rail system and fixings on the dam face should be periodically inspected and maintained to ensure:
- Fasteners have not loosened
- Rails are at the correct alignment
- Galvanic separation elements are in place
- Any corrosion is maintained to acceptably low levels.

It may be that a cathodic protection system (either an active system or use of zinc anodes) will be required as a retrofit to reduce the rate of corrosion.

4.8.3 Conduit pipework

4.8.3.1 Description
The inclined intake pipework is encased in a mass concrete block at the base of the upstream face of the dam and then enters the conduits under the dam where an isolation valve is provided. The conduit pipework runs from the isolation valve at the upstream end to the downstream toe of the dam where it discharges through the fixed cone valves.

The pipework features a number of couplings to accommodate thermal expansion and for pipe/valve installation/dismantling.

4.8.3.2 Operation
Operation is covered by the valve operation (refer Section 4.7.4).

4.8.3.3 Maintenance
The pipework should be periodically inspected for damage to epoxy coatings and evidence of corrosion and remedied in accordance with manufacturer/supplier requirements. Particular attention should be paid to corrosion occurring around contacts with pedestals.

Couplings should be inspected for leaks and remedied in accordance with manufacturer/supplier requirements.

Low friction pads on each of the concrete pedestals should be positioned and maintained in accordance with manufacturer/supplier requirements.

It is proposed that a mobile lifting frame is to be used to for maintenance purposes (refer Section 4.7.5). Pipework may be transported to the downstream end of the concrete conduits using this lifting frame and removed from the conduits using an external mobile crane. Given the intended lengths of pipework (approximately 24m) requested by the Contractor; it is possible that sections of
A summary of the valves on each pipeline of the outlet facility, generally listed in downstream order, is provided below:

- DN1000 penstock primary isolation double eccentric butterfly valve – electric actuation.
- DN80 bypass pipe isolating gate valve – manual actuation.
- DN80 primary bypass gate valve – electric actuation.
- DN150 anti-vacuum air valve downstream of primary isolation valve – automatic actuation.
- DN50 air release valve upstream of the fixed cone valve – automatic actuation.
- DN850 fixed cone valve (FCV) – electric actuation.
- DN300 fixed cone valve (FCV) – electric actuation.

Further details of the proposed valves for the outlet facility are provided in the Stage 4 Detailed Design Report (T+T, 2018).

4.8.4.2 Operation

All valves shall be operated in accordance with manufacturer/supplier recommendations, and to the operation procedures and requirements for the dam (refer separate reservoir management and outlet works operation manual). [to be prepared by Waimea Water]

The penstock isolation valves at the upstream end are electrically actuated (as well as manually) and will be remotely operated from the control room for the dam. Normally the isolation valves will be fully open and will only be closed to isolate the conduit pipework for maintenance or emergency purposes. A small diameter bypass pipe and valves are also provided to assist with the operation and maintenance of the isolation valves.

The FCDVs are electrically actuated (as well as manually) and will be remotely operated from the control room for the dam. These FCDVs will be operated on a daily basis to regulate and mix the flows released into the Lee River. The upper intake is expected to be used more frequently than the lower intake. The fixed cone valves are set at an elevation such that they can freely discharge and operate up to the 10 year ARI design flood tailwater level.

The FCDVs should not be operated when partially submerged, to avoid damage, unless an emergency situation exists.

The FCDVs shall be operated to maintain velocities in accordance with manufacturer/supplier recommendations and to maintain the velocities in the upstream pipework to the limits specified above.

Access to the fixed cone valves is via ladder/stairs/platforms from the berm at the toe of the dam. Access to the isolation valves is along the length of the conduit under the dam. Procedures for inspection and manual operation of the valves will need to account for confined spaces requirements and operation of the ventilation equipment.
4.8.4.3 Maintenance

All valves shall be maintained in accordance with manufacturer/supplier recommendations. Notwithstanding manufacturer/supplier recommendations, all valves should be regularly “exercised” to ensure that they remain in good working order.

Removable joints and flanges are provided so that valves can be removed if required for maintenance or replacement. A removable spool piece is provided in the rare event that the primary isolation valve needs to be removed.

It is proposed that a mobile lifting frame is to be used to for maintenance purposes (refer Section 4.7.5). Valves may be transported to the downstream end of the concrete conduits using this lifting frame and removed from the conduits using an external mobile crane.

4.8.5 Moveable lifting frame

A mobile lifting frame is to be used to lift and move sections of pipework, primary isolation valves and other miscellaneous equipment to the downstream end of the conduits. At the downstream end of the conduits the equipment can be lifted up and out of the conduit end chamber with a mobile crane. Tools and equipment shall be lowered using the small tool hoist provided for this purpose.

The mobile lifting frame wheels run on the invert of the conduit, either side of the pipework support saddles and the overhead manual lifting hoist is located close to the ceiling of the conduit to maximise the lifting height. The mobile lifting frame would normally not be kept in the conduit, as it would restrict access down the conduit and the damp environment will increase the potential for corrosion. The frame will need to be kept in a suitable covered storage area such as the control building, and when required, it would need to be lowered into the downstream end of the conduits using a mobile crane.
5 Surveillance

5.1 General

The following sections detail the operational phase monitoring and surveillance requirements for the Waimea Dam and are subject to review on a regular basis.

The objective of the routine surveillance is to maintain a complete record of the dams’ behaviour and detect, as early as possible, any signs of potentially adverse behaviour so that causes can be assessed, corrective action taken, or in the extreme, emergency action procedures can be implemented. Surveillance also offers the ability to check the validity of design assumptions against observed behaviour.

The observers or management must maintain a continuous plot of data and ready availability of records so that trends can be detected and evaluated. They must also react promptly and in accordance with instructions where an alert level reading is obtained. The alert levels assigned have a margin of comfort applied and do not, in themselves, represent a dangerous condition if the alert level is just exceeded. They do, however, require evaluation if the alert reading is verified as not being an incorrect reading. A copy of the alert criteria levels has been provided in Appendix E.

As outlined in the NZSOLD Guidelines 2015, the use of instruments to monitor the performance of a dam should be considered an aid rather than a replacement for visual observation.

The most of the dam safety instruments collect data in real time and transmit this to the off-site control room to enable remote monitoring. This system enables automated raise of alarms or other unusual behaviour and is essential for the Waimea Dam given it is a remotely operated site (i.e. no operation staff are located on site full time).

Redundant surveillance systems have been installed to allow cross checking of results or backup if one instrument fails. Manually read instrumentation is also provided as a backup to the electronic instruments.

Frequent routine surveillance is required to enable an up to date understanding of the performance of the Waimea Dam and its appurtenant structures. This routine surveillance comprises of weekly visual inspections and monthly monitoring reviews as outlined below.

Further to the routine surveillance, dam performance reviews are also required to provide a more thorough assessment of the observed dam performance. Four types of dam safety review should be undertaken as per NZSOLD and general industry guidelines:

- Intermediate dam safety reviews (IDSR).
- Comprehensive Dam Safety Reviews (CDSR).
- Special Inspections and Special Dam Safety reviews (SDSR) (e.g. after unusual events).
- Inspection and testing of appurtenant structures and gate and valves.

Inspections are undertaken as part of the dam safety reviews.

Specific equipment inspection requirements are not covered in this document and are set out in the equipment manuals.
5.2 **Inspections**

5.2.1 **General**

Inspections of the dam consist of routine and special inspections. Routine inspections shall be undertaken at least weekly. Weekly inspection and surveillance requirements are set out in Section 5.2.2 below. Inspections should follow a set route to ensure a consistent approach to monitoring.

Further to these requirements, other inspections should also be undertaken in accordance with the NZSOLD Dam Safety Guidelines 2015 and the design basis for the Waimea Dam:

- Intermediate dam safety review (IDSR) inspections.
- Diver inspections.
- Comprehensive dam safety review (CDSR) inspections.
- Inspections after unusual events.

5.2.2 **Weekly visual inspections**

This section sets out detailed requirements for routine monitoring. The frequency and scope of this monitoring may be subject to change following annual and/or five yearly inspections and occurrence of large flood events and as a longer duration performance database is obtained. Any changes should preferably be approved by the original designers, or by a suitably experienced dam engineer.

Forms which may be used for surveillance and monitoring records are provided in Appendix B and C.

Visual inspections undertaken by the Dam Owner and reported to the Dam Safety Consultant should be made on a routine basis normally in conjunction with reading monitoring points. However, personnel visiting the facility at any time should be made familiar with monitoring requirements and be required to check in passing for any signs of potential adverse behaviour. Special inspections are required after unusual events as discussed in Section 5.2.5.

Visual inspections should be based on a regular defined “route march” and recorded on a suitable form or electronically with backing up within 24 hours.

Generally all exposed surfaces in the close vicinity of the dam, particularly those below reservoir level, should be inspected to check for any signs of cracking, slumping, new wet patches, springs, corrosion, settlement and the like, or basically any significant change from the normal condition. Any obvious deterioration of any structure must also be noted.

Weekly visual inspections (i.e. undertaken at least three times during each calendar month with the interval between successive monitoring not exceeding two weeks) should as a minimum include the following:

**Access**

- Condition of access roads and tracks including slope stability.

**Rainfall and lake level**

- Rainfall and lake level readings should recorded using the staff gauge and rain gauge.

**Debris boom**

- Floating debris accumulating on the debris boom should be noted, so that it may be removed before it becomes a potential threat to spillway operation. It is possible for water logged debris to pass underneath the debris boom and it should be removed as soon possible.
Concrete face
- Inspections of the upstream embankment face should be made when reservoir levels are low.

Embankment
- Inspect dam crest and downstream face for any signs of movement, rainfall erosion and/or seepage and/or discoloured water.
- Inspect abutments for any sign of seepage or movement.
- Inspect seepage areas for any changes or muddy water.
- Inspect downstream face of embankment, abutment areas, conduit and service spillway interfaces for emerging seepage, slumping, instability or signs of cracking or deformation.

Parapet wall and crest ramp
- Inspect walls any signs of movement, concrete deterioration, spalling or unusual cracking.

Spillway and bridges
- Inspect the ogee weir for any sign of damage or debris.
- Inspect chute and flip bucket and plunge pool for any signs of movement or debris.
- Inspect overflow relief outlets for evidence of flow.
- Inspect bridges for signs of damage or deterioration.

Flow monitoring weirs
- Inspect weir structures for debris and algae, and clean as required to preserve accuracy of flow rates derived from water level recorder data.
- Where automated readings are not available, drain outflow rates shall be manually measured using bucket and stopwatch (Discrepancies between automated and manual readings may be an indication of debris or organic growth)
- Observe and describe flow clarity in relative terms.

Outlet works
- Inspect the fixed cone dispersion valves and supporting concrete structures for any sign of deterioration.

Fish pass (not dam safety)
- Inspect condition of wet well, grouted rock channel and splitter box.

5.2.3 IDSR inspections

Inspections of the dam are required as part of the annual IDSR to confirm satisfactory behaviour or identify deficiencies by a thorough visual examination of the dam and review of monitoring data. Annual inspections should be undertaken by an experienced dams engineer in conjunction with an annual deformation survey. If possible, the inspections should be carried out when the reservoir is at a high level and the water clarity is good.

IDSR inspections shall be undertaken by the Dam Safety Consultant and the Dam Operator, and reported to the Dam Owner. ISDR inspections shall be as per the weekly inspections and include the following additional visual inspections:

Intake and upstream face
• No specific requirements for annual inspection. Diver inspections every two years (i.e. every second IDSR).
• After a maximum of three years of operation, an inspection of the condition of upstream face of the dam and spillway weir should be undertaken. This may require qualified divers if the reservoir level remains high up to this time. If a diver inspection is required, it should be undertaken in accordance with Section 5.5.

Outlet works
• In accordance with the conduit entry procedures, inspect the conduit pipelines and isolation valves.
• Inspect the fixed cone dispersion valves and supporting concrete structures for any sign of deterioration.
• Observe equipment testing that contributes to dam safety.

Mechanical/electrical
• Inspect all mechanical and electrical equipment that is essential from a dam safety perspective for deterioration, damage and wear.

Reservoir
• Inspect the reservoir margins and shoreline for slope instability including identified landslips (in accordance with Condition 92 (e) of the resource consents).

Upstream & downstream
• Inspect for changes in human activity or natural environment which may have an impact on dam safety operations.

5.2.4 CDSR inspections
CDSR inspections shall be undertaken by the CSDR team and include the IDRS inspection requirements. Additional inspection criteria will be determined by the CSDR team as part of their initial review of the dam documentation.

5.2.5 Special inspections
These inspections are on an as-needs basis. They might occur as a result of a sudden and significant change in seepage or deformation or some other unusual behaviour.

As an indication, inspections should be made after significant earthquakes (noticeably felt at the site) and during and following significant floods. In the early years of operation in particular, the spillway performance should be inspected and monitored during floods.

Unusual events include the following:
• Prolonged extreme winds.
• Large floods.
• Extreme rainfall at the dam site.
• Earthquakes that are sufficient to be felt locally and result in recorded shaking of 0.17g or greater as recorded by the dam foundation seismograph.
• Sudden and unexpected deterioration of any structure or surface, including any slip or landslid impacting on the bypass, reservoir, dam or spillways.
As soon as possible after an earthquake detected by the seismograph(s) on site and exceeding 0.17g, local observers should undertake a close visual inspection of the whole dam facility, including reservoir slopes, and monitor all instrumentation.

Where there are signs of significant wind or rain damage, any alert levels are exceeded or there are signs of possible adverse behaviour from earthquake, the Dam Safety Consultant should be informed and requested to advise on the matter. The Dam Safety Consultant may need to visit the site to provide suitable advice.

Occasional inspections of the upstream embankment face may be made when reservoir levels are low.

5.2.6 Diver inspections

Underwater inspections shall be undertaken by qualified divers at least every two years and as part of the two yearly inspections. The underwater inspection should include the intake screens, inclined pipework, and concrete face and plinth of the dam.

Underwater inspection is best carried out when the reservoir is at a low level, the water clarity is good, and the lake water temperature is moderate. Diver time at the dam intake is restricted given the reservoir depth and site altitude.

The Dam Operator is required to close the outlet works for the duration of diver inspections for safety reasons. Given the environmental flow release requirements set out in the resource consents, any temporary closure of the outlet works shall be undertaken in consultation with Tasman District Council.

5.3 Instrumentation

5.3.1 Description

The dam safety instrumentation consists of the following instruments as shown on the Drawings:

- Two reservoir water level loggers to measure water level from the IDF peak water level of 202.53 m RL down to the minimum operating level of 166.5 m RL and a barometric pressure logger.
- Manual staff gauges at the spillway ogee and on the upstream face of the dam.
- One rain gauge at the dam.
- An embankment seepage collection system at the toe of the dam consisting of a geomembrane faced rockfill bund and perforated HDPE pipe collector drains.
- Spillway underdrains.
- Four seepage measurement weirs with water level loggers for toe seepage and spillway underdrains.
- Settlement pins on the bridges, spillway chute wall and parapet wall.
- Survey constellation pillars to enable settlement survey.
- One profilometer buried under the crest at the top of the Zone 3B material to measure longitudinal embankment settlement (including access points at either end for measurement).
- Three settlement plate instruments located along the crest.
- Four insertion flowmeters in the outlet pipework upstream of the fixed cone discharge valves.
- Two pressure sensors on each outlet pipeline near the upstream isolation valve.
- Valve position indicators for the isolation and FCD valves.
• Security cameras on the dam crest and toe berm.

The dam safety and operational instruments that are required for real time monitoring and operation of the scheme are connected to the onsite telemetry system, and the communications system (i.e. from the on site control building to the external operations room). This enables remote operation and monitoring.

Additional operational instrumentation for the fish pass consists of a full bore flowmeter, pressure sensors, wet well water level meter and pump on/off switches.

5.3.2 Reservoir instruments

5.3.2.1 Water level

Remote monitoring and recording of the reservoir water level (and therefore operation of the service spillway) is provided by two independent reservoir water level probes and loggers located within metal pipe sleeves fastened on the concrete face either side of the intakes down to the minimum operating level of 166.5 m RL. An additional water level logger is also provided for additional monitoring of spillway operation at the ogee crest.

The electronic water level probes are backed up by staff gauges at the spillway and on the upstream face down the right abutment to enable manual reading of water level should the electronic instruments be out of service.

5.3.2.2 Temperature recorder

TBC by others. Not a dam safety instrument.

5.3.2.3 Dissolved oxygen recorder

TBC by others. Not a dam safety instrument.

5.3.3 Rain gauge

A rain gauge is provided at the dam crest to enable interpretation of seepage results during and following large rainfall events, and early warning of potentially large river floods. The rain gauge records precipitation in 15 min increments to enable total volume and rainfall intensity to be measured.

The rain gauge consists of an electronic tipping bucket type rain gauge with a data logger unit and is connected to the on site telemetry system for transmission to the remote control room off-site. The unit includes backup storage of recorded data as a backup to the on site telemetry system.

5.3.4 Seepage collection measurement

5.3.4.1 Embankment seepage

Embankment seepage monitoring is provided to enable early detection of changes in seepage that might indicate unusual performance of the concrete face, plinth and or rockfill. This facility is expected to be especially important following earthquake events.

Seepage through the dam embankment and foundation is intercepted by a geomembrane lined rockfill bund (seepage collection bund) at the toe of the dam. Perforated HDPE collector pipes to drain the collected seepage into two measurement weirs either side of the diversion culvert.

The embankment seepage flow measurement weirs feature a reinforced concrete stilling flume with a 90 deg angle V notch weir plate at the end of the flume. Water levels are measured automatically
by the water level logger (located in a vertical PVC pipe) located in the flume upstream of the weir, and converted to flow by use of a calibrated V notch weir equation.

Manual check measurements of the weir flows are required by use of a bucket and stopwatch.

These weirs are accessed from the outlet works steel access platforms. Access to the weirs shall be strictly from the top of the outlet chamber via the ladders either side of the FCDV’s. The FCDV area is isolated with barriers on the lower landing to prevent access to the monitoring weirs via this landing (i.e. to stop operational staff walking in front of the FCDV’s which may automatically start to operate).

In addition to the seepage monitoring weirs, an observation well (standpipe) is located either side of the conduit to enable measurement of the water surface profile during first filling. These standpipes can be manually dipped from the toby box at the berm surface level.

5.3.4.2 Spillway underdrains

Spillway underdrains are provided to enable monitoring of the potential uplift pressures under the spillway chute due to water flowing into joints and/or groundwater seepage. These drains have capacities significantly larger than the anticipated design flows and are primarily included to enable identification of potential spillway joint defects.

The perforated HDPE drains are located underneath the end of the ogee weir, under each transverse joint in the chute and under the chute walls on both sides. The underdrainage features two separate lines (true left and true right) with non-perforated collector pipes which extend beyond the end of the chute (at the flip bucket interface) to discharge to a reinforced concrete twin flume chamber with 90 deg angle V notch weir plates at the end of each flume. The twin flumes are located below the road on the true left abutment of the toe access berm as shown on the Drawings.

Subject to actual drain flows following commissioning, the water levels are measured automatically by the water level logger (located in a vertical PVC pipe) located in the flume upstream of the weir, and converted to flow by use of a calibrated V notch weir equation. The weir plate may require adjustment to suit the actual drain flows following commissioning. The drain flows are connected to the site telemetry system for transmission to the remote control room. Manual check measurements of the weir flows are required by use of a bucket and stopwatch.

Access to the weirs is from the toe access berm down the true left abutment.

5.3.4.3 Flipbucket underdrain

An underdrain is provided beneath the flipbucket to enable monitoring of the potential uplift pressures due to groundwater seepage. Given the expected low and intermittent flows, outflows at the wingwall structure are measured manually measurements by use of a bucket and stopwatch.

5.3.4.4 Spillway overflow drains

The spillway underdrain system features overflow/relief drains in the highly unlikely event that the underdrain system blocks potentially resulting in high uplift pressures. These drains are set such that they would flow when the spillway floor slab uplift pressures exceed 2.5 m. The outlet wingwall structures are located to enable visual inspection to see if the overflows are operating and hence flagging that further action may need to be taken. Manual measurements can be taken at the outfalls by use of a bucket and stopwatch.
5.3.5 Settlement instrumentation

5.3.5.1 General

Settlement instruments are provided at the Waimea Dam to enable ongoing monitoring for deformation/movement of the embankment and appurtenant structures. Deformation survey requirements area outlined in Section 5.5.

5.3.5.2 Survey pillar constellation

Survey pillars are provided to enable accurate levelling survey of the dam at the settlement pins. Details of the survey pillars and survey procedures are covered in the Waimea Dam survey specification attached in Appendix E (TBC).

5.3.5.3 Settlement pins

Settlement pins are located on the bridges, top of the parapet wall and top of the true right spillway chute wall. These enable terrestrial survey and ongoing deformation assessment of these structures.

Special access arrangements apply for the top of the parapet wall and the spillway chute wall. Harness attachment points are provided for safe access to the spillway chute wall.

5.3.5.4 Settlement Plates

Three metal settlement plates are situated on the dam crest. These instruments consist of a 600 mm by 600 mm HDG MS plate founded on the top of the Zone 3B rockfill with a steel measurement rod in a PVC tube housing that extends to the dam crest. The level of the top of the steel rod is surveyed as part of the overall dam settlement survey to determine the level of the buried Zone 3B rockfill.

5.3.5.5 Profilometer

A profilometer is buried under the crest at the top of the Zone 3B material. This instrument enables measurement of embankment settlement over the length of the dam crest. The instruments are accessed from trafficable reinforced concrete chambers (with DI lids) at each end of the crest and read with a portable unit provided for this purpose.

Specific procedures for reading and maintaining this instrument are covered in the profilometer operation and maintenance manual attached in Appendix G (TBC).

5.3.6 Outlet works instrumentation

5.3.6.1 Outlet pipework flowmeters

Four insertion flowmeters are located on the outlet pipework upstream of the fixed cone discharge valves (FCDV’s). These flowmeters enable automatic real time measurement and recording of discharge flows from each line. The flowmeters are connected to the on site telemetry system and transmitted off site to the remote control room.

Operation and maintenance of these instruments is covered in that attached suppliers O&M manual (refer Appendix G) (TBC).

5.3.6.2 Valve position indicators

Valve position indicators are included on the isolation valves and FCDVs to enable confirmation of the valve positions and secondary estimation of discharge flows. The instruments are read manually from the measurement plate fastened behind the indicator.
5.3.6.3 Pressure sensors

Two pressure sensors are located on each outlet pipeline near the upstream isolation valve to monitor for potential screen blockage.

Operation and maintenance of these instruments is covered in that attached suppliers O&M manual (refer Appendix G) (TBC).

5.3.7 Security cameras

Three pole mounted security cameras are provided at the Waimea Dam for the purposes of real time monitoring and security. The cameras are located on the dam crest at the true right and true left abutments, and at the toe of the dam on the true left abutment by the flip bucket. The cameras are powered by the dam site power network.

The dam crest cameras enable real time monitoring of the reservoir (esp. the spillway and debris boom areas). The dam toe camera enable monitoring of the spillway discharge and outlet works.

The cameras are intended to take continuous footage which is transmitted off site for further processing and storage. Limited on site storage at each camera is also provided for backup (TBC).

The specific camera technical details and operation and maintenance of these instruments is covered in that attached suppliers O&M manual (refer Appendix G) (TBC).

5.3.8 Seismographs

The purpose of the instruments is to facilitate an appropriate and proportional level of response to an earthquake event. Very large earthquakes as identified by the seismographs may require activation of emergency action procedures as covered in the separate operational phase EAP document.

There are two seismograph sensors, one at the dam crest (housed in the winching chamber) and one at the toe of the dam (within the control building or on the adjacent rock slope). The seismographs are powered by the dam site power network with the data logger housed within the on site control room. The seismograph sensors are automatically triggered by large earthquakes and the recorded measurements transmitted off site to the remote control room.

Automatic alarms are set for different levels of shaking (refer Section 5.6).

The specific technical details and operation and maintenance of these instruments is covered in that attached suppliers O&M manual (refer Appendix G) (TBC).

5.4 Routine monitoring requirements

Routine monitoring of the dam safety instrumentation consists of real time monitoring of the automated instruments as described above. Routine review of the collected data and reporting shall be undertaken on a weekly basis, generally including the visual inspections undertaken in Section 5.2. Reservoir levels and daily site rainfalls are to be maintained and recorded as part of the dam monitoring dataset.

We recommend that the monitoring frequencies set out in this manual be reviewed as part of the dam’s first CDSR.

All data should be recorded in a suitable electronic database that is backed up. A checklist is supplied in Appendix C for recording manual measurements and inspection notes.
5.5 Deformation surveys

Deformation surveys of the embankment dam should be undertaken annually, preceding or in conjunction with annual inspections, except as may be appropriate after an unusual event. It is preferable to undertake the surveys at a consistent time of year with a high reservoir level.

The deformation surveys shall be undertaken to the requirements of the Waimea Dam Survey Specification (TBC).

Metal settlement pins are located on the bridges (located on the outside kerbs), and on the top of the parapet wall and the true right spillway chute wall to enable survey and ongoing deformation assessment of these structures. Harness anchor poitsn are provided for safe access to these pins. Survey of these instruments shall be undertaken in accordance with the Waimea Dam survey specification (TBC).

In addition, settlement plates have been installed on the top surface of the Zone 3B rockfill with a steel rod extending to the dam crest inside a PVC housing. The top point of this steel rod should be surveyed.

5.6 Alert criteria

Alert criteria for the instrumentation are specified in Table 5.1 below. These criteria should be reviewed regularly and as part of the IDSR (see Section 6.2).

**Table 5.1 Dam instrumentation alert levels**

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Unusual behaviour alert</th>
<th>Design limit alert</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spillway underdrains</td>
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<td>TBC</td>
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<tr>
<td>Embankment seepage drains (per drain)</td>
<td>30 l/sec (TBC)</td>
<td>100 l/sec</td>
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<td>Seismographs</td>
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<td></td>
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<tr>
<td>Foundation</td>
<td>0.17g</td>
<td>0.64g</td>
</tr>
<tr>
<td>Crest</td>
<td>0.58g</td>
<td>1.90g</td>
</tr>
<tr>
<td>Reservoir water level loggers</td>
<td>199.14 m RL (base of parapet wall)</td>
<td>202.53 m RL (IDF)</td>
</tr>
</tbody>
</table>

5.7 Reservoir water quality sampling

Condition 106 of the resource consent requires monitoring of the reservoir water quality at or near the deepest point in the reservoir. This includes monthly manual water sampling (e.g. from a boat in the reservoir) and laboratory testing for a range of parameters. Condition 106 also requires continuous measurement and recording (hourly logged values) of reservoir temperature (at 8 levels) and dissolved oxygen (at three levels continuously from November to April inclusive).

The dissolved oxygen and temperature probes are connected to the on site telemetry system and real time data monitoring is possible from these instruments. The logged data is transmitted off-site to the remote control room for storage.

Specific operational procedures in provided for boat access to the reservoir and taking of water quality samples. *Operational procedures to be prepared by Waimea Water following commissioning of the dam*
6 Dam safety reviews

6.1 General

Periodic dam safety reviews are required for the Waimea Dam in accordance with the resource consent and the NZSOLD Dam Safety Guidelines 2015. Dam safety reviews shall be undertaken by suitable qualified and experienced dam safety engineers with specialist inputs from others where required.

The associated visual inspections shall follow a set route to ensure a consistent approach. Inspection requirements are covered in Section 5.2.

Dam safety reviews should be undertaken at a suitable time in each calendar year, and in accordance with the requirements of the NZSOLD Guidelines 2015. Table 6.1 below summarises the types of inspection and the required inspection frequencies.

<table>
<thead>
<tr>
<th>Review type</th>
<th>Interval between successive reviews (range)</th>
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<tr>
<td>Intermediate dam safety reviews (IDSR)</td>
<td>9 months – 1 year 3 months (annually)</td>
</tr>
<tr>
<td>Comprehensive dam safety review (CDSR)</td>
<td>4 years 6 months – 5 years 6 months (five yearly)</td>
</tr>
<tr>
<td>Special inspections/ Special dam safety reviews (SDSR)</td>
<td>As confirmed with the Dam Safety Consultant</td>
</tr>
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</table>

6.2 Intermediate dam safety reviews (IDSR)

Intermediate dam safety reviews shall be undertaken annually in accordance with the NZSOLD Guidelines 2015 requirements for High PIC dams. Refer to Section 6.1 for general requirements.

The intermediate dam safety reviews (IDSR) should include:

- A review of the year’s surveillance data trends including deformation survey.
- A visual check on the components of the facility including the reservoir in the vicinity of the dam and the identified landslide areas within the reservoir margin.
- Diver inspection of the intake screens and pipework.
- Visual condition assessment of the outlet works.
- Discussion with operations staff on operation and maintenance and any issues of potential significance.
- An evaluation of the dam’s performance since the last IDSR.
- Preparation of a report that identifies dam safety issues, changes to monitoring or visual inspection frequencies, or additional items to be monitored. The report should be in accordance with the latest version of the NZSOLD Guidelines.

6.3 Two yearly reviews

The two yearly IDSR’s shall be carried out as per the annual IDSR but shall also include the following:

Deformation survey:

- Undertake a deformation survey to the requirements of the Waimea Dam survey specification attached in Appendix E.

Underwater inspection:
• Undertake diver inspection of intake, concrete face and plinth areas in accordance with Section 6.1.1 above.

6.4 **Comprehensive Dam Safety Reviews (CDSR)**

CDSR’s should be undertaken every 5 years in accordance with the NZSOLD Dam Safety Guidelines 2015. Refer to Section 6.1 for general requirements.

The Comprehensive Dam Safety Review (CDSR) is more comprehensive and targeted at confirming the safety of the main dam, including confirmation of design and construction standards in the light of current technical knowledge. Otherwise, it would embody the elements of the annual inspection.

The bulk of the inspection work and reporting would be by a senior dams engineer experienced in the CFRD design and construction, and knowledgeable of current technology. A senior engineering geologist would be desirable to have on the inspection team to provide advice on updated geological knowledge and any changes in seismic hazard.

The CDSR should include an inspection of the condition of upstream face of the dam and spillway weir. This may necessitate the use of qualified divers if the reservoir level is high at this time. If a diver inspection is required, it should be carried out when the lake water temperature is moderate and the water clarity is good.

In addition to making physical inspections and reviewing all surveillance data, the CDSR team would also examine all relevant records related to design and construction (in each team member’s field of expertise). On completion of all review work, the CDSR team should produce a combined report, focussing on safety of the main embankment dam. If any area of uncertainty is identified, or similarly the team identifies any aspect possibly needing upgrading because of changed knowledge (e.g. hydrology and floods), recommendations for resolving the issues should be made. The issues raised in previous annual and five yearly reports should be given special attention.

6.5 **Special Dam Safety Reviews (SDSR)**

Special dam safety reviews (SDSR) are undertaken on an as-needs basis. Refer to Section 6.1 for general requirements.

SDSR’s are typically required following unusual behaviour (such as a significant change in seepage or deformation). Otherwise SDSR’s should be made after significant earthquakes and after significant floods. The Dam Owner/Operator may however request a special dam safety review at any time.

The scope of a SDSR is typical confirmed to suit the specific requirements that have triggered the review, noting the requirements for IDSRR and CDSR (as outlined above).

Earthquake magnitudes that give recorded shaking (as recorded by the seismograph) at the dam foundation of greater than 0.17g (i.e. OBE) trigger a requirement for an inspection by a representative of the Dam Safety Consultant. There may also be instances where the Dam Owner requires an inspection and review by the Dam Safety Consultant for earthquakes that have a magnitude less than the OBE values.

The need for special inspection/review should be confirmed with the Dam Safety Consultant and undertaken as set out in the current revision of the Waimea Dam operational phase Emergency Action Plan. These inspections/reviews shall follow the procedures in the Emergency Action Plan or the monthly observation requirements above.
7 Applicability

This report has been prepared for the exclusive use of our client Waimea Water, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Authorised for Tonkin & Taylor Ltd by:

..........................................................
Mark Foley
PROJECT DIRECTOR

Prepared by:
• Dominic Fletcher (CPEng) WATER RESOURCES AND DAMS ENGINEER

Reviewed by:
• Mark Taylor (CPEng) PROJECT MANAGER
• John Grimston (CPEng) SENIOR DAMS ENGINEER
• David Bouma (CPEng) SENIOR DAMS ENGINEER
Appendix A: Select drawings

Draft 4

30 January 2019
Appendix B: Inspection Checklists

Draft 4

30 January 2019
Appendix G: Equipment manuals

- TBC

Draft 4
30 January 2019
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<th>Reviewed by</th>
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Distribution:

- Waimea Water: e-copy
- Tonkin & Taylor Ltd (FILE): e-copy

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14 August 2018
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<td>Dam designer (Tonkin &amp; Taylor Ltd)</td>
<td>2</td>
<td>Mark Foley Project Director</td>
<td>021 927 330</td>
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SPECIAL NOTE

This EAP outlines the procedures and processes for the Dam Owner/Operator to follow in the event of a threat to the safety of the Waimea Dam. Multi-agency response is necessary should external parties be at risk and the fundamental role of the Dam Owner/Operator is to notify the New Zealand Police and Nelson Tasman Civil Defence Emergency Management Group.

If dam failure is considered to be imminent, immediately ring 111 and report the incident to the New Zealand Police.

The available warning time to the nearest downstream residents from the start of dam failure is in the order of 5 - 10 mins before the flood wave arrives (approx. 30 - 40 mins until flood peak). Therefore it is essential that immediate notification is given to the New Zealand Police to enable evacuation of the potentially affected areas downstream.

The highest risk to life and buildings in the unlikely event of a failure of the Waimea Dam is the area to the east and north of Brightwater.

This EAP will be provided to Waimea Water as a word document for updating and formatting of the supplied content to suit Waimea Water’s requirements. Input into this EAP by the supporting emergency services is essential to enable compatible processes. Updates to this EAP following testing may be necessary.
1 Purpose of the EAP

This Emergency Action Plan (EAP) is for the operational phase of the Waimea Dam, post commissioning (as distinct from the construction phase which is covered separately). The EAP has been prepared generally in accordance with the New Zealand Society on Large Dam (NZSOLD) Dam Safety Guidelines 2015.

This draft is intended to provide an indication of the envisaged content for the operative EAP for the completed dam. Details will need to be completed and particular descriptions in this document will need to be modified and, sections added or deleted, to match the final physical and organisational arrangements following construction.

This EAP will be provided to Waimea Water as a word document for updating and formatting of the supplied content to suit Waimea Water’s requirements. Input into this EAP by the supporting emergency services is essential to enable compatible processes. Updates to this EAP following testing may be necessary.

This EAP describes the processes for:

a  Identification and assessment of potential dam safety threats which may threaten the integrity of the dam and require action.

b  Procedures for declaring a dam safety emergency and classifying which level of emergency response is required.

c  Procedures with specific actions during dam safety emergencies only, to avoid or otherwise reduce the potential for dam failure, and in the event of a dam failure to prevent or reduce the potential for loss of life and/or property damage downstream. Specific actions are assigned to each of the responsible parties for the classified level of emergency response.

d  Communications protocols to provide timely warnings in a systematic way to the appropriate emergency management agencies for their implementation. In case of an emergency affecting the integrity of the dam, procedures for initiating warning of endangered downstream populations are specified, consisting essentially of notification of local emergency services.

The responsibilities and actions for each organisation are outlined in this document. It is intended that each organisation will keep this EAP readily available to assist staff in rapid decision making if and when necessary.

Detailed public warning and evacuation procedures are the responsibility of the New Zealand Police. They may call upon the Civil Defence authorities for aid. Information is included in this document to permit these agencies to develop effective warning and evacuation procedures.

In the event of a dam safety emergency being declared that endangers the integrity of the dam and has the potential to affect external parties (e.g. downstream property and/or life) (i.e. a potential emergency or imminent failure level event), the Dam Owner must notify the appropriate contacts for the New Zealand Police, Nelson Tasman Civil Defence Emergency Management Group (CDEM) and regional authorities. Imminent failure level events shall be notified immediately.

Special Note

If an imminent failure level event is declared, immediately ring 111 and report the incident to the New Zealand Police.

The available warning time to the nearest downstream residents from the start of dam failure is in the order of 5 - 10 mins before the flood wave arrives (approx. 30 - 40 mins until flood peak). Therefore should an imminent failure event be declared, it is essential that immediate notification
is given to the New Zealand Police to enable evacuation of the potentially affected areas downstream.

The highest risk to life and buildings in the unlikely event of a failure of the Waimea Dam is the area to the east and north of Brightwater.

EAP are ‘living’ documents that require regular review and use. This EAP is to be updated by the Dam Owner regularly, with formal written notification of any amendments being circulated to each holder of a controlled copy. Annual exercising/testing of the EAP is strongly recommended.

This draft EAP has been prepared under the assumption that the hydropower add on is not a component of the final dam arrangement. If a hydropower component is constructed then the EAP will require updating as appropriate.

The Contractor will be required to prepare an EAP for the construction of the dam. Dam commissioning procedures are covered separately in the commissioning plan.

The EAP does not cover:

- Communications with insurers, news media or dam storage water user/customers in the event of a dam safety emergency being declared.
- Emergencies external to the dam site area or sabotage, bomb threat, riot, severe storms, fires (including forest fire).
- Oil and hazardous substance spills.
- Personal accidents, drowning and major accidents.
- Fish and wildlife losses.
- Response to criminal actions.
- Power supply emergencies.
- Dam safety incidents such as large floods or changes in dam performance that do not endanger the integrity of the dam or downstream persons and property. These incidents are covered separately in the surveillance manual.

Emergencies not specifically identified in this EAP shall be handled by the Dam Owner using procedures outlined in the EAP which are appropriate to the potential damage to the dam and to the threat to life, property and water supply posed by the emergency.
2  Summary of EAP Responsibilities

2.1  Dam Owner

The Dam Owner has responsibility to operate the dam in a manner that is considered to meet sound engineering and professional standards, to meet all relevant legislative guidelines or requirements and in accordance with procedures set out in the NZSOLD Guidelines\(^1\).

From an emergency planning perspective the Dam Owner is responsible for taking ownership of the EAP including:

a  Providing advice in the preparation of this EAP.
b  Complying with the detail of this EAP.
c  Ensuring that all the staff involved in the operation of the dam are familiar with this EAP, and the obligations in it.
d  Ensuring that suitably trained and authorised staff are available to competently assess potential dam safety threats and declare and classify dam safety emergencies. The staff must be familiar with this EAP. Authorised staff must discharge the responsibilities of the Dam Owner over the emergency event duration until termination and documentation procedures are completed.
e  Suitably trained and authorised staff are deemed to be those that can:
   - Recognise potential dam safety threats and dam safety emergency situations as listed in this plan, and understand their possible effects on the integrity and safety of the dam.
   - Understand that the example potential dam safety threats in Section 3 are not an exhaustive list of every possible condition that could arise, and that judgement must be judiciously applied when assessing situations.
   - Acknowledge the importance of providing early notification to the New Zealand Police, Civil Defence and potentially affected parties downstream, of potential dam safety threats and/or dam safety emergencies at the dam site.
   - Are authorised to notify the relevant parties, declare a dam safety emergency and enact the procedures of this EAP.
   - Accurately monitor, record and report on reservoir levels in relation to reservoir staff gauge and dam crest.
   - Accurately complete the Notification Report as shown in Appendix C.
   - Readily access the emergency contact numbers required to notify the New Zealand Police and Civil Defence (see contact list in Appendix D).
   - Operate communication equipment used to convey emergency messages (e.g. cellphone, email, satellite phone, radio).
   - Correctly interpret and manage the implementation of the preventative actions set out in Section 3.4 of this plan.
   - Liaise with the Dam Safety Consultant where specialist advice is required. The acquisition of such advice must not delay the notification of potential dam safety threats and/or declaration of a dam safety emergency.
   - Safely supervise any of the operational tasks that may be necessary to remedy dam safety threats.

\(^1\) New Zealand Dam Safety Guidelines, 2015 - NZSOLD
f Having facilities and procedures in place to give warnings to New Zealand Police and Civil Defence in the event of dam safety emergency situations or potential dam safety threats that may arise at the dam site.

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

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

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

2.2 New Zealand Police

The New Zealand Police are responsible for maintaining law and order during an emergency. The New Zealand Police are often required to accept initial responsibility for coordination of an emergency response, followed by transfer of this role to the appropriate lead agency (once confirmed). In the instance of a dam safety emergency being declared at the Waimea Dam, the New Zealand Police are the first party to be notified under this EAP.

Specific nominated responsibilities of the New Zealand Police in relation to dam safety emergencies at the Waimea Dam include:

a Providing advice in the preparation of this EAP.

b Include consideration of this EAP with other Police plans and procedures in the region.

c Having systems in place to receive notifications of potential dam safety threats and providing timely notification of the Dam Owner.

d Having systems in place to receive notifications of dam safety emergencies to enable early implementation of Police procedures.

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

f Establishing and maintaining a notification system for warning downstream residents, as well as Fire and Ambulance Services, in the event of a dam safety emergency at the dam site.

2.3 Civil Defence

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

The responsible civil defence group is the Nelson Tasman Civil Defence Emergency Management Group (Nelson Tasman CDEM).

2.4 Fire and Emergency New Zealand and St John Ambulance

Both Fire and Emergency New Zealand and St John Ambulance may be notified of potential dam safety threats by members of the public and/or by the New Zealand Police of a dam safety emergency.

Fire and Emergency New Zealand and St John Ambulance develop and maintain their own specific procedures relating to emergency situations including potential dam safety threats and dam safety emergencies. This EAP outlines the linkages to such plans and procedures, noting that coordination and integration of the relevant aspects of each organisation’s plans is necessary.
2.5 Dam Safety Consultant

The Dam Safety Consultant for the Waimea Dam shall be available to provide dam safety advice where requested by the Dam Owner on potential dam safety threats and in the event a dam safety emergency is declared.

The Dam Safety Consultant will be confirmed following commissioning. Typically, the dam designer (Tonkin & Taylor Ltd) is retained as the Dam Safety Consultant.

In the unlikely event that the Dam Safety Consultant is not contactable, the Dam Owner should seek specialist advice from other suitably qualified dam safety engineering persons/organisations.
3 EAP Response Process

3.1 Process

The response process for potential dam safety threats is described in this section. The response process generally involves identification of a dam safety threat, assessment, declaration of a dam safety emergency and classification (where appropriate), notification to the relevant parties (as appropriate), actions, and termination of the event and documentation of the event and response.

Once a potential dam safety threat is identified, the threat shall be assessed and where appropriate a dam safety emergency declared. Declared dam safety emergencies are classified as follows (as per the NZSOLD Guidelines 2015):

- Internal event – Only impacts on the Dam Owner and the response can be managed internally.
- Potential emergency – Has the potential to affect external parties and the Police, CDEM, emergency services, and local and regional authorities should be notified of the situation.
- Imminent failure – An event that will affect external parties is underway. A dam failure has either occurred, is occurring or is obviously about to occur. The Police, CDEM, emergency services, and local and regional authorities should be immediately notified of the situation.

Figure 3.1 below outlines the assessment and response procedures to be followed.
Note 1: An incident is defined as an occurrence that requires a response from one or more agencies, but is not an emergency.

Note 2: An emergency is defined as a situation that poses an immediate risk to life, health, property, or the environment and requires a coordinated response.

Figure 3.1 Process chart for management of a potential dam safety threat and dam safety emergencies (from NZSOLD New Zealand Dam Safety Guidelines 2015).
3.2 Identification, evaluation and classification procedures

3.2.1 General

The Waimea Dam does not require permanent staff to be located at the dam site, however regular inspections are required as per the Operation, Maintenance and Surveillance document(s) (which are included in the Dam Safety Management System, DSMS).

Identification of a potential dam safety threat may be derived from automatic sensors and control equipment at the dam, and/or reported by the dam operational staff, forestry operational staff, and/or downstream residents/members of the public. Automated warnings/alerts/alarms from monitoring equipment will go to the Dam Owner.

Reports from the public may be received by the Dam Owner, CDEM or the New Zealand Police. It is most important that any organisation receiving a report carries out its duties as set out in Section 4 of this EAP.

The Dam Owner or staff, on receipt of a message indicating a potential problem with the dam (i.e. a potential dam safety threat) will, without delay, inspect the dam and undertake an assessment resulting in an incident or dam safety emergency being declared.

It is the responsibility of the Dam Owner to assess potential dam safety threats and declare these are either an incident or a dam safety emergency. Where the Dam Owner declares a dam safety emergency, the dam owner shall also classify the emergency as an internal event, potential emergency, or imminent failure type event.

In the event that the Dam Owner cannot be contacted, CDEM may assume the role of assessing the potential dam safety threat and declaring and classifying dam safety emergencies. CDEM will decide whether a declared potential emergency or imminent failure type event requires declaration of a Civil Defence Emergency.

Where a dam safety emergency is declared, the Dam Owner shall investigate the cause, and instigate the necessary actions. Where the Dam Owner declares a potential emergency or imminent failure type event, they shall notify the New Zealand Police and Nelson Tasman CDEM as per the notification requirements in Section 3.3.3 of this EAP.

3.2.2 Identification and evaluation of dam safety threats

Some examples of potential dam safety threats specific to the Waimea Dam are provided in Table 3.1 below. These potential dam safety threats relate to the potential credible failure modes identified as part of the detailed design (refer T+T Stage 4 Detailed Design report for further information).

The examples give an indication of what might constitute a dam safety threat and do not provide a comprehensive list of all threats. Judgement is an essential part of the threat identification process, and a cautious approach to raising potential dam safety threats is strongly recommended (i.e. if there is any doubt on whether something constitutes a potential dam safety threat, this should be raised for further assessment). The dam inspection and potential dam safety threat assessment process is also used to confirm threats.

Table 3.1 also provides suggested criteria for assessing a threat as an incident or dam safety emergency, and classification of dam safety emergency type. Further details on classifying dam safety emergencies are provide in Section 3.2.3 below. A flowchart for classifying dam safety emergencies is provided in Figure 3.2.
### Example potential dam safety threats and classification

<table>
<thead>
<tr>
<th>Example potential threat description</th>
<th>Incident or emergency</th>
<th>Typical emergency classification</th>
<th>EAP section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water flowing through a breach in the dam.</td>
<td>Dam safety emergency</td>
<td>Imminent Failure</td>
<td></td>
</tr>
<tr>
<td>Spillway flow scouring and undermining the downstream face of the dam.</td>
<td>Dam safety emergency</td>
<td>Imminent Failure</td>
<td></td>
</tr>
<tr>
<td>Water overtopping the dam causing large scale scouring of the downstream face of the dam.</td>
<td>Dam safety emergency</td>
<td>Imminent Failure</td>
<td></td>
</tr>
<tr>
<td>Reservoir water level at or above maximum design flood level (202.53 m RL)</td>
<td>Dam safety emergency</td>
<td>Potential Emergency</td>
<td></td>
</tr>
<tr>
<td>Excessive seepage likely to result in unravelling of the downstream face of the dam</td>
<td>Dam safety emergency</td>
<td>Potential Emergency</td>
<td></td>
</tr>
<tr>
<td>Failure or impending failure of the dam spillway</td>
<td>Dam safety emergency</td>
<td>Potential Emergency</td>
<td></td>
</tr>
<tr>
<td>Spillway blockage with lake level rising</td>
<td>Dam safety emergency</td>
<td>Potential Emergency</td>
<td></td>
</tr>
<tr>
<td>Earthquake causing major damage and a risk of uncontrolled reservoir release</td>
<td>Dam safety emergency</td>
<td>Potential Emergency</td>
<td></td>
</tr>
<tr>
<td>Lake level at or above 199.13 m RL (1.93m above NTWL)</td>
<td>Dam safety emergency</td>
<td>Internal event</td>
<td></td>
</tr>
<tr>
<td>Slumping, cracking or erosion of the dam or its abutments</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>New seepage, sudden increase in seepage rates or a murky appearance to the seepage from the dam</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Damage to concrete face or parapet wall or loss of freeboard</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure of dam instrumentation, early warning or communications systems.</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spillway blockage or rockfall into spillway with reservoir below NTWL.</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landslides into the reservoir.</td>
<td>Internal event</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Automated drain flows given an alert</td>
<td>Incident</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Non scheduled valve closure</td>
<td>Incident</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

### 3.2.3 Classification of dam safety emergencies

#### 3.2.3.1 General

Each declared dam safety emergency shall be classified by the Dam Owner as either an internal event, potential emergency or imminent failure situation. A flowchart for classifying dam safety emergencies is provided in Figure 3.2 below. Further details on classification criteria are provided in the following sections also.
3.2.3.2 Definition of an incident

An incident is defined in the NZSOLD Guidelines 2015 as an occurrence that requires a response from one or more agencies, but does not pose an immediate risk to life, health, property and/or the environment. Incidents typically include, but are not limited to, unexpected plant operation, alert...
levels from dam surveillance instrumentation or identified changes in dam performance that are subsequently confirmed as being of a minor nature.

The separate dam surveillance manual covers routine actions for unusual events. These unusual events may also initially be considered dam safety threats before assessment and declaration as an incident.

3.2.3.3 Definition of Imminent Failure situation

An Imminent Failure Situation is when the dam shows evidence of an imminent dam failure with catastrophic consequences that would also effect external parties.

This is the most serious emergency for the dam and requires immediate notification to the NZ Police and Civil Defence and immediate action. Declaration of a Civil Defence Emergency by CDEM is likely.

3.2.3.4 Definition of Potential Emergency situation

A Potential Emergency is a condition of a serious nature developing suddenly or unexpectedly that may endanger the integrity of the dam or downstream property and/or life. If preventative action is not taken this situation can worsen to become an Imminent Failure situation.

A Potential Emergency situation requires immediate action.

Example Potential Emergency situations include, but are not limited to those described in Table 3.1.

3.2.3.5 Definition of Internal Event situation

An Internal Event is an event which takes place, or a condition which develops, that is not normally encountered in the routine operation of the dam and may have the potential to endanger its structure.

Internal Events must be evaluated to determine whether there has been any damage requiring correction, special safety measures needing to be implemented, and to assess if performance is in accordance with the design expectations.

Possible dam safety threats that would be classified as internal events include, but are not limited to those described in Table 3.1 above.

3.3 Notification procedures

3.3.1 Notification priorities

Following declaration of a dam safety emergency that is classified as a potential dam emergency or imminent failure type event, authorised staff of the Dam Owner shall notify the appropriate persons within the New Zealand Police, CDEM, Tasman District Council, the Dam Safety Consultant and other persons within the Dam Owner organisation. A notification flow chart and emergency contact list (with the appropriate persons and contact details) are appended to this EAP (refer Appendices C and D).

Coordination and prioritisation of the emergency management response downstream of the dam is the responsibility of the lead agency (definition follows in the next paragraph). The lead agency is to notify the potentially affected persons downstream of the dam in accordance with their communication protocols.

The New Zealand Police should be the first organisation to receive notification. This is because initial emergency response is usually coordinated by the New Zealand Police, and early notification
will enable confirmation of whether additional resources from Nelson Tasman CDEM are necessary. In the event that a Civil Defence Emergency is declared, the Nelson Tasman CDEM will assume the emergency response coordination role, noting that up until the time that CDEM are ready to assume this role the New Zealand Police typically act as the lead agency.

The following notification priorities are recommended for the lead agency in the event evacuation is required:

a Alert residents of the Lee Valley and lower Wairoa Valley by telephone to self evacuate if possible. NOTE: After an earthquake or a severe flooding event, telephone may not be a viable means of communication and/or access to/from the zone of potential inundation downstream may be severely limited.

Alternative means of communication are recommended and providing downstream residents with information to facilitate self-evacuation may be suitable (i.e. self evacuate or seek higher ground following large earthquakes).

b Alert residents to evacuate all the low lying areas along the Waimea River flood plain including residents of Brightwater, Hope, Spring Grove, Richmond and Redwood Valley.

c Advise residents to stay away from the Lee, Lower Wairoa and the Waimea Rivers.

Special Notes
If dam collapse is imminent (i.e. Imminent Failure situation), and the New Zealand Police and Nelson Tasman CDEM cannot immediately be contacted on the above numbers, ring 111 and report the incident to the New Zealand Police.

3.3.2 Notification format
When reporting to other services (e.g. New Zealand Police, and CDEM etc.) the Dam Owner should convey the following information (Appendix C provides a form):

a Name of person making report and organisation they represent.

b Name of dam (Waimea Dam) and location details.

c Description of problem.

d Location of identified cause of concern:
  – In relation to embankment (i.e. halfway up from toe).
  – In relation to the outlet.
  – In relation to dam crest.
  – In terms of what part of the dam is affected (i.e. upstream slope, downstream slope or crest).

e An estimate of the quantity of any unusual flow, as well as a description of flow quality (i.e. clear, cloudy, muddy, etc.).

f A reading of the reservoir level.

g An indication of whether the reservoir water level is rising, stable or falling.

h The current weather conditions at the site.

i An indication of whether the situation appears to be worsening, remaining stable, or improving.

j An indication of whether the situation appears to be containable or not.

k Anything else that the notifier considers to be important.
This information must be passed immediately to the New Zealand Police and Nelson Tasman CDEM and immediate confirmation of receipt received. The message must also be confirmed by emailing a completed copy of the Notification Form shown in Appendix C.

3.4 Preventative and emergency actions

3.4.1 Introduction

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

This section of the EAP outlines the actions to be taken by the Dam Owner following declaration of a dam safety emergency. The EAP cannot cover every possible condition, and judgement by the Dam Owner with design advice from the Dam Safety Consultant and/or other suitably qualified organisations will be necessary in other situations.

3.4.2 Preventative actions

Preventative actions need to be taken prior to and during the emergency situation. The important factor in the effectiveness of the EAP is the prompt detection and evaluation of information obtained from instrumentation and/or physical inspection and surveillance procedures.

The time factor for the onset of an emergency to awareness of imminent danger and its effect on the workability of the EAP is critical. Timely implementation of the EAP is a crucial element in its effectiveness and appropriate effective warning systems are imperative for emergency authorities to eliminate or minimise downstream effects or endeavour to avert substantial damage to the dam.

The primary action is to notify the New Zealand Police and local Civil Defence if there is the potential for an uncontrolled release of water from the dam.

The specific preventative actions required will vary to suit the identified dam safety threat. Preventative actions may include:

- Drawing down the reservoir water level in advance of a predicted significant rainfall event.
- Initiating emergency drawdown of the reservoir.
- Notifying the New Zealand Police to enable precautionary evacuation and/or exclusion zones to be implemented.

3.4.3 Emergency actions

3.4.3.1 Imminent Failure actions

In the event of an Imminent Failure situation being declared, the Dam Owner shall immediately undertake the following actions:

- Immediately notify the New Zealand Police and Nelson Tasman CDEM in accordance with the Notification Plan.
  - Declaration of a Civil Defence Emergency is likely.
- Fully open the outlet valves, if safe to do so, to assist in lowering the water level in the reservoir.
- Vacate the immediate vicinity downstream of the dam.
- Contact the Dam Safety Consultant for advice on possible further remedial action.
- Monitor, document and photograph dam status if safe to do so.


- Continue to liaise with, and provide information to, the New Zealand Police and Nelson Tasman CDEM as necessary.

Suitably trained staff should also be available on a 24-hour basis to respond quickly in the event of any imminent Failure situation being declared and to supervise necessary responses and/or remedial works. In preparation for possible remedial action the Dam Owner should at all times operate the dam in accordance with approved operating procedures and this EAP.

3.4.3.2 Potential Emergency actions

In the event of a Potential Emergency situation being declared, the Dam Owner shall immediately undertake the following actions as a minimum:

- Immediately notify the New Zealand Police and Nelson Tasman CDEM in accordance with the Notification Plan.
- Immediately inspect dam (as far as personnel safety permits) to assess the situation, with a Dam Safety Consultant if available.
- Fully open the outlet valves to assist in lowering the water level in the reservoir, if safe to do so.
- Monitor, record and report reservoir levels at least hourly.
- Contact the Dam Safety Consultant for advice on possible further remedial action.
- Continue to liaise with, and provide information to, the New Zealand Police and Civil Defence as required.

Suitably trained staff will also be available on a 24-hour basis to respond quickly in the event of any Potential Emergency situation being declared and to supervise necessary responses and/or remedial works. In preparation for possible remedial action the Dam Owner should at all times operate the dam in accordance with approved operating procedures and this EAP.

Further specific actions are described in Sections 3.4.3.4, 3.4.3.6 and 3.4.3.5 below for example dam safety threats that could result in a Potential Emergency being declared.

3.4.3.3 Internal event actions

Internal Event situations do not represent an immediate danger to the dam and therefore will not in themselves endanger property or lives downstream of the dam. All Internal Events shall be recorded in the appropriate documentation as per Appendix C of this EAP.

Nevertheless, a preliminary notification should be issued to the New Zealand Police and Nelson Tasman CDEM in accordance with the notification requirements shown in Section 5.3 of this EAP. Timely notification of a potential dam safety emergency can significantly reduce the downstream impacts and save lives.

Further specific actions are described in Sections 3.4.3.4, 3.4.3.6 and 3.4.3.5 below for example dam safety threats that could result in an Internal Event being declared.

3.4.3.4 Earthquake response actions

An inspection of the dam and appurtenant structures should be undertaken if an earthquake is felt or reported in the dam area. The categorisation of a declared dam safety emergency (i.e. as Internal Event, Potential Emergency or Imminent Failure) and therefore required actions depends on the condition of the dam only, rather than the size of the earthquake.

The dam is designed to safely withstand earthquakes up to the maximum credible earthquake (which considers an Alpine Fault rupture and Waimea Fault rupture). The design seismic criteria for
the dam include earthquakes with horizontal peak ground accelerations (pga) as per Table 3.2 below.

**Table 3.2**  
**Design peak ground accelerations for use in assessment of dam safety threat and response**

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Peak ground acceleration at foundation seismograph</th>
<th>Peak ground acceleration at dam crest seismograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating basis earthquake (OBE) – Design event where only minor repairable damage to the dam and appurtenant structures is intended to occur.</td>
<td>0.17g</td>
<td>0.55g</td>
</tr>
<tr>
<td>Safety evaluation earthquake (SEE) – Design event where significant damage to the dam and appurtenant structures is allowable provided this does not result in dam failure.</td>
<td>0.64g</td>
<td>1.90g</td>
</tr>
</tbody>
</table>

As the Waimea Dam features two seismographs, automated warning of an earthquake felt at the dam site is possible. The seismographs will enable the magnitude of the earthquake at the dam crest and foundation to be measured. Use of the seismograph data is the primary means of rapid assessment of the dam safety threat posed by an earthquake.

An alternative method of assessing the magnitude of an earthquake is by relatively subjective assessment of the felt effects. Table 3.3 below provides a guide for gauging the earthquake intensity using the Modified Mercalli Intensity scale (refer Appendix H for the full intensity range).

**Table 3.3**  
**Key Modified Mercalli Intensity Scales for earthquake response**

<table>
<thead>
<tr>
<th>Category (MM)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>MM4: Largely observed</td>
<td>Generally noticed indoors, but not outside, as a moderate vibration or jolt. Light sleepers may be awakened. Walls may creak, and glassware, crockery, doors or windows rattle.</td>
</tr>
<tr>
<td>MM5: Strong</td>
<td>Generally felt outside and by almost everyone indoors. Most sleepers are awakened and a few people alarmed. Small objects are shifted or overturned, and pictures knock against the wall. Some glassware and crockery may break, and loosely secured doors may swing open and shut.</td>
</tr>
<tr>
<td>MM6: Slightly damaging</td>
<td>Felt by all. People and animals are alarmed, and many run outside. Walking steadily is difficult. Furniture and appliances may move on smooth surfaces, and objects fall from walls and shelves. Glassware and crockery break. Slight non-structural damage to buildings may occur.</td>
</tr>
</tbody>
</table>

Note: Table adapted from GNS Science New Zealand website, September 2012.

Assessment of the dam safety threat and corresponding required actions will differ depending on whether or not the intensity of the earthquake is more or less than the OBE and/or a MM5 in the Modified Mercalli Intensity scale. A flow chart outlining the dam safety threat assessment process following an earthquake is presented in Figure 3.3 below.

The recommended actions as based on the earthquake size are provided below.
EARTHQUAKE FELT AT DAM SITE
Reported by seismographs and or felt intensity (MM)

Recorded PGA’s ≥ SEE (refer Table 3.2)
and/or felt intensity ≥ MM7
(refer Appendix H)

YES

LARGE EARTHQUAKE ACTIONS

NO

Recorded PGA’s ≥ OBE (refer Table 3.2) and/or felt intensity ≥ MM5
(refer Table 3.3)

YES

MODERATE EARTHQUAKE ACTIONS

NO

SMALL EARTHQUAKE ACTIONS

Figure 3.3  Earthquake actions flowchart
Large Earthquake actions (for recorded PGA’s ≥ SEE (refer Table 3.2) and/or felt intensity ≥ MM7)

- Immediately notify the New Zealand Police and Nelson Tasman CDEM and implement communication protocols as per Section 5.
- Commence emergency dewatering procedures as a precautionary action.
- Inspect the dam using site cameras immediately and review monitoring data where this is available.
- Monitor embankment seepage and other instrumentation for indications of a potential dam breach. If conditions indicate a dam breach is likely then immediately implement the Imminent Failure situation actions.
- Undertake site inspection as soon as practicable, noting that road access to the dam following very large earthquakes is unlikely and helicopter access is likely necessary.

重大地震行动（对于记录到的PGA ≥ SEE（参考表3.2）和/或感受烈度 ≥ MM7）

- 立即通知新西兰警察和南马海岸应急协调区（CDEM）并实施通讯协议，如第5章所述。
- 启动应急脱水程序作为预防措施。
- 使用现场摄像头立即检查大坝，并查看可用的监测数据。
- 监视坝体渗流和其他仪器，以查明潜在大坝崩解的迹象。如果条件表明大坝破裂的可能性巨大，应立即实施即刻失败情况行动。
- 尽早进行现场检查，注意，非常大的地震后，道路可能无法进入大坝，可能需要实施直升机搜索。

Moderate Earthquake actions (where Large Earthquake criteria not met and for recorded PGA’s ≥ OBE (refer Table 3.2) and felt intensity ≥ MM5 (refer Table 3.3))

- Inspect the dam using site cameras immediately and review monitoring data where this is available.
- Monitor embankment seepage and other instrumentation for indications of a potential dam breach. If conditions indicate a dam breach is likely then immediately implement the Imminent Failure situation actions.
- Undertake site inspection as soon as practicable, noting that road access to the dam following very large earthquakes is unlikely and helicopter access is likely necessary.

中等地震行动（当大震标准未满足时，且记录到的PGA ≥ OBE（参考表3.2）和感受烈度 ≥ MM5（参考表3.3））

- 使用现场摄像头立即检查大坝，并查看可用的监测数据。
- 监视坝体渗流和其他仪器，以查明潜在大坝崩解的迹象。如果条件表明大坝破裂的可能性巨大，应立即实施即刻失败情况行动。
- 尽早进行现场检查，注意，非常大的地震后，道路可能无法进入大坝，可能需要实施直升机搜索。
Inspect and report on the embankment, abutments, spillway and appurtenant structures. Check for springs, change in seepage rates, deformation, erosion and concrete damage. Record location, extent and severity of any damage.

- If major damage has occurred at the dam and there is a risk of uncontrolled release of the stored water, this should be treated as a Potential Emergency situation as defined in Section 3.2.3.
  
  Major damage and a risk of uncontrolled release is considered to exist if:
  - Significant loss of freeboard with the potential for the dam to be overtopped (i.e. dam crest and settled and/or parapet wall has moved significantly).
  - Spillway blockage has occurred and the spillway is likely to operate prior to removal of blockage.
  - Significant damage to spillway concrete lining has occurred and the spillway is likely to operate prior to repairs being carried out.

- Notify the New Zealand Police and Nelson Tasman CDEM of the situation at the dam site if significant changes from normal conditions are observed.
- Test and confirm operation of communication systems and dam instrumentation.
- Monitor seepage flows for indication of possible damage to the concrete face or its joints. If seepage flows increase, contact the Dam Safety Consultant to determine possible remedial actions.
- If visible damage has occurred, evaluate and determine whether special safety measures or corrective action is required.
- Undertake deformation survey of the dam, spillway and appurtenant structures as soon as reasonably practicable, and assess extent of deformation relative to design criteria (Dam Safety Consultant to advise).

Small Earthquake actions (recorded PGA’s < OBE (refer Table 3.2) and felt intensity < MM5 (refer Table 3.3))

- Inspect the dam as soon as reasonably practical.
- Inspect and report on the embankment, abutments, spillway and appurtenant structures. Check for springs, change in seepage rates, deformation, erosion and concrete damage. Record location, extent and severity of any damage.
- If major damage has occurred, implement moderate earthquake dam procedures.

3.4.3.5 Flood response actions

The following actions are recommended for large floods with actions corresponding to the flood magnitude and assessed dam condition. Given the reservoir level changes over the duration of a flood event, the flood actions are staged to suit the reservoir water level.

Lake level at or above 199.13 m RL (1.93m above NTWL)

This is the level that the base of the parapet wall joins the concrete face slab and is just above the design mean annual flood level (MAFL) (199.09 m RL).

- Provide a suitably trained observer at the dam site who can accurately monitor and report on the situation.
- Test and confirm operation of communication systems and dam instrumentation.
- Monitor and record meteorological forecasts, rainfall, all dam instrumentation and seepage flows.
• Alert operations staff to the situation.
• Monitor the debris accumulation on the upstream debris boom.

**Lake level at or above 200.48 m RL (3.28 m above NTWL)**

• Provide early notification to the New Zealand Police and Nelson Tasman CDEM of reservoir water level and standby should water level continue to increase.
• Continue to implement measures as per MAFL actions above.
• Remote monitoring of seepage flows is unlikely due to tailwater level drowning the instruments.
• Monitor spillway operation at least hourly from a safe distance and only if safe to do so.
• Monitor for evidence of a Potential Emergency situation.

**Reservoir water level at or above maximum design flood level (202.53 m RL)**

• Immediately notify the New Zealand Police and Nelson Tasman CDEM and implement communication protocols as per Section 5.
• Continuously monitor and record meteorological forecasts and rainfall.
• Evacuate any on site personal away from the dam and close security gates to prevent/discourage access over the spillway bridges.
• Remote monitoring of seepage flows is unlikely due to tailwater level drowning the instruments.
• Monitor spillway operation continuously from a safe distance on the true left abutment only and only if safe to do so.
• Continuously monitor for evidence of a Potential Emergency or Imminent Failure situation.

### 3.4.3.6 Spillway condition responses

**Failure or impending failure of the dam spillway**

• Monitor spillway conditions from a safe distance.
• Notify the New Zealand Police to prepare for an Imminent Failure scenario to be declared.

**Spillway blockage or rockfall into spillway**

If such an event has occurred and forecast inflows into the reservoir are likely to result in the reservoir level exceeding the NTWL before the blockage can be cleared, this should be treated as an Potential Emergency situation as defined in Section 3.2.3.

Otherwise:

• Provide preliminary warning to the New Zealand Police and Civil Defence in accordance with the notification requirements shown in Section 5.3.
• Mobilise suitably experienced staff, machinery/equipment and remove the blockage from the spillway.
• Monitor reservoir level closely and if the water level is rising, increase discharge from the outlet valves as required to limit the rise in water level.
• If unable to clear the blockage, this should be treated as a Potential Emergency situation as defined in Section 3.2.3.
• Contact the Dam Safety Consultant for advice on possible further remedial action.
3.4.3.7 Other occurrences

New seepage, sudden increase in seepage rates or a murky appearance to the seepage from the dam

- Record and photograph location, extents and estimate rate of any new seepage.
- Monitor and record seepage rates, take samples of murky seepage.
- Evaluate the situation and determine whether special safety measures or corrective action is required.
- If the seepage issue is evaluated as severe enough to jeopardise the safety of the dam this should be treated as a Potential Emergency situation as defined in Section 3.2.3. Rockfill dams can withstand significant seepage before stability issues occur.
- Contact the Dam Safety Consultant for advice on possible further remedial action.

Excessive seepage likely to result in unravelling of the downstream face of the dam

- Monitor and record all seepage flows continuously with manual measurements at least hourly if required and safe to do so.

Slumping, cracking or erosion of the dam or its abutments

- Record and photograph location, extent and severity of the damage.
- Evaluate the damage and determine whether special safety measures or corrective action is required.
- If the damage is evaluated as severe enough to jeopardise the safety of the dam this should be treated as a Potential Emergency situation as defined in Section 3.2.3.
- Contact the Dam Safety Consultant to determine possible remedial actions.

Damage to concrete face or parapet wall or loss of freeboard

- Record and photograph location, extent and severity of the damage.
- If a loss of freeboard, conduct a deformation survey.
- If a loss of freeboard, lower the water level as deemed necessary by increasing the discharge from the outlet valves.
- Evaluate the damage and determine whether special safety measures or corrective action is required.
- If the damage is evaluated as severe enough to jeopardise the safety of the dam this should be treated as a Potential Emergency situation as defined in Section 3.2.3.
- Contact the Dam Safety Consultant for advice on possible further remedial action.

Failure of dam instrumentation, early warning or communications systems

- As soon as reasonably practical mobilise experienced Staff to the dam site with suitable monitoring and communications equipment to remedy the situation or monitor the dam until the failed equipment can be restored.
  - Immediate mobilisation required if early warning system is involved.
  - Immediate mobilisation required if inclement weather is forecast and the dam spillway is likely to become operational.

Landslides into the reservoir

The dam has 5.63 m of freeboard with the reservoir at NTWL. It is expected to be capable of accommodating waves that may be generated by the identified landslides as described in the design.
report. However should it become apparent that a landslide around the reservoir rim may be mobilising, the following actions are recommended:

- Provide preliminary warning to the New Zealand Police and Civil Defence in accordance with the notification requirements shown in Section 5.3.
- Engage a suitably experienced geologist to investigate the landslide. This will likely require boat access to the reservoir.
- Where access permits, deformation monitoring of the landslide should be undertaken.
- Contact the Dam Safety Consultant for advice on possible further remedial action.

### 3.5 Termination and Documentation

A dam safety emergency, once declared, shall not be terminated by the Dam Owner until potential failure has been addressed and the increased risk of failure has been alleviated or the failure incident has ended (e.g. dam break flood has completely receded).

Civil Defence Emergencies can only be declared and terminated by people with specifically designated roles in accordance with the Civil Defence Emergency Management Act.

Following a dam safety emergency, the Dam Owner shall fully document the emergency response in a report. The report should include discussion on:

- The dam safety threat that initiated the emergency.
- The response actions taken by the Dam Owner and all emergency service agencies.
- The extent of any damage to the dam and/or appurtenant structures.
- The extent and effect of any downstream inundation.
- The justification for terminating the dam safety emergency.
- The strengths and weaknesses of the existing EAP including the emergency management procedures, equipment, resources and leadership.
- Corrective actions to address any identified weaknesses in the EAP.

The complete report should subsequently be circulated to all relevant parties to communicate any lessons learned and for continuous improvement.
4 EAP Responsibilities

4.1 Dam Owner

4.1.1 General

The Dam Owner has a responsibility to operate the dam in a manner that is considered to meet sound engineering and professional standards, to meet all relevant legislative guidelines and in accordance with the Waimea Dam Operating Procedures. These procedures should consider the latest revision of the NZSOLD Guidelines and the Resource Consent conditions.

From an emergency planning perspective the Dam Owner is responsible for:

a Providing advice in the preparation of this EAP.
b Complying with the detail of this EAP.
c Ensuring that all the staff involved in the operation of the Waimea Dam are familiar with this EAP, and the company obligations in it.
d Ensuring that suitably trained and authorised staff are available to competently assess potential dam safety threats and declare and classify dam safety emergencies. The staff must be familiar with this EAP. Authorised staff must discharge the responsibilities of the Dam Owner over the emergency event duration until termination and documentation procedures are completed.
e Having facilities and procedures in place to give warnings to New Zealand Police and Civil Defence in the event of dam safety emergency situations or potential dam safety threats that may arise at the dam site.
f Maintaining a schedule of the expertise, staff, materials and equipment to counter threats to the integrity of the dam.
g Maintaining an arrangement with the Meteorological Service to be provided with heavy rainfall warnings.
h Testing and maintaining the effectiveness of this Emergency Action Plan.

4.1.2 Suitably trained staff

Suitably trained staff are deemed to be those who can:

a Recognise potential dam safety threats and dam safety emergency situations as listed in this plan, and understand their possible effects on the integrity and safety of the dam.
b Understand that the example potential dam safety threats in Section 3 are not an exhaustive list of every possible condition that could arise, and that judgement must be judiciously applied when assessing situations.
c Acknowledge the importance of providing early notification to the New Zealand Police, Civil Defence and potentially affected parties downstream, of potential dam safety threats and/or dam safety emergencies at the dam site.
d Are authorised to notify the relevant parties, declare a dam safety emergency and enact the procedures of this EAP.
e Accurately monitor, record and report on reservoir levels in relation to reservoir staff gauge and dam crest.
f Accurately complete the Notification Report as shown in Appendix C.
g Readily access the emergency contact numbers required to notify the New Zealand Police and Civil Defence (see contact list in Appendix D).
h. Operate communication equipment used to convey emergency messages (e.g. cell phone, email, satellite phone, radio).

i. Correctly interpret and manage the implementation of the preventative actions set out in Section 3.4 of this plan.

j. Liaise with the Dam Safety Consultant where specialist advice is required. The acquisition of such advice must not delay the notification of potential dam safety threats and/or declaration of a dam safety emergency.

k. Safely supervise any of the operational tasks that may be necessary to remedy dam safety threats.

4.1.3 Identification, assessment and classification of emergencies

The Dam Owner is responsible for identifying dam safety threats, assessing these threats and classifying them as incidents, or dam safety emergencies. The Dam Owner is responsible for declaring and classifying the type of dam safety emergency in accordance with Section 3 of this EAP.

4.1.4 Notification

The Dam Owner is responsible for notifying the emergency services of dam safety emergencies that may affect external parties (i.e. Potential Emergency and Imminent Failure situations). Notification shall be in accordance with the requirements of Section 5.3. The emergency services area responsible for all subsequent notification of potential affected downstream residents and evacuation (refer Section 4.2 below).

4.1.5 Preventative and Emergency Actions

The Dam Owner is responsible for implementing all preventative and emergency actions at the dam in accordance with Section 3.4 of this EAP.

4.1.6 Termination and documentation

The Dam Owner is responsible for terminating, notifying and documenting a dam safety emergency in accordance with Section 3.5 of this EAP.

4.2 Emergency services responsibilities

4.2.1 New Zealand Police

The New Zealand Police are responsible for maintaining law and order during an emergency. The New Zealand Police are often required to accept initial responsibility for coordination of an emergency response, followed by transfer of this role to the appropriate lead agency (once confirmed). In the instance of a dam safety emergency being declared at the Waimea Dam, the New Zealand Police are the first party to be notified under this EAP.

Specific nominated responsibilities of the New Zealand Police in relation to dam safety emergencies at the Waimea Dam are:

a. Providing advice in the preparation of this EAP.
   
   [Not yet provided at the time of writing this draft of the EAP]

b. Include consideration of this EAP with other Police plans and procedures in the region.

c. Having systems in place to receive notifications of potential dam safety threats and providing timely notification of the Dam Owner.
Having systems in place to receive notifications of dam safety emergencies to enable early implementation of Police procedures.

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

Maintaining a current contact list of all residents downstream from the Waimea Dam that may be affected by failure of the dam.

Establishing and maintaining a notification system for warning downstream residents, as well as Fire and Ambulance Services, in the event of a dam safety emergency at the dam site.

4.2.2 Civil Defence

The local civil defence group for the Tasman District is the Nelson Tasman Civil Defence Emergency Management Group.

It is important that this EAP is compatible with the current Nelson Tasman CDEM emergency plans, and that Civil Defence planning specifically considers a dam safety emergency from the Waimea Dam.

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

Civil Defence planning responsibilities can be summarised as:

a Providing advice in the preparation of this plan.
   [Not yet provided at the time of writing this draft of the EAP]

b Notifying the Dam Owner of any external event of which they have knowledge that may affect the safety of the Waimea Dam.

c Providing advice on the compatibility of this EAP with the current Nelson Tasman Civil Defence plan.

d Maintaining an easily accessed contact system to ensure they can receive early warnings, and keeping the Dam Owner informed of any external events and/or information which may assist in assessing a potential dam safety threat at the dam site.

e Maintaining their own plan for the handling of emergencies that may arise out of a sudden release of water from the Waimea Dam (i.e. an Imminent Failure situation).

Civil Defence emergency contacts are maintained by the Tasman District Council.

Civil Defence should advise the Dam Owner of any external event known to them that could be considered to pose a possible threat to the Waimea Dam (e.g. extreme weather warnings from the Meteorological Service).

Civil Defence should have systems in place to allow easy contact from the Dam Owner, or any other agency or individual wishing to advise of potential dam safety threats relating to the Waimea Dam.

4.2.3 Fire and Emergency New Zealand and St John Ambulance

Both Fire and Emergency New Zealand and St John Ambulance may be notified of potential dam safety threats advised by members of public and/or by the New Zealand Police of a dam safety emergency.

Fire and Emergency New Zealand and St John Ambulance are responsible for notifying the Dam Owner of a potential dam safety threat where this is given by a member of the public.
Fire and Emergency New Zealand and St John Ambulance develop and maintain their own specific procedures relating to emergency situations including potential dam safety threats and dam safety emergencies. This EAP outlines the linkages to such plans and procedures, noting that coordination and integration of the relevant aspects of each organisations plans is necessary.

Fire and Emergency New Zealand and St John Ambulance organisations should maintain easily accessible contact systems to allow receipt of warnings from the New Zealand Police and/or CDEM.

4.3 Dam safety consultant

The Dam Safety Consultant for the Waimea Dam shall be available to provide dam safety advice where requested by the Dam Owner on potential dam safety threats and in the event a dam safety emergency is declared.

The Dam Safety Consultant will be confirmed following commissioning. Typically, the dam designer (Tonkin & Taylor Ltd) is retained as the Dam Safety Consultant.

The Dam Safety Consultant should provide up to date emergency contact details for inclusion in the EAP. Where a key person is unavailable for a set period of time (e.g. is off shore), the Dam Safety Consultant should advise the Dam Owner.

In the unlikely event that the Dam Safety Consultant is not be contactable, the Dam Owner should seek specialist advice from other suitably qualified dam safety engineering persons/organisations.
5 Emergency preparedness

5.1 Access to site

The Waimea Dam is located on the Lee River, a tributary of the Waimea River in the Tasman District as shown in Figure 5.1. The dam site is located on the Lee Valley Road, approximately 16.5 km from the State Highway 6 junction with River Terrace Road in Brightwater. An alternative road route to the site, approximately 30km long, is via the Edward Street / State Highway 6 junction in Wakefield. Figure 5.1 shows both main routes to the dam with the River Terrace Road Route in Blue and the longer Edward Street Route in Green.

Figure 5.1 Dam location map

There are other, more difficult, forestry access routes to the dam site in the event that the main roads are not passable but these may require four wheel drives / farm bikes and portions on foot. However, these may not be passable in severe storm conditions or after a major earthquake.

The location of the dam makes it possible that access to the site will be unavailable in the event of extreme weather or major earthquake.
However, in the event that access cannot be gained in a timely manner then it may be possible to contact and use the assistance of residents local to the dam. If possible, provision should be made for some degree of training to enable these residents to assist should such an eventuality occur. The safety of the residents in undertaking any assistance should be given due consideration.

Air access to the dam by helicopter is also an option; however landing of a float plane on the reservoir could be dangerous due to the risk of floating forestry debris. The nearest airstrip is Nelson Airport approximately 33 km from the dam by road.

Table 5.1 provides coordinates for the Waimea Dam site to three common coordinate systems.

### Table 5.1 Waimea Dam coordinates

<table>
<thead>
<tr>
<th>Coordinate System</th>
<th>Latitude or Northing</th>
<th>Longitude or Easting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latitude and Longitude (WGS84)</td>
<td>41° 28' 13.2&quot; S</td>
<td>173° 09' 40.9&quot; E</td>
</tr>
<tr>
<td>New Zealand Transverse Mercator (NZTM2000)</td>
<td>5409017 mN</td>
<td>1613473 mE</td>
</tr>
<tr>
<td>New Zealand Map Grid (NZMG49)</td>
<td>5970712 mN</td>
<td>2523466 mE</td>
</tr>
</tbody>
</table>

Further discussion about site access in conjunction with flood inundation mapping is contained in Section 6.6 and Appendix B.

### 5.2 Response During Darkness or Adverse Weather

Planning for emergency access should work on the premise that it is dark and raining, and/or that Lee Valley Road will be impassable due to storm or earthquake induced slips and/or flooding. Hence, access for emergency action or repairs may be difficult and/or dependent on the availability of earthmoving equipment to clear slips and debris. It is also likely that helicopter availability may be compromised by other priorities.

### 5.3 Communications systems

This section briefly describes the communications systems available at Waimea Dam.

[To be confirmed once communications systems at the dam site have been confirmed.]

Arrangements include:

- Satellite Phone.
- Radio communications via the SCADA network.
- Fibre communications via fibre cable to site.

The Dam Owner will ensure that these communication systems are maintained and remain operable as far as is reasonably possible.

### 5.4 Work at the site

The location of all staff involved in investigating or monitoring potential dam safety threats and dam safety emergencies at the dam should wherever possible be known to another responsible person at all times.

Wherever possible two people should assess dam safety threats at the site. Contact must be established and regularly maintained with an external party at no more than one hourly intervals.
The Dam Owner will ensure that suitably trained staff will be available to cope with all reasonable activities required under this EAP and the Operating Procedures under foreseeable weather and post-earthquake conditions.

The Dam Owner will ensure that appropriate safety equipment and information (including a copy of this Emergency Action Plan) and the routine test and inspection records are kept at the site.

Site lighting may not be working at the dam, especially during or following a natural hazard event. Therefore, for action during periods of darkness staff should use vehicle headlights and take battery and vehicle operated spotlights to site.

Communications from the site could be significantly more difficult during periods of adverse weather or in post-earthquake conditions. It is therefore important that all the systems are regularly checked throughout a response event and that care is taken to ensure all messages are correctly received.

5.5 Emergency power supplies

The power supply to the dam may be interrupted following a natural hazard event. For this reason a backup power supply is located at the dam. However, if required additional emergency power supply can be obtained from the sources noted in Appendix E.

5.6 Sources of Equipment and Materials

5.6.1 Special equipment

Special equipment in the form of earthmoving plant may be required under certain situations. This plant is large, slow to move and therefore due allowance must be made for the time it will take to reach the site. Wherever possible equipment located in the vicinity of the dam should be used.

Civil Defence has the right to commandeering equipment in the event of a Civil Defence Emergency and therefore, close coordination should be maintained with Civil Defence. As the Waimea Dam is a High PIC structure, and one of the key credible potential failure modes (spillway blockage) requires heavy vehicle access to respond (excavators to clear debris), priority should be given to re-establishment of vehicle access to the dam in the event of a dam safety emergency.

Special equipment sources are noted in Appendix E.

5.6.2 Supplies and materials

Riprap, sandbags and other construction materials can be sourced from the local suppliers noted in Appendix E.

5.7 Warning system

The Dam Owner should establish an early warning system as an integral part of the EAP. Possible early warnings system could comprise some or all of the following:

- Rainfall gauges (upstream of the dam and at the dam site, existing rainfall gauge: Lee at Trig F).
- Backup power supply.
- Telemetry transmission of monitoring data.
- River flow gauges on the Lee River immediately downstream of the dam and on the Lee River upstream of the confluence with the Wairoa River.
- A monitoring station should be set up at the dam site which receives automatic rainfall readings, real time reservoir levels, spillway discharge and seepage flows. The monitoring station should include landline, satellite telephone, mobile telephone, radio, and facsimile facilities.

5.8 Testing of the EAP

5.8.1 Tests

Where a test of the EAP is proposed, ensure that the communication messages commence with the words “This is a limited communications test (or full communications, or operational test, as appropriate), of the notification procedure for the Waimea Dam”, or similar, so that there is no doubt that it is a test, and not an emergency. Three levels of testing are given in this EAP as below:

a) Limited tests within the Dam Owner’s organisation

It is the responsibility of the Dam Owner to initiate and coordinate a limited communication test involving only the Dam Owner at least once every year. Additional tests shall be conducted at the discretion of the Dam Owner whenever justified by staff changes or for other reasons.

b) Full communication tests

It is the responsibility of the Dam Owner to initiate and coordinate a full communications test involving the entire notification procedures for a dam safety emergency including an imminent failure type event.

c) Operational tests

It is the responsibility of the Dam Owner, in consultation with the relevant emergency services, to organise operational tests of the EAP procedures. The timing of the operational tests shall be determined in cooperation with the New Zealand Police and the Civil Defence agencies.

Coordination of the civil emergency response procedures is the responsibility of the Regional Civil Defence Officer, and the local civil defence response authorities. Given the initial role the New Zealand Police are likely to have in any actual emergency it is important that this is considered in operational testing.

The Dam Owner will participate in planning and execution of the operational tests including development of the form or scenario for each test, providing information as required and sending out notification of the simulated breach.

5.8.2 Test reporting

The Dam Owner shall maintain a record of each test on the “Record” form which follows, noting the date and time of the test and the person initiating it.

Test reports shall be sent by each participant to the Dam Owner using the “Test Report” form in Appendix G.

The Dam Owner shall prepare and keep a brief summary of each test, noting the problems encountered and steps taken to eliminate similar problems in the future.

5.9 EAP review and revision procedures

5.9.1 General

Ongoing review is essential maintaining the currency and effectiveness of the EAP for the agencies responsible for emergency response. While the responsibility for review and maintenance of the EAP
rests with the Dam Owner, it is important that the responsible emergency services provide regular review and feedback into the EAP.

5.9.2 Dam Owner

The Dam Owner should undertake the following actions to maintain the EAP:

a  On receipt of the EAP

The Dam Owner shall arrange for operational staff and emergency services staff that have a role in the emergency response to receive a thorough briefing on the EAP to facilitate review. The New Zealand Police, Nelson Tasman CDEM and other emergency services may also run their own internal training courses and these may result in review comments being supplied to the Dam Owner.

Familiarity with the procedures for emergency reporting, notification, and action is essential for timely implementation. Regular testing of the EAP is often the most effective means of maintaining familiarity and facilitating meaningful review.

b  Periodic review

The Dam Owner shall arrange periodic review of the EAP with the local Civil Defence staff to maintain familiarity with the regularly updated EAP. Periodic reviews and briefings shall be scheduled at a frequency to be agreed between the Dam Owner, the New Zealand Police, Civil Defence and any other appropriate outside agency.

Review by outside agencies should be recorded in the revision form found in Appendix I.

c  Revisions

The Dam Owner is responsible for ensuring the currency of this EAP and for arranging for the preparation of revisions and the distribution of copies thereof to all EAP recipients.

Proposed revisions should be addressed to the Dam Owner. A complete list of all revisions shall be maintained at the front of the EAP.

5.9.3 Emergency Services

Copies of the EAP will need to be sent to all relevant agencies. Each agency involved in the emergency response will be asked to provide inputs into the regular review, updating and testing of the EAP. The purpose of the review will be primarily to check the EAP for compatibility with the emergencies services own procedures, and provide comment on the adequacy and suggested improvements to the Dam Owner.

The emergency services are also expected to participate in the full communications and operational tests. It is expected that updates to the EAP will be required following testing. EAP testing forms are included in Appendix G.

The Dam Owner will arrange for the revision of the EAP as necessary. A record of review by outside agencies shall be maintained by the Dam Owner (refer Appendix I).
6 Supporting Information

6.1 Waimea Dam technical data

Table 6.1 below summarises the key technical information for the dam.

<table>
<thead>
<tr>
<th>Embankment characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment type</td>
<td>Concrete Face Rockfill Dam (CFRD)</td>
</tr>
<tr>
<td>Embankment volume (approximate)</td>
<td>435,000 m³</td>
</tr>
<tr>
<td>Nominal crest elevation (excluding camber)</td>
<td>201.23 m RL</td>
</tr>
<tr>
<td>Top of parapet wall (excluding camber)</td>
<td>203.13 m RL</td>
</tr>
<tr>
<td>Design Camber</td>
<td>0.3 m</td>
</tr>
<tr>
<td>Maximum dam height (from riverbed to dam crest on CL)</td>
<td>53 m</td>
</tr>
<tr>
<td>Crest length (approximately)</td>
<td>220 m</td>
</tr>
<tr>
<td>Crest width (excluding abutment turning area)</td>
<td>6 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hydrology, reservoir and flood routing characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Catchment area</td>
<td>77.5 km²</td>
</tr>
<tr>
<td>Normal top water level (NTWL)</td>
<td>197.2 m RL</td>
</tr>
<tr>
<td>Reservoir storage at NTWL</td>
<td>13 Mm³</td>
</tr>
<tr>
<td>Reservoir area at NTWL</td>
<td>630,000 m²</td>
</tr>
<tr>
<td>Inflow design flood peak water level (IDFL)</td>
<td>202.53 m RL</td>
</tr>
<tr>
<td>Reservoir storage at IDFL</td>
<td>16.6 Mm³</td>
</tr>
<tr>
<td>200 year ARI flood level</td>
<td>200.48 m RL</td>
</tr>
<tr>
<td>Reservoir storage at 200 year ARI flood</td>
<td>15.2 Mm³</td>
</tr>
<tr>
<td>Reservoir storage at top of parapet wall (203.13 m RL)</td>
<td>16.8 Mm³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Spillway characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary spillway type</td>
<td>Ogee Weir</td>
</tr>
<tr>
<td>Ogee weir effective length (on arc)</td>
<td>41.89 m</td>
</tr>
<tr>
<td>Peak outflow – Mean Annual Flood (MAF)</td>
<td>179 m³/s</td>
</tr>
<tr>
<td>Peak outflow – 200 year ARI flood</td>
<td>472 m³/s</td>
</tr>
<tr>
<td>Peak outflow – Inflow Design Flood (IDF) (PMF)</td>
<td>1060 m³/s</td>
</tr>
<tr>
<td>Dam crest flood outflow – Reservoir level at top of parapet wall</td>
<td>1152 m³/s</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Spillway and Energy dissipation characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Chute length (plan – ogee crest to start of flip bucket)</td>
<td>124 m</td>
</tr>
<tr>
<td>Chute width, narrow section</td>
<td>20 m</td>
</tr>
<tr>
<td>Chute horizontal transition length</td>
<td>71 m</td>
</tr>
<tr>
<td>Chute vertical curve length</td>
<td>21 m</td>
</tr>
<tr>
<td>Chute minimum height of concrete lining</td>
<td>3.0 m</td>
</tr>
</tbody>
</table>
Dissipation type
Flip bucket radius
Bucket lip level

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dissipation type</td>
<td>Flip Bucket</td>
</tr>
<tr>
<td>Flip bucket radius</td>
<td>20 m</td>
</tr>
<tr>
<td>Bucket lip level</td>
<td>156.6 m RL</td>
</tr>
</tbody>
</table>

**Outlet characteristics**

<table>
<thead>
<tr>
<th>Outlet type</th>
<th>Sloping pipes on the upstream face with removable screens and valve control.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of outlets</td>
<td>2</td>
</tr>
<tr>
<td>Outlet level – Upper (elevation of top of bellmouth)</td>
<td>181.5 m RL</td>
</tr>
<tr>
<td>Outlet level – Lower (elevation of top of bellmouth)</td>
<td>163.0 m RL</td>
</tr>
<tr>
<td>Pipe diameter and material</td>
<td>DN1000 epoxy coated steel</td>
</tr>
<tr>
<td>Control type</td>
<td>Fixed Cone Discharge Valves (2x DN850 and 2x DN300 valves(^1)) Butterfly isolation valves (2x DN1000)</td>
</tr>
<tr>
<td>Maximum design discharge capacity (No valve manufacturer velocity limits applied)</td>
<td>20 m(^3)/s (dewatering)</td>
</tr>
<tr>
<td>Concrete conduit size under embankment (internal dimensions)</td>
<td>Twin 2.5 m wide x 4.0 m high</td>
</tr>
</tbody>
</table>

**River tailwater characteristics (at confluence with spillway)**

| Tailwater level MAF | 150.85 m RL |
| Tailwater level 200 year ARI | 153.46 m RL |
| Tailwater level IDF (PMF) | 156.54 m RL |

6.2 **Storage elevation curve**

Figure 6.1 presents the reservoir water storage versus water elevation relationship.

![Waimea Dam Storage Elevation Curve](image)

*Figure 6.1 Storage elevation curve for the Waimea Dam indicating NTWL.*
6.3 Spillway rating curve

Figure 6.2 below shows the rating curve (flow rate versus reservoir level) for the Waimea Dam spillway.

![Waimea Dam spillway rating curve](image)

Figure 6.2  Spillway rating curve.

6.4 Emergency drawdown capacity

The emergency drawdown capacity of the Waimea Dam is controlled by the outlet works. Further details on the drawdown process and procedures, constraints and timeframes are provided in Appendix F.

6.5 Surveillance instrumentation

The surveillance instrumentation for the Waimea Dam is outlined separately in the Surveillance Manual and the attached Drawings in Appendix A.

6.6 Potential inundation maps

A dam break analysis has been undertaken to map the zone of potential inundation downstream of the dam in the event of a dam break event. This analysis also determined the likely flood wave travel times to given an indication of the likely evacuation times to potentially affected areas downstream.

The zone of potential inundation is the indicative area that might be flooded to a depth of 0.5 m or deeper as a result of dam failure. Persons and property in this zone are at risk following a dam failure. Preparation by others (e.g. the New Zealand Police and Nelson Tasman CDEM) of evacuation zones and planned cordons around this area should be based on this zone noting the zone is indicative and a precautionary approach is essential.

The flood wave travel times are based on hydraulic modelling and give an indication of the available time to evacuate potentially affected locations downstream. Indicative peak water depths are also included on the inundation maps.
The inundation maps attached in Appendix B are provided for emergency management planning and evacuation purposes. These inundation maps are subject to review and updates and are intended to be shared with external agencies who have a role in emergency planning for the Waimea Dam (e.g. New Zealand Police and Nelson Tasman CDEM).

The appended inundation maps present the zone of potential inundation for:

1. A sunny day failure (e.g. after a major earthquake).
2. A rainy day failure (e.g. during a very large flood event).

The inundation maps present the rainy day failure scenario for the Probable Maximum Flood event (i.e. the largest possible flood in the catchment) in the Lee River catchment coinciding with the 100 year event in the other tributaries of the Wairoa and Waimea Rivers.

The elapsed time from dam breach initiation until the first wave arrives (warning time) and the elapsed time to the peak water depth is given at specific locations downstream from the Waimea Dam in Table 6.2. The highest risk to life and buildings in the unlikely event of a failure of the Waimea Dam is the area to the east and north of Brightwater.

### Table 6.2 Indicative Flood wave travel times at key locations

<table>
<thead>
<tr>
<th>River Chainage (m)</th>
<th>Location</th>
<th>Time for flood-wave to first arrive (min)</th>
<th>Time for peak water depth to occur (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Waimea Dam</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>2910</td>
<td>Lucy Creek confluence</td>
<td>3</td>
<td>38</td>
</tr>
<tr>
<td>8220</td>
<td>Fairdale</td>
<td>11</td>
<td>42</td>
</tr>
<tr>
<td>12720</td>
<td>Wairoa River confluence</td>
<td>15</td>
<td>48</td>
</tr>
<tr>
<td>16470</td>
<td>State Highway 6 bridge at Brightwater</td>
<td>20</td>
<td>61</td>
</tr>
<tr>
<td>20330</td>
<td>Wai-iti River confluence</td>
<td>27</td>
<td>76</td>
</tr>
<tr>
<td>24220</td>
<td>State Highway 60 bridge</td>
<td>64</td>
<td>98</td>
</tr>
</tbody>
</table>

The basis of the appended inundation maps is covered in the Lee Valley Dam (now Waimea Dam) Dam Break Analysis and Hazard Assessment report (T+T, 2009) or subsequent updates.
7 Construction Emergency Action Plan

The Constructor (or an appropriate person appointed by the Dam Owner) is required to provide a separate Construction and Commissioning EAP to cover the period whilst the dam is being built and commissioned. Construction is not covered by this EAP.
8 Applicability

This report has been prepared for the exclusive use of our client Waimea Water, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by: Reviewed by:

……………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………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Appendix A: Selected Drawings

Draft 2
14 August 2018
Inundation map showing the depth of flooding from the junction of the Wairoa and Lee Rivers to the mouth of the Waimea River.
Assessment of inundation area based on $dv$ (depth multiplied by velocity)
Appendix C: Notification checklists and forms

- Notification Flow Chart
- Notification Report
WAIMEA DAM EMERGENCY ACTION PLAN
NOTIFICATION REPORT

1. NAME/Organisation ......................................... 2. DATE ............. 3. TIME ............
4. DAM NAME ........................................................................................................
5. PROBLEM
  5.1 Description ........................................................................................................
  .............................................................................................................................
  5.2 Specific Area of Problem at Site ........................................................................
  .............................................................................................................................
6. UNUSUAL FLOW
  6.1 Quantity ..............................................................................................................
  6.2 Quality ...................................................................................................................
  .............................................................................................................................
7. WATER LEVEL
  7.1 Reservoir level ......................................................................................................
  7.2 Rising/Stable/falling .............................................................................................
  .............................................................................................................................
8. SITUATION
  8.1 Improving/stable/worsening/containable .............................................................
  8.2 Other Comments ..................................................................................................
  .............................................................................................................................
9. GENERAL COMMENTS
  9.1 Weather conditions .............................................................................................
  .............................................................................................................................
  .............................................................................................................................
10. NOTIFICATION CHECKLIST (tick when done, record time and name)

   Waimea Water   Time .............   Person Contacted .............
   New Zealand Police   Time .............   Person Contacted .............
   Nelson Tasman CDEM   Time .............   Person Contacted .............
   Ambulance   Time .............   Person Contacted .............
   Fire and Emergency New Zealand  Time .............   Person Contacted .............
   Dam Safety Consultant (Tonkin & Taylor Ltd)  Time .............   Person Contacted .............

SIGNATURE ...........................................................................................................
Distribution: Dam Owner, New Zealand Police, Nelson Tasman CDEM
Appendix D: Contact List

Draft 2

14 August 2018
Waimea Dam - Dam safety and emergency contract list

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Name/ Position</th>
<th>Phone</th>
<th>After hours phone</th>
<th>Email address for documentation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DAM OWNER &amp; OPERATOR</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waimea Water</td>
<td>(Dam Owner)</td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>EMERGENCY SERVICES (POLICE, FIRE &amp; EMERGENCY, AMBULANCE) – DIAL 111 IN AN EMERGENCY</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Zealand Police</td>
<td>Emergency</td>
<td>111</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tasman District Control Room</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Wakefield Police Station</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nelson Tasman CDEMG</td>
<td>Group Civil Defence Duty Officer</td>
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<tr>
<td>Ambulance</td>
<td>Emergency</td>
<td>111</td>
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<tr>
<td></td>
<td>St John’s Ambulance</td>
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<tr>
<td>Fire and Emergency New Zealand</td>
<td>Emergency</td>
<td>111</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>TECHNICAL ADVISORS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Dam Safety Consultant</td>
<td></td>
<td></td>
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<tr>
<td>Tonkin &amp; Taylor Ltd (Designer)</td>
<td></td>
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<tr>
<td><strong>OTHER RELEVANT STAKEHOLDERS</strong></td>
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<tr>
<td>Tasman District Council</td>
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</tbody>
</table>
Appendix E: Sources of Equipment and Materials

Draft 2
14 August 2018
**Earthmoving equipment**

Earthmoving equipment required can be sourced from these suppliers:

<table>
<thead>
<tr>
<th>Company</th>
<th>Address</th>
<th>Telephone</th>
</tr>
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<tbody>
<tr>
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</tbody>
</table>

**Supplies and materials**

Riprap, sandbags and other construction materials can be sourced from the following local suppliers.

<table>
<thead>
<tr>
<th>Company</th>
<th>Address</th>
<th>Telephone</th>
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<tbody>
<tr>
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</table>

**Generator**

If required, a generator can be sourced from:

<table>
<thead>
<tr>
<th>Company</th>
<th>Address</th>
<th>Telephone</th>
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**Dewatering pumps**

Dewatering pumps can be sourced from:

<table>
<thead>
<tr>
<th>Company</th>
<th>Address</th>
<th>Telephone</th>
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</tbody>
</table>

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14 August 2018
Appendix G: EAP Testing Forms

Draft 2

14 August 2018
**WAIMEA DAM EMERGENCY ACTION PLAN**

**TEST REPORT**

**TO:** Operations & Maintenance Coordinator  
Tasman District Council  
Private Bag  
NELSON

**FROM:**

**DATE:**  
**FILE:**

**TYPE OF TEST:**
- [ ] Limited Communications Test
- [ ] Full Communications Test
- [ ] Operational Test

_(Please tick type of test)_

**TIME AND DATE FIRST NOTIFIED:**

**NOTIFIED BY:**

**NOTIFICATIONS MADE:**

**NAME** | **AGENCY(IES)** | **TIME OF CALL**
---|---|---

**MESSAGE RECEIVED/PASSED ON**

---

**COMMENTS ON TEST:**

---

_(If space is insufficient use additional sheet(s))_

---

**Signature**
WAIMEA DAM EMERGENCY ACTION PLAN

RECORD OF BRIEFING SESSIONS AND FULL COMMUNICATIONS AND OPERATIONAL TESTS

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Initiated by</th>
<th>Description</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tbody>
</table>

Draft 2

14 August 2018
The Modified Mercalli Intensity Scale*

<table>
<thead>
<tr>
<th>Category</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>MM 1: Imperceptible</td>
<td>Barely sensed only by a very few people.</td>
</tr>
<tr>
<td>MM 2: Scarcely felt</td>
<td>Felt only by a few people at rest in houses or on upper floors.</td>
</tr>
<tr>
<td>MM 3: Weak</td>
<td>Felt indoors as a light vibration. Hanging objects may swing slightly.</td>
</tr>
<tr>
<td>MM 4: Largely observed</td>
<td>Generally noticed indoors, but not outside, as a moderate vibration or jolt. Light sleepers may be awakened. Walls may creak, and glassware, crockery, doors or windows rattle.</td>
</tr>
<tr>
<td>MM 5: Strong</td>
<td>Generally felt outside and by almost everyone indoors. Most sleepers are awakened and a few people alarmed. Small objects are shifted or overturned, and pictures knock against the wall. Some glassware and crockery may break, and loosely secured doors may swing open and shut.</td>
</tr>
<tr>
<td>MM 6: Slightly damaging</td>
<td>Felt by all. People and animals are alarmed, and many run outside. Walking steadily is difficult. Furniture and appliances may move on smooth surfaces, and objects fall from walls and shelves. Glassware and crockery break. Slight non-structural damage to buildings may occur.</td>
</tr>
<tr>
<td>MM 7: Damaging</td>
<td>General alarm. People experience difficulty standing. Furniture and appliances are shifted. Substantial damage to fragile or unsecured objects. A few weak buildings are damaged.</td>
</tr>
<tr>
<td>MM 8: Heavily damaging</td>
<td>Alarm may approach panic. A few buildings are damaged and some weak buildings are destroyed.</td>
</tr>
<tr>
<td>MM 9: Destructive</td>
<td>Some buildings are damaged and many weak buildings are destroyed.</td>
</tr>
<tr>
<td>MM 10: Very destructive</td>
<td>Many buildings are damaged and most weak buildings are destroyed.</td>
</tr>
<tr>
<td>MM 11: Devastating</td>
<td>Most buildings are damaged and many buildings are destroyed.</td>
</tr>
<tr>
<td>MM 12: Completely devastating</td>
<td>All buildings are damaged and most buildings are destroyed.</td>
</tr>
</tbody>
</table>

*Adapted from GNS Science New Zealand website, September 2012
Appendix I: Review and revision records

Draft 2
14 August 2018
<table>
<thead>
<tr>
<th>Agency</th>
<th>Date Reviewed</th>
<th>Remarks</th>
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<tbody>
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14 August 2018

Draft 2
Detailed design peer review comments and responses

The peer review comments were received from Opus as part of the Stage 4 design and as summarised in the Stage 4 Peer review Design Review Producer Statement (PS2) schedule titled “Waimea Dam – Stage 4 Design Peer Review”.

The following peer review comments were received from EGL as part of the Stage 4 design as relates to their role as reviewer for the site specific seismic hazard assessment undertaken by GNS, and determination of vertical spectra, time history record selection and scaling by T+T.

<table>
<thead>
<tr>
<th>Item</th>
<th>EGL peer review comment</th>
<th>T+T response</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>The procedure for determining vertical spectra is OK.</td>
<td>Noted.</td>
</tr>
<tr>
<td>b</td>
<td>The time-histories were selected in 2011, and we have no problem with what has been selected. However, have you considered using any earthquakes recorded since 2011? (e.g. Cook Strait, Grassmere or Kaikoura)</td>
<td>We considered use of alternative earthquake records such as the Kaikoura, Grassmere and Cook Strait events, but given the GNS 2011 records were confirmed to still be applicable (in terms of the updated PSHA using procedure for selecting ground motions in NZS1170.5) we continued with them.</td>
</tr>
<tr>
<td>c</td>
<td>The procedure for scaling the horizontal time-histories is appropriate.</td>
<td>Noted.</td>
</tr>
<tr>
<td>d</td>
<td>Do you intend to use vertical time-histories in analyses? We are not sure how the estimates of vertical ground motion are to be used in any analyses. If vertical time-histories are required we assume they will be the vertical components associated with the records already selected. How will they be scaled? Same factors as for horizontal or derive new factors dependent on vertical period of interest?</td>
<td>We have used vertical time histories in the dynamic analyses for the CFRD embankment, primarily as a sensitivity check. We used the vertical components associated with the already selected ground motions. We scaled them using the same factors as for the horizontal (derived from the NZS1170.5 procedure), and then multiplied them by the V/H=0.9 factor.</td>
</tr>
</tbody>
</table>
The following peer review comments were received from Opus as part of the T+T report Lee Valley Dam, Response to Peer Review of Stage 3 Design, December 2013, and subsequent correspondence throughout the Stage 4 detailed design process.

<table>
<thead>
<tr>
<th>Item</th>
<th>Opus peer review comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref ID 010 – Adopted design criteria</td>
<td>While the review matters previously raised have been substantially addressed in the supplied 3 design documentation, and while it is recognised that this documentation represents 80% detailed design development, there are still some aspects of the adopted performance criteria that could benefit from further consideration and/or elaboration. In some instances this may simply be to give a clear direction to the completion of stage 4 design detail and the drafting of associated construction specifications. The adopted construction diversion flood capacity now clearly defined at a 1 in 1000 AEP event is supported, noting this capacity relates to the combined culvert discharge plus controlled overtopping via reinforced rockfill. The classification of the access bridges in terms of their significance to the ongoing safe operation of the impoundment and/or their role in facilitating a response to a dam safety incident could benefit from further elaboration and justification. Superficially at least it could be argued that functionality of one or both of the bridges could be required to safely manage the impoundment, and as such the performance criteria may need to be higher than that currently adopted through the adopted importance level rating. This matter may affect the detailing of such aspects as the seismic linkages / restraints yet to be designed.</td>
</tr>
</tbody>
</table>

<p>| | T+T response in December 2013 | Further T+T response for Stage 4 design |
| | T+T agrees that the bridges are critical for access and this shall be elaborated in the Stage 4 report. The critical nature of the bridges will be reflected in the design of seismic linkages and restraints and Stage 4 design drawings. | We have included seismic linkage details to restrain the bridge in the event of large earthquakes up to and including the SEE. Refer to Stage 4 T+T Drawings (BRG Drawing pack) 27425-BRG-140, BRG-145 &amp; BRG-152. |</p>
<table>
<thead>
<tr>
<th>Item</th>
<th>Opus peer review comment</th>
<th>T+T response in December 2013</th>
<th>Further T+T response for Stage 4 design</th>
</tr>
</thead>
<tbody>
<tr>
<td>f</td>
<td>A more detailed consideration regarding the performance criteria for the bridges concerns the side impact barrier systems. We presume the slow speed environment enables the normal lateral deflection criteria to be reduced, but this matter is not specifically covered in the documentation.</td>
<td>More detailed elaboration of the performance criteria for the bridge side impact barrier systems will be provided in the Stage 4 documentation.</td>
<td>We have included specification of standard CSP type guard rails. Refer to Stage 4 T+T Drawings (BRG Drawing pack) 27425-BRG-144, BRG-148 &amp; BRG-150.</td>
</tr>
<tr>
<td>g</td>
<td>It is common practice to rate bridge capacity in terms relative to normal highway standards that will be readily understood by transport operators. The usual classification method refers to % of class I loading where class I is equivalent to 85% of the HN design loading component of the current HN-HO-72 loading. We recognise that these bridges may reasonably be designed for loadings below full highway standard, but the structures should still be identified using standard bridge design methodology and posting terminology. Appropriate permanent signage should also be installed.</td>
<td>T+T agrees that appropriate signage should be installed. Signage requirements will be included in the construction contract documentation. We do not necessarily agree that they should be related to a class or percentage of the Transit Bridge Manual, instead we propose to include signage stating the maximum axle load.</td>
<td>Construction Issue drawings will include a note requiring the installation of signs stating the maximum axle design load of 8.2 tonnes (Dwg BRG-100 and 101).</td>
</tr>
</tbody>
</table>

**Ref ID 012 – Potential hydropower scheme**

Progress with design of the outlet works and hydropower scheme design has addressed many of the earlier review points, but the finalisation of the dam design is still subject to the decision to include or allow for future hydropower development. Subject to the manner in which the two level intake system is expected to be routinely operated, the penstock sizing for hydropower might be open to further design optimisation. Peak operating velocities would become quite high for reliance on individual supply penstocks under full generation load. Furthermore, the selection of terminal discharge valve type might be open to savings if these facilities were only

The hydropower option has been taken to feasibility design level only and further work is still necessary during Stage 4 to complete the mechanical and electrical aspects of the dam design. On 30 August 2013 WWAC advised T+T to put the mini-hydro on hold and exclude the mini-hydro from being progressed to detail design. However WWAC still wish that the provisions for the mini-hydro are provided for, including the possibly of considering this as potential design build option as part of the dam construction. During Stage 4 design T+T will discuss this matter further with WWAC and seek agreement on the point at which they would want to be able to add on the hydro. This may be at the downstream end of the concrete conduit or close to the valves at the upstream end. This issue is likely to be considered further if the mini hydro option is to be pursued by

The hydropower scheme remains on hold. Waimea Water’s instruction was to allow for provisional future installation of a hydropower scheme, and this has been considered in the layout at the dam toe and the penstock design by WSP which allows for future hydropower generation (refer WSP Penstock)
<table>
<thead>
<tr>
<th>Item</th>
<th>Opus peer review comment</th>
<th>T+T response in December 2013</th>
<th>Further T+T response for Stage 4 design</th>
</tr>
</thead>
<tbody>
<tr>
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<td>to be used infrequently when generation flow was not possible. The matter of the integration of the twin intake pipelines with the hydropower manifold such that controlled mixing is achieved without headloss compromising expected generation output is yet to be effectively established.</td>
<td>WWAC. Further analysis of the potential hydropower scheme is outside T+T’s scope of work at this stage.</td>
<td>Hydraulic and Mechanical Design reports).</td>
</tr>
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</table>

Ref ID 013 – Site hydrology

The Opus Interim Report Number 07, dated 7 June 2013 identifies this Peer Review Topic as complete. The topic related primarily to various hydrology issues previously raised and addressed in or prior to completion of Stage 3.

Ref ID 014 – Diversion concept

The following overview comments relate primarily to the responses provided to the various specific hydrology issues raised in the previous stage of this review.

Table 9.3 Responses to Opus hydrology peer review are generally appropriate. The incremental risk analysis approach is supported along with the manner of handling diversion flood risk during the progressive development of the embankment.

The potential for partial blockage of the diversion culverts from forestry debris is present. Overtopping of the downstream reinforced rockfill face applying during the critical intermediate stages of the embankment construction may mitigate this risk to some extent. The likelihood of floating or semi – submerged debris creating significant blockage once the quick rise berm is in place is probably low, but it does remain a residual risk.

The specific design of the rockfill face reinforcement and the resultant level of security against erosion during major diversion flood handling is a critical aspect of the design. The manner in which this debris risk to the diversion and the erosion risk to the reinforced rockfill are to be

Contract documentation will draw attention to the risk debris poses to the diversion strategy and will require debris control measures to be considered and addressed by the Contractor.

The downstream reinforced rockfill (meshing) design adopted and described in the Stage 3 design report incorporates design features based on ICOLD guidelines, research and construction experience over the last 50 years.

The Stage 3 meshing design has been informed by evaluating the critical flow rate for unravelling of the downstream face rockfill (resulting from flood events during construction) using published techniques as well as carrying out seepage and downstream face stability analyses. However, we note that ICOLD Bulletin No. 89 “Reinforced Rockfill and Reinforced Fill for Dams”, 1993, states that the effects of debris are not readily subjected to rational analysis. Hence knowledge of surface mesh protection systems which have survived actual operating conditions currently provides the best basis for design. The Stage 3 design incorporates published recommendations stemming from research into the impacts of debris on reinforced rockfill and incorporates review comments by Len McDonald who is a co-author of ICOLD Bulletin No. 89.

River diversion provisions and design are covered by FHTJV and their temporary diversion works designers (GHD) for Stage 4 (refer GHD design documentation).

A summary of how the proposed river diversion works have been considered for the permanent works is given in the T+T Stage 4 Design report Sections 7, 9 and 26.2.
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<td>addressed during stage 4 / Contractor design remains an important consideration for the project, and there is an expectation of specific debris control measures being required and reinforcement performance being confidently established.</td>
<td>We believe the meshing design described in the Stage 3 design documentation, combined with the conservative flood diversion standard adopted establishes an adequate level of confidence in the safe handling of predicted diversion flood flows. To increase the level of confidence in the downstream reinforced rockfill we propose that the Contract Documentation will require regular evaluation and review of the meshing by the Engineer during construction.</td>
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<td>Ref ID 015 – Spillway discharge capacity</td>
<td>We endorse the change in philosophy from a primary uncontrolled spillway and a fusible secondary spillway to the single larger capacity uncontrolled spillway. The ability to make use of physical hydraulic model study data for a similar spillway on the proposed Tillegra Dam in New South Wales to inform the design of the spillway for the Lee Valley Dam is very helpful. The general layout and geometry of the single uncontrolled spillway appear appropriate. We note that Figures 15-1, 15-2 and 15-3 clearly demonstrate that the proposed dam will cause minimal attenuation of floods passing through the reservoir impounded by the dam. This is not surprising given the geometry of the reservoir. However this is a key point that needs to be communicated to the client and stakeholders to dispel any (incorrect) public perception that the presence of a dam upstream will reduce the flood risk downstream.</td>
<td>The final report will clearly communicate the level of attenuation provided and state that the purpose of the dam is not intended to reduce the flood risk downstream.</td>
<td>Refer Section 17.4 of T+T Stage 4 detailed design report for flood routing. Dam purpose is clearly stated in the executive summary and Section 1.</td>
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<td>Section 15.5.1 refers to a “computed aerated water surface profile”. However no indication is given as to how self-aeration of the water surface profile down the spillway chute has been calculated, nor what the profiles are for the OBF and the MDF and how these profiles influence lining height of the chute will also be more clearly communicated.</td>
<td>The final report will state more clearly how self-aeration of the water surface profile down the spillway chute has been calculated and provide OBF and MDF profiles accounting for self-aeration. How these factors influence lining height of the chute will also be more clearly communicated.</td>
<td>Refer Section 17.7 of T+T Stage 4 detailed design report for spillway chute hydraulics. The water surface profile and height of</td>
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<td>influence the design lining height for the chute. It is important to recognise that self-aeration of spillway flow is not able to be reproduced in a physical hydraulic model study due to scale effects so any water surface profiles from such a source will underestimate the corresponding water surface profiles in the prototype structure. Some clarity on this aspect of the design would be helpful.</td>
<td></td>
<td>chute walls was also reconsidered as part of Stage 4 and the wall height increased in the 10% slope and downstream transition section to provide additional freeboard in this area.</td>
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**Ref ID 016 – Spillway robustness**

<p>| c | The assessment of the potential for cavitation on the spillway chute in Section 15.5.2 relies on very simplistic cavitation thresholds based on flow velocity and discharge per unit width. The magnitude of the flow velocities down the spillway chute under the OBF (26 m/s) implies that the hydraulic performance of the chute is on the margins of where the risk of cavitation damage starts to sharply increase. If the maximum flow velocity for the OBF is 26m/s, then the maximum flow velocities for more extreme floods up to the MDF (PMF) would be higher still and the cavitation risk higher accordingly. It would be appropriate to undertake a more rigorous analysis of cavitation risk. Best practice for cavitation risk assessment follows the approach of the USBR Engineering Monograph No. 42 Cavitation in Chutes and Spillways in which the cavitation index of the flow is compared with the incipient cavitation index for typical irregularities. In the case of the spillway chute for the Lee Valley Dam, the irregularities that should be checked using a cavitation index approach include joint offsets, surface roughness irregularities and sharp changes in invert and side wall slope. | We agree that an analysis of cavitation using methods presented in USBR Engineering Monograph No. 42 is appropriate. Analysis carried out to date using EM42 indicates cavitation is unlikely to be a problem however this will be finalised and reported on in Stage 4. | Refer Section 17.7.4 of T+T Stage 4 detailed design report for spillway chute cavitation assessment summary which concludes that the potential for cavitation is likely to be low with consideration of joint irregularities, surface roughness and chute geometry changes. |
| d | Section 15.7 discusses the design of the plunge pool to absorb the jet from the flip bucket at the end of the | We will add further discussion on the flip bucket trajectories into the Stage 4 report. The length of the trajectory of the jet is 1.4 m shorter | Refer Section 17.9 of T+T Stage 4 detailed design report for spillway chute cavitation assessment summary which concludes that the potential for cavitation is likely to be low with consideration of joint irregularities, surface roughness and chute geometry changes. |</p>
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<td>spillway chute. The discussion notes that the effective lip angle of the flip bucket and the air resistance parameter in Kawakami’s (1973) method were changed to replicate the jet trajectories measured in the physical hydraulic model of the Tillegra Dam Spillway. However jet trajectories from a flip bucket in a spillway physical model cannot account for the breakup of a prototype jet by air resistance and air entrainment so that prototype jet trajectories will differ from model ones. The difference in jet trajectories between the model and prototype should be estimated for the OBF and MDF to ensure that all conclusions made on the basis of the physical model jet trajectory interpolation / extrapolation remain valid.</td>
<td>for the OBF (3% reduction) and 8.3 m shorter for the MDF (11% reduction) if the original Kawakami coefficient of air resistance is adopted rather than the modified coefficients based on the physical model results. Our assessment that the scour resulting from flip bucket action is not expected to affect the flip bucket and dam embankment stability remains valid even if the shorter trajectory lengths are adopted. This will be elaborated on in our Stage 4 report.</td>
<td>report for plunge pool details and consideration of scour due to hydraulics loads.</td>
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<td>It would be helpful to list the key design parameters for the flip bucket and plunge pool; • Jet take-off velocity vo • Jet take-off head Ho • Flip bucket flow depth tb • Jet take-off Froude number Fro</td>
<td>The design parameters selected based on water profile and jet trajectory measurements from the physical model study will be presented in the Stage 4 report and are as follows: OBF: • Velocity - 24.0 m/s (at lip of flip bucket) • Flow depth - 2.0 m (at lip of flip bucket) • Effective lip angle - 34° • Froude number - 8.3 (at entry to flip bucket) (n=0.014 model) MDF: • Velocity - 27.3 m/s (at lip of flip bucket) • Flow depth - 3.4 m (at lip of flip bucket) • Effective lip angle - 36° • Froude number - 6.2 (at entry to flip bucket) (n=0.014 model)</td>
<td>Refer Sections 17.8 and 17.9 of T+T Stage 4 detailed design report for flip bucket and plunge pool details.</td>
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<td>The sidewalls of the spillway flip bucket are strongly convergent to assist in concentrating the spillway jet. However there is no discussion of the effect of this sidewall convergence on the flip bucket jet trajectory. Despite the concentration of the jet by the converging flip</td>
<td>Referring to the CAD screen shot below, the flip bucket shape is derived by sweeping the trapezoidal chute cross section about the axis that defines the centre of the flip bucket radius. Thus the supercritical flow should ‘see’ an approximately constant cross</td>
<td>Refer to Section 17.9 of T+T Stage 4 detailed design report for further discussion on the plunge pool scour extent uncertainties and</td>
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<td>bucket sidewall, the jet will start to spread laterally due to the effects of air resistance and air entrainment. No account of divergence of the prototype jet due to these effects appears to have been taken in the plunge pool design. Lateral spread of the prototype jet through the air could cause it to have a wider spread than assumed for the plunge pool design. This has implications for the lateral dimensions of the plunge pool.</td>
<td>sectional area as it passes through the flip bucket. Thus, the sidewalls are not convergent, though they do appear so in plan.</td>
<td>approach as discussed with Waimea Water, FHTJV and WSP Opus. We have also included a 3 m deep concrete cutoff at the termination of the concrete facing to provide additional mitigation against scour undermining.</td>
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|      | The physical model study indicates that the jet from the flip bucket is likely to be characterised as follows:  
- The edges of the jet have less momentum and fall to the plunge pool earlier than the main body of the jet.  
- The top of the jet is characterised by two rooster tails, especially at higher flow rates. The presence of these rooster tails influences the impact distribution on the plunge pool floor as the bulk of the water is concentrated within each rooster tail, thus concentrating the impact on the plunge pool floor.  
The photographs following illustrate the shape of the jet from the physical model study for Tillegra for the highest flow rate considered for that study (1495 m³/s prototype, significantly higher than the MDF for Lee Valley Dam). | | |

The photographs following illustrate the shape of the jet from the physical model study for Tillegra for the highest flow rate considered for that study (1495 m³/s prototype, significantly higher than the MDF for Lee Valley Dam).
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Although lateral spreading is anticipated (maximum width of 31 m approximately for OBF and 39 m approximately for MDF), the impact width of the core of the jet, which contains the bulk of the flow, is expected to be smaller (approximately 15 m for OBF and 18 m for MDF). Although the impact width of the core of the jet is larger than the pre-excavated base of the channel, it is substantially less than the width of the pool at the water surface. The maximum width of the jet is expected to be slightly larger than the width of the pool at the water surface.

As noted in the report, it is likely that scour beyond pre-excavation extents could occur for the mean annual flood, and scour beyond pre-excavation extents is highly likely for the larger events (OBF and PMF) – this additional scour relates to depth as well as lateral extent. However, the additional scour is considered acceptable since it is not expected to affect the flip bucket and dam embankment stability, though it could potentially affect the left side cut slope immediately adjacent and the in situ right bank slope located downstream of the pre-excavated pool.

Rather than enlarging the pre-excavation extents (increased construction cost) we propose that an observational approach is adopted whereby maintenance is carried out as required. Further discussion will be incorporated into the Stage 4 documentation to clarify the plunge pool design approach and...
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<td>ensure the “Foundation Committee” confirms the suitability of the plunge pool rock for the proposed observational approach.</td>
<td>Refer Section 5.5 of T+T Stage 4 detailed design report for further discussion on seiche. Refer Section 29 for updated references list.</td>
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<td>Ref ID 017 – Response of reservoir to dynamic disturbance</td>
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<td>Reservoir Seiching:</td>
<td>Further discussion on seiching will be added into the Stage 4 report to address this point and missing references will be added.</td>
<td>Refer Section 5.4 of T+T Stage 4 detailed design report for further discussion on landslide waves. The modelling approach for the landslide waves means that the wave height is not sensitive to landslide velocity. Refer Section 5.2 of T+T Stage 4 detailed design report for further discussion on wave runup. We have chosen not to include some of the suggested landslide characteristics because</td>
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<td>Although it is hinted that the magnitude of seiche waves induced by rupture of a nearby earthquake fault is likely to be small, the discussion in Section 6.3 in respect of this matter is not closed off conclusively. While we concur that any seiche waves generated in this manner are likely to be small, we suggest that the discussion is closed off more conclusively by simple estimation of the likely magnitude of seiche waves induced by nearly earthquakes. The references Murty (1979) and Synolakis and Uslu (2003) are missing from the list of references at the end of the report.</td>
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<td>Landslide Generated Waves:</td>
<td>Further discussion on landslide generated waves will be added into the Stage 4 report to address the matters raised and the requested information added. We also confirm that wave runup on the dam face has been accounted for.</td>
<td>Refer Section 5.4 of T+T Stage 4 detailed design report for further discussion on landslide waves. The modelling approach for the landslide waves means that the wave height is not sensitive to landslide velocity. Refer Section 5.2 of T+T Stage 4 detailed design report for further discussion on wave runup. We have chosen not to include some of the suggested landslide characteristics because</td>
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<td>It is noted in Section 6.4.1 that a landslide velocity of 19 m/s was selected on the basis of information presented in a companion report. We would anticipate that the expected landslide velocity for landslide scenarios 1 and 2 would have a possible range rather than a single definitive value. How sensitive are the predicted impulse wave heights to the range of likely landslide velocities? It would be appropriate to list in the design report the input parameters (landslide mass, landslide angle, landslide velocity, reservoir depth and landslide width etc.), predicted impulse wave heights at the landslide source and the attenuated wave heights at the site of the dam for each landslide scenario. It is not clear from the description in Section 6.4 if the incident heights of the attenuated impulse waves take</td>
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Tonkin & Taylor Ltd
Waimea Dam - Stage 4 Detailed Design Report
Waimea Water

October 2018
Job No: 27425.100
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<td>account of wave runup on the sloping face of the dam. Wave runup on the dam face should be taken account of.</td>
<td>In modern CFRDs it is common practice to use an internal plinth extension to reduce upstream excavations and enable construction of a constant width external plinth. Both ICOLD Bulletin 141 (2010) “Concrete Face Rockfill Dams Concepts for Design and Construction” as well as Cruz P., Materon B. and Freitas M. (2010), “Concrete Face Rockfill Dams” provide discussion and coverage of the matter. We understand that many dams have adopted this approach and for example include several Brazilian Dams (Itá, Machadinho, Monjolinho, Barra Grande, Campos Novos) as well as Babagon Dam, Malaysia; Caracoles, Argentina; Bakun, Malaysia; Merowe, Sudan, and Berg River, South Africa. Based on our telephone conversation with Ian Walsh on 8 August 2013, we understand that the reviewer’s concern is not with the concept of a downstream plinth extension in itself, but rather with the fact that the downstream portion is not anchored to the rock in a similar manner to the external portion. The recommended procedure in ICOLD Bulletin 141 is to combine the width of both the external and internal plinth to evaluate the hydraulic gradient across the plinth. The width of the external doweled plinth section is defined by the need for a practical grouting platform to construct a three-row grout curtain, while the internal slab width supplements the requirements for the allowable hydraulic gradient through the foundation. The purpose of the dowelling is to hold down the plinth such that it can act as a grout cap during the grouting exercise, not to hold down the plinth against long term uplift pressures from the reservoir. ICOLD Bulletin 141, quoting ICOLD Bulletin 70, states that dowels anchoring the plinth to</td>
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<td>these are shown on the landslide map which we consider provides reasonable indication of size location and extent.</td>
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Ref ID 018 – Seepage control treatment costs

While the plinth and associated foundation preparation and treatment approaches are conventional for CFRD developments, there are still many details to be finalised in stage 4. The target maximum hydraulic gradient is not fully satisfied with the nominal plinth footprint dimension, and appears to be based upon reliance on the downstream shotcrete extension. This approach is less robust than providing an anchored plinth contact of the requisite length. While the classification of the exposed rock mass during construction can reasonably be applied to determining the plinth footprint, the approach appears to be ambiguous with regard to the limiting hydraulic gradient adopted for design. The plinth design was refined further as part of the Stage 4 design process and includes dowels on the downstream extension slab where required to meet the target hydraulic gradients as summarised in Section 11.2.3 of the T+T Stage 4 detailed design report and shown on Dwgs 27425-PNH-100 to 155.
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<td>The limiting permeability target of 6 lugeons for the upper 15 m of the grout curtain presented in section 7.4.1 is noted, but there is no explicit discussion correlating this target with the expected seepage flow rates and the resultant relationship to the internal drainage performance of the embankment as adopted for design.</td>
<td>The limiting permeability target of 6 Lu is selected based on the recommendations in Houlsby (1990) “Construction and Design of Cement Grouting” and in ASCE (2007) “Dam Foundation Grouting”. We will provide additional text in the Stage 4 design report to clarify how the target correlates with the expected seepage flows based on seepage modelling carried out during Stage 3 design.</td>
<td>Refer Section 12 of T+T Stage 4 detailed design report and Section 5 of the Civil and Dam Works Specification for further discussion on grouting.</td>
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<td>We recognise the nature of grouting to be very responsive to progressive information obtained on grout takes and water test results during construction. However, given the extent of subsurface foundation characterisation now obtained, the grouting methodology and means of verifying effectiveness could usefully be elaborated on with a view to giving direction to future drafting of the technical specifications. Drilling flushing and water test sequences, initial grout mix, pressure targets and expected takes could all be described for this significant element of the works, noting the mitigation of risks associated with rock mass dilation/ hydraulic jacking on the steep abutments etc. Although only 5m deep, the design grouting layout presented appears to be almost more in the nature of a triple row curtain rather than blanket (or consolidation) grouting, so the design report would benefit from elaboration on the designers</td>
<td>Further elaboration on matters pertaining to grouting will be provided in either the Stage 4 documentation or construction contract documentation.</td>
<td>Refer Section 12 of T+T Stage 4 detailed design report and Section 5 of the Civil and Dam Works Specification for further discussion on grouting.</td>
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<td>expectations concerning the need for confining the curtain grout takes through pregrouting the outer rows, and the risk of needing shallow consolidation grouting beyond the plinth / starter dam footprint.</td>
<td>Further elaboration on matters raised pertaining to the plinth and its geometry will be provided in the Stage 4 documentation. Based on published information we expect that grouting pressure in the order of 100-200 kPa at the top of the hole, increasing at approximately 25 kPa per metre of depth will be appropriate. Confirmation that the doweled plinth can adequately resist the grout uplift pressures will be covered in the Stage 4 report. We acknowledge that it is appropriate to verify the amount of thermal and shrinkage crack control reinforcement presented in the Stage 3 drawings against NZS3101 code requirements. This will be done and reported on in Stage 4 and should additional reinforcing be required drawings will be modified as necessary. Cruz P et al. (2010) advise that for practical reasons, in modern CFRDs transverse contraction joints are no longer incorporated along the plinth (unless differential settlements are a concern), however they are likely to be required at cold joints locations. With this in mind they may be necessary in the vicinity where the plinth transitions from the top of the starter dam to the abutment. Stage 4 documentation is expected to require that PVC waterstops be incorporated into plinth contraction joints.</td>
<td>Refer Sections 11 and 12 of T+T Stage 4 detailed design report and Section 5 of the Civil and Dam Works Specification for further discussion on the plinth design and consideration of grouting uplift pressures. The plinth and doweled structural design specifically considered the uplift pressures due to grouting. There are no contraction joints in the plinth with continuous reinforcement provided and construction joints. Horizontal water stops are not required at the construction joints. Further specific details of the plinth geometry and transitions are shown on Dwgs 27425-PNH-100 to 155.</td>
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**Ref ID 019 – Rockfill properties**

<p>| a    | While the trial embankment findings are preliminary in nature, the low fines production appears to be inconsistent with the conservative degree of permeability | Noted that this matter is not a reviewer concern. Consideration will be given to the benefit of incorporating further discussion on rock handling aspects in Stage 4 documentation. | Refer Sections 13 and 14 of T+T Stage 4 detailed design report and Section 3 of the |</p>
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<td>anisotropy adopted. This conservative position is not a review concern per se’ but the design report would benefit from further discussion on rock handling aspects leading to the future draughting of the technical specification and giving direction to the construction phase trials referred to, such as:</td>
<td>Civil and Dam Works Specification for further discussion on embankment properties and how these have been considered in the stability modelling. Additional embankment trials undertaken in early 2018 have also informed the presented Stage 4 design and the Contractor’s construction methodology.</td>
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<td>- Lift height vs compaction plant characteristics with a view to optimising density with minimum crushing and fines production leading to layering and permeability anisotropy. We note that a we are of the view that use of heavy vibrating rollers in excess of the 10 tonne capacity are likely to produce enhanced outcomes.</td>
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<td>- The potential for sluicing during compaction of these rockfill products (not gravels) to enhance performance parameters through softening of particle contacts and redistributing fines through the lift.</td>
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<td>Ref ID 020 – Seismically induced embankment deformations</td>
<td>While the order of expected deformations is clearly presented in the design report, including the potential after shock condition, the report would benefit from elaboration on the relationship of the rockfill material characteristics to strain related behaviour. Specifically the degree of conservatism and the sensitivity to assumptions for the following:</td>
<td>For the purpose of design it has been necessary to make assumptions about the characteristics of the rock fill based on geotechnical investigations and laboratory testing to date. Conservative assumptions have been made in the analyses to provide results that represent the lower bound region of a sensitivity range. Rockfill strength parameters have been derived with reference to Barton and Kjaernsli (1981) with parameters initially set at expected values. The expected values were assessed from the site investigation and test data. For stability and deformation modelling, the expected values have been discounted to provide an assessed lower bound strength envelope. The UCS was discounted by a factor of 0.75, the porosity increased by 1.25, and the dry density factored by 0.9 to provide this assessed lower bound.</td>
<td>Refer Section 14 of T+T Stage 4 detailed design report for further discussion on embankment properties and how these have been considered in the stability modelling including strains and sensitivity analyses.</td>
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<td>- What is the sensitivity to adopted moduli values?</td>
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<td>- What is the potential for strain softening and /or post peak strength reduction of the coarse rockfill zones under “large” deformation strains relative to the yield strength parameters adopted for analysis? i.e. to what degree is the interlock/dilation component being relied upon for determination of first yield, and how sensitive is the deformation analysis to loss of this</td>
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<td>strength under “large” strain such as the aftershock case?</td>
<td>A maximum shear modulus of 470MPa has been adopted for the rockfill (approximately equivalent to a shear wave velocity of 485 m/s) which represents the lower bound of values for well compacted rockfill materials referenced by Materon (2011) of 457 to 610 m/s. By combining lower bound strength and stiffness values in analyses for this stage we have not further degraded the values when considering aftershock analyses. For the aftershock analyses we have however assumed rupture of the concrete face, and assessed the embankment stability and further displacement with full flow through. For full flow through we have conservatively adopted high anisotropy in embankment permeability characteristics which results in a substantially saturated embankment. This is considered a significantly conservative case without further degrading material strength characteristics. Stage 4 and Construction contract documentation will require that the Designer review and confirm that the rockfill material characteristics encountered during construction are as envisaged during design. References: Shear Strength of Rockfill, Barton and Kjaernsli (1981). Considerations on the Seismic Design of High Concrete Face Rockfill Dams (CFRDs), Materon and Fernandez, 2011.</td>
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<td>- What is the potential for the finer fill zones to exhibit limited compaction related interlock/dilation strength relative to conventional more coarse CFRD experience, and thereby require more conservative strength parameters to be adopted for large deformation response prediction?</td>
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<td>A maximum shear modulus of 470MPa has been adopted for the rockfill (approximately equivalent to a shear wave velocity of 485 m/s) which represents the lower bound of values for well compacted rockfill materials referenced by Materon (2011) of 457 to 610 m/s. By combining lower bound strength and stiffness values in analyses for this stage we have not further degraded the values when considering aftershock analyses. For the aftershock analyses we have however assumed rupture of the concrete face, and assessed the embankment stability and further displacement with full flow through. For full flow through we have conservatively adopted high anisotropy in embankment permeability characteristics which results in a substantially saturated embankment. This is considered a significantly conservative case without further degrading material strength characteristics. Stage 4 and Construction contract documentation will require that the Designer review and confirm that the rockfill material characteristics encountered during construction are as envisaged during design. References: Shear Strength of Rockfill, Barton and Kjaernsli (1981). Considerations on the Seismic Design of High Concrete Face Rockfill Dams (CFRDs), Materon and Fernandez, 2011.</td>
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Ref ID 021 – Seismically induced deformations in parapet wall

<p>| a | While the order of expected deformations for the articulated capping wall is clearly presented in the design report, it is not self-evident that the adopted details are optimal for this situation. The report would benefit from discussion / elaboration of the options, and transparent selection of the preferred solution for final design. Specifically: | We agree that the report would benefit from some elaboration on the review matters raised and this will be provided in the Stage 4 report. The relative height difference between the roadway and the wall elevation is a pragmatic/judgemental balance between: - Sufficient mass required to maintain global stability under various load cases. | Refer Section 16 of T+T Stage 4 detailed design report and Dwgs 27425-PPW-130 and 150 for further discussion on parapet wall design and selected arrangements. |</p>
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|      | • What is the basis for the selected roadway height relative to the wall elevation?  
• What would be the benefit or disadvantage of including a toe detail on the wall and/or incorporating a structural joint to the membrane to avoid local differential deformations?  
• The seal detail appears questionable from a serviceability perspective. What would be the implication of incorporating a flexible centre bulb water stop detail? | • Length of ramp required to access the crest road off the spillway bridge.  
• Limiting parapet wall panels size to enable pre-casting.  
• Contractors advise using a 100 tonne crane it is practical to handle a 12 tonne load – the panel could be 4 m long. It is possible to increase the wall height – but the trade-off will be in the length of panel to maintain an overall handling weight of about 12 tonnes. Contractor advice takes into consideration wind loads and propping lengths etc.  
The vertical joints of the parapet wall are not likely to be subjected to water load very often. The base of wall is more than 1.9 m above NTWL and is also just above the Mean Annual Flood (MAF) water level. Therefore the water tightness of the parapet wall vertical joint is far less of a concern than for the face slab. Nonetheless, incorporating a central bulb water stop in the vertical joint may be relatively easy to construct if the parapet wall is poured insitu rather than made of precast panels. Consideration will be given to this matter, including how such a detail would incorporate the compressible joint filler, in Stage 4 and details will be amended accordingly if deemed appropriate.  
Consideration and discussion of the potential effects of embankment settlement on the parapet wall will be deferred to Stage 4. | Refer Sections 15 of T+T Stage 4 detailed design report for further discussion on the concrete face, and Dwgs 27425-ECF-100 to 153 for adopted arrangements including re-entrant angles and joint layout. |

Ref ID 022 – Concentrated stresses in the membrane (concrete face)

a While the generic membrane design discussion in section 7.6.1 is acknowledged, the current plinth profile illustrated on the drawings does include a re-entrant angle on the right abutment hat has potential to locally concentrate in-service stresses in the membrane. The design report would benefit from acknowledgment of this potential, and discussion on the potential effects of this situation in the context of the generic design. Based on our telephone conversation with Ian Walsh on 8 August 2013, we understand that the reviewer’s concern lies primarily with face slab joint locations around the re-entrant angle in the plinth. We agree that the face slab joint locations around such re-entrant angles in the plinth warrant refinement and these, along with consideration of face starter slab configurations, will be carried out in Stage 4. Refer Sections 15 of T+T Stage 4 detailed design report for further discussion on the concrete face, and Dwgs 27425-ECF-100 to 153 for adopted arrangements including re-entrant angles and joint layout.
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<td>detailing, including the degree of reinforcement, relationship to slip formed joints, need for any additional local strain relief details etc.</td>
<td>In the interim we provide the following comments in relation to plinth re-entrant angles. It is not uncommon in CFRD dams to have re-entrant angles so they are not a concern per se’. ICOLD Bulletin 141 states that face slabs generally move towards the centre of the dam and away from the dam abutments, and that most slabs are generally in compression, except at the abutments. Cruz et al (2010) and ICOLD Bulletin 141 reference Cooke and Sherard (1987) who state that there is no possibility of high contact stress and spalling at perimeter joints in dams of low to moderate height (less than about 75 m). This is because there is little compression in the slab before the reservoir is filled and the joint opens and offsets moderately when the reservoir load is applied. Nonetheless, the face slab design presented in the Stage 3 drawings still provides edge reinforcement at the perimeter joint to prevent spalling at perimeter waterstop prior to reservoir filling, as is recommended for high dams and by ICOLD Bulletin 141.</td>
<td>Further refinements to the perimetric joint reinforcement and slab thickness have been included in the Stage 4 design as presented on the Dws referenced above.</td>
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**Ref ID 023 – Uplift under the spillway lining**

While the spillway lining anchorage and drainage system is described in sections 15.5.3 and 15.5.4 of the design report, the report would benefit from presenting a specific discussion on the potential failure mechanisms associated with excessive hydraulic loading on the lining, specifically covering the implications of the drainage layout and positioning of educator features, including the implications of venting provisions and the longitudinal interconnection of the under drainage pipework. This elaboration should include discussion on the partial load factors adopted for structural design, in the context of the degree of confidence established in the loading condition in question.

Based on our telephone conversation with Ian Walsh on 8 August 2013, we understand that the reviewer’s concern lies primarily with the potential for the underdrains to become pressurised should the longitudinal drains become blocked at the downstream end. This concern is acknowledged and modifications to drain details will be undertaken in Stage 4. We propose to substantially reduce the risk of pressurisation occurring by providing additional venting to the longitudinal drains. Structural design and reinforcing of the spillway concrete slab will be covered in Stage 4 documentation.

Refer Sections 17.7.6 of T+T Stage 4 detailed design report for further discussion on spillway underdrainage. Additional venting, drainage detailing, and slab reinforcement undertaken as part of the Stage 4 detailed design are presented on Dws 27425-SPL-100 to 165 (see Dws 27425-SPL-106 and 143 for drainage).
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| a    | While the mounting of twin DN1200 steel offtake pipework and associated 2x 14m 2 screen facilities is described in the design report, there are several details that require further discussion or elaboration. Specifically:  
  - The durability of copper water stops is not discussed. Cases of slightly acidic soft water chemistry can adversely affect the life of copper elements exposed to such water.  
  - The need for, or benefit of, adopting a secondary flexible centre bulb water stop in the vertical membrane joints is not discussed. | Based on our telephone conversation with Ian Walsh on 8 August 2013, we understand that the reviewer’s concern regarding the durability of copper water stops stems from Otago experience with low pH soft water that is aggressive to copper. The Lee catchment is quite different from the Otago conditions where soft water occurs. Although it is unlikely that the water chemistry in the Lee Valley will adversely affect the life of copper water stops, we consider it appropriate to investigate the matter further in Stage 4 design. Interestingly we can find no references to adverse effects on copper water stops in ICOLD Bulletin 141, Cruz et al 2010 or internet searches. Should Stage 4 investigations indicate that the copper water stops may be adversely affected by water chemistry, alternative PVC arrangements will be specified. Incorporation of a central bulb water stop is not usual practice in CFRD dams. | There is no new information obtained during Stage 4 to suggest that a different water stop arrangement is required. |
<p>| b    | The cast in rail mounting brackets shown on the drawings would appear to conflict with the effective travel of the slip form. | The proposed cast in rail mountings are intended to be flush with the face of the concrete so should not compromise the ability to slip form. Further details will be provided in Stage 4. The slip forming process may need to be altered slightly for this slab to accommodate placement of the cast in fittings. We expect that the extruded concrete curbing will be used to mount the cast in fittings prior to slip forming. | Refer T+T Dwgs 27425-ECF-100 to 153 for adopted cast in bracket arrangements and WSP Mechanical Dwgs for rails. |
| c    | The ability of the slab to handle local loading associated with extreme loading of the screen assemblies is not specifically addressed. This aspect is not automatically able to be extrapolated from generic/empirical membrane details used previously. | This issue has not been resolved in detail as part of Stage 3 design and will be addressed in Stage 4 design. Assessment of local effects will be made and if thickening or additional reinforcing is required, then the design will be amended. | Further structural assessment was undertaken as part of Stage 4 design and confirmed not specific thickening for additional reinforcement is required to accommodate operational loads on the concrete face due to the intakes. Trimmer |</p>
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<td>d</td>
<td>The accommodation of slipform guides and potential conflict with the extruded kerb concrete is not discussed.</td>
<td>We do not believe that there is necessarily a conflict between the slipform guides and the extruded curb. ICOLD Bulletin 141 advises that the extruded kerb process is probably the most important improvement to CFRD construction in recent years. The process was originally implemented to facilitate construction and has been used on numerous CFRDs using slip forming to construct the face. The curb provides a competent, clean surface for the subsequent operations of form placement, reinforcement placement and slab construction. We consider that this concern would easily be resolved by a competent contractor familiar with construction of CFRD dams.</td>
<td>No further comments.</td>
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<td>e</td>
<td>Alternative lower profile screen layouts that might provide reduced cantilever loading on the membrane are not identified nor evaluated for final design.</td>
<td>M&amp;E design will be completed in Stage 4 and it is intended that a low profile screen will be designed to reduce cantilever loading on the face slab and rail system.</td>
<td>Screen design has not been completed and is part of a design build contract based on the performance specification prepared by WSP Opus.</td>
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**Ref ID 025 – Shear keys in spillway crest and starter dam**

- a | The design report would benefit from including a brief discussion on the intended functionality of the shear key details for the various concrete structures, leading to supporting justification for the adopted layouts shown. | In light of the reviewer’s comments during Stage 4 design we will further consider the design and layout of the shear keys for the various concrete structures. The Stage 4 report will include a brief discussion on shear key design and drawings modified if design changes are considered necessary. | Refer Sections 10.3 and 17.6.2 of T+T Stage 4 detailed design report for further discussion on the shear keys adopted for the starter dam and ogee weir. |

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\t\"auckland\projects\27425\27425.100\workingmaterial\27 stage 4 documentation\stage 4 design report\appendix i peer review comments responses\2018-10-09.daf.rpt.waimea dam stage 4 detailed design report peer review comments and responses rev 01.docx"
Dear Mark

Waimea Dam – Summary of Mott MacDonald Involvement of Detailed Design of Waimea Dam

Introduction

This letter is provided for inclusion in the Waimea Dam design documentation to summarise Mott MacDonald’s expert advice and technical guidance to the design team with hands on direct input for the Waimea Concrete Face Rockfill Dam (CFRD).

Tonkin + Taylor (T+T) engaged Mott MacDonald as T+T’s sub-consultant to provide design and design review services for the dam.

The Mott MacDonald dam specialists providing the above services are:

- Eric Guilleminot; and
- Philippe Cazalis de Fondouce

CV’s for Eric and Philippe are attached for reference which demonstrate their extensive international experience in design and construction of large dams and hydropower projects of different types including CFRD.

Scope of works by Mott MacDonald

Mott MacDonald was engaged by T+T for the following services:

- As a key output, to provide a PS1 producer statement for the design of the dam embankment demonstrating design leadership of this key project element;
- Assist in revising the design of the dam for the revised seismicity including revision of filters, review of seismically induced settlements;
- Review the dam design in respect of the site investigations carried out during the ECI phase (i.e. trial embankment, additional drill holes etc);
- Review the contractors proposed temporary diversion strategy and its interface with the dam;
- Agree and assist in completion of detailing the dam specifically the CFRD concrete face; plinth; wave wall; and embankment zoning;
- Assist in updating the detailed design report;
- Review contractor suggested alternatives;
- Provide recommendations on dam monitoring and instrumentation devices (e.g. deformation markers, inclinometers, seismographs, drainage measurement, reservoir monitoring);
- Assist T+T to write technical specifications;
- Participate in workshops and meetings;
- Address peer review and regulatory review comments;
- Provide designer site construction observation;
- Provide advice on dam filling and commissioning.

Project visit in 2017

We confirm that Eric Guilleminot and Philippe Cazalis de Fondouce came to New Zealand from Saturday 16 to Thursday 21 December 2017. During that New Zealand visit, the following was undertaken by the Mott MacDonald team members:
- A review of existing key design reports including the Feasibility and Stage 3 Design reports and Drawings
- Visited the dam site on both abutments and inspected site materials from the Stage 3 trial embankments in the company of the T+T senior design team
- Met with the peer reviewer Ian Walsh of Opus International Consultants
- Met with Waimea Water
- Discussed with T+T initial comments on the Stage 3 design and areas for further development and investigation during Stage 4.

Summary of areas of key input

We can confirm that we have provided specific input and design detailing and/or advice on:

- Seismic design and analysis of the dam embankment
- Filter compatibility
- Dam zoning arrangements
- Grout curtain, grout curtain details and Specification
- Seepage bund options
- Plinth perimetric joints and reinforcing details,
- Concrete face joint details
- Concrete face to parapet wall joint details
- Review of FHTJV’s draft diversion design
- Dam Drawing details.

We have reviewed the Design report and confirm that we agreed to being named as co-authors of this report given our high level of involvement in the design of Waimea Dam project.

While T+T has been the lead for the spillway design, we confirm that we are satisfied that the spillway arrangements are consistent with current international practice.

We clarify that we have not been intimately involved in the detailed design of the mechanical and electrical aspects given that WSP-Opus is T+T’s expert designer for this component of the project. We can however confirm that we are satisfied with the proposed concepts of the inclined intakes and valve arrangements.

Attendance at key workshops and meetings

We provide a summary of key meetings and workshops that we have been party to. Other telecom discussions have been held with T+T on points of detail in addition to the meetings listed below.

- 16 to 21 December 2017 – Site visit and briefing with T+T and Ian Walsh (Opus)
- 15 January 2018 – Meeting with T+T to discuss grout curtain design and requirement for seepage modelling and analysis
- 31 January 2018 Meeting with FHTJV, Waimea Water and T+T to discussion site geology
- 16 February 2018 Waimea Dam – Value Engineering Workshop with FHTJV, T+T and Waimea Water
- 12 April 2018 Meeting with T+T to discuss grout details and Specification
- 8 May 2018 Meeting with T+T to discuss filters, plinth and starter dam arrangements
- 11 June 2018 Meeting with T+T to review Dam drawing package
- 14 August 2018 Meeting with T+T to prepare for Value Engineering workshop on 15 August 2018
- 5 November 2018 Meeting with Waimea Water, T+T, Damwatch and TDC to address Damwatch comments
- 9 November 2018 Meeting with T+T to discuss spillway waterstops and drainage comments received by Damwatch
- 10 December 2018 Meeting with T+T to discuss required inputs during Construction to satisfy NZSOLD Dam Safety Guidelines 2015
Teleconferences and emails

We can confirm that we have had significant email, conference call and telephone contact with T+T staff on average at least weekly during the ECI phase in 2018 and at times conversations and/or communications on a daily basis.

Closure

We trust that this letter summarises Mott MacDonald’s extensive design involvement in the Dam from December 2017 until present. We confirm that we have been involved in scoping required design attendance during construction and that we are available to undertake these services.

Should Waimea Water require further clarification we request that this information is sought via T+T as the lead consultant.

Yours sincerely,

[Signatures]

Eric Guilleminot

Philippe Cazalis de Fondouce
ERIC GUILLEMINOT

Nationality: French
Year of Birth: 1969
Profession: Specialist Hydropower Engineer
Specialisation: Hydropower, dam, hydraulic and structures
Position in Group: Technical Director and Practice Leader, Hydropower
Year of joining Group: 2016

KEY QUALIFICATIONS

Over 25 years of experience in dam and hydropower engineering, covering all aspects including preliminary site investigations, feasibility studies, design review, detailed design development, construction liaison, operation and maintenance requirements, and risk assessment for both new and existing projects.

Has been involved in the design and assistance during construction of several projects with some including very high discharge capacity spillways, high dams, long tunnels and underground powerhouses.

Professional experience has been acquired through international consulting engineers’ companies, residing and working in Singapore, Australia, France, Turkey, Argentine, Venezuela, Panama and Brazil, and executing missions to Pakistan, China, Laos, the Philippines, Indonesia, Nepal, China, Fiji, Papua New Guinea, Peru, New Caledonia, Guyana, Morocco and Algeria.

EDUCATION AND PROFESSIONAL STATUS


Member of the French Committee on Dams and Reservoirs of ICOLD.

Chair Person of the Pumped Storage session at Hydropower & Dams conference ASIA 2018.

EXPERIENCE RECORD

2016 – present  MOTT MACDONALD GROUP
Technical Director, and Practice Leader for Hydropower

Key projects:

- **Project:** Owner’s engineer, Kidston PSP, Queensland Australia (2017-2018 ongoing)
  - **Client:** Genex Power
  - **Project value:** AUD$330 million capital cost
  - **Positions held:** Project manager
  - **Man-Months:** 400 months
  - **Short description of project:** Technical feasibility Study and ECI Management for a 250MW underground pump storage plant with two reversible pump/turbines.
  - **Activities performed:** Led the international team for optimization of the project. Responsible for the liaison with all the relevant stakeholders and agencies associated with the project.
  - **Benefits delivered/value added:** Organised and led the review process for optimizing projects performances and revenues while minimizing costs and risks. Our connected thinking resulted in real cost savings for the project which confirmed the commercial viability of the project.
Project: Owner’s Engineer, Suki Kinari, Pakistan (2017-2018 ongoing up to 2022)
Client: SK Hydro
Project value: US$1300 million capital cost
Man-Months: 250 months
Positions held: Design review manager
Short description of project: Design review for an 880MW brownfield hydropower project with Pelton turbines and 22km long tunnel being constructed in a tectonically active region with complex geological background.
Activities performed: Led the international team for design review of the prepared by the EPC Contractor China Gezhouba Group Company Ltd. Carried out site visits to assist the project implementation.
Benefits delivered/value added: Organised and led the design review process for optimizing projects performances and revenues while minimizing costs and risks, and ensuring that the design is always ahead of the construction.

Client: Tonkin and Taylor / Waimea Water Augmentation Committee
Project value: US$100 million capital cost
Positions held: Dam specialist for the review of the detailed design and assistance during the ECI process.
Short description of project: The project consists in a 52m high CFRD built on the Waimea river for irrigation and water supply purposes. The dam, located in a highly seismic environment, is also classified as a high Potential Impact Category Project, and designed in accordance with NZSOLD 2000 Dam Safety guidelines.
Activities performed: Dam specialist providing technical advice and support to the designer Tonkin and Taylor for the review of the detailed design and assistance during the ECI process.
Benefits delivered/value added: Helping Tonkin and Taylor to design this CFRD dam following the state of the art. Proposed several optimisations on the spillway.

Project: Feasibility study, Toledo Pump Storage Plant, Philippines (2017)
Client: Citicore Power Inc.
Project value: US$180 million capital cost
Man-Months: 25 months
Positions held: Project principal
Short description of project: A concept design and feasibility study for a 200MW brownfield pumped storage development.
Activities performed: Led the team for site visit, conceptual design, power energy calculation, cost estimate and feasibility study.
Benefits delivered/value added: Identified an alternative design for the upper reservoir with low social impact and low unit cost.

Project: Due diligence, Nido pumped storage and solar combined project, Chile (2016)
Client: Total
Project value: US$1080 million capital cost
Man-Months: 25 months
Positions held: Project director
Short description of project: A 720MW pumped storage project to work jointly with a proposed 1600MW solar plant, in the Atacama Desert, using desalinated water.
Activities performed: Responsible for the due diligence review of prior feasibility studies performed by others, CAPEX and OPEX, and comments on proposed operating modes and conceptual design.
Benefits delivered/value added: Identified an alternative design with an underground powerhouse with much less risk than the selected scheme.

Project: Technical adviser, Nam Pha Gnai Dam, Laos (2016)
Client: DSK Group
Project value: US$20 million capital cost
Man-Months: 2 months
Positions held: Dam specialist
Short description of project: Mott MacDonald acted as technical advisor for a 65m high concrete gravity dam with a surface spillway for 8000m$^3$/s. Project developed by a private company without involvement of Chinese contractors.

Activities performed: Regular site visits to support on technical aspects for the owner.

Benefits delivered/value added: Drastically reduced the amount of reinforcement recommended by the designer. Savings 4% of the project cost.

Client: Citicore Power
Project value: US$80 million capital cost
Man-Months: 25 months
Positions held: Project principal

Short description of project: Concept and tender design for a 30MW run of river hydropower plant in the Isabella province. The project includes a 45m high concrete gravity dam, a 2km long tunnel and a powerhouse equipped with two Francis turbines.

Activities performed: Reviewed a team of international and local engineers and scientists for a tender design and coordinated all the field investigations.

Benefits delivered/value added: Assisted and guided client in a new venture in the hydropower field. Project has demonstrated commercial viability and is currently at implementation phase.

Project: Owner’s engineer, Sukarame, Indonesia (2016)
Client: Velcan Energy
Project value: US$30 million capital cost
Man-Months: 25 months
Positions held: Reviewer

Short description of project: Review of the tender design for a 7MW run-of-river hydropower plant (HPP).

Activities performed: Reviewed a team of international and local engineers and scientists.

Benefits delivered/value added: Optimised the buried penstocks. Improved the technical quality of the deliverables to match the high expectations of the client.

Other project experience:

2013 – 2016 ENTURA, Australia
Specialist Hydro Power Engineer

Kidston pump storage plant, Genex Power, Australia (2015-2016) – Project manager/specialist hydropower engineer for a bankable feasibility study. Led a team of international and local engineers and scientists and co-ordinated all the field investigations. Responsible for the liaison with all the relevant stakeholders and agencies associated with the project. The project involves a 450MW underground pump storage plant with two reversible pump/turbines. The upper reservoir is a membrane lined reservoir built on top of the existing waste rock dump.

Nagmati Dam Feasibility Study, Department of Irrigation of the Government of Nepal, Nepal (2015) – Dam specialist for a feasibility study. Led a team of international and local engineers and scientists and co-ordinated the requirements for field investigations, additional analysis, and updating the feasibility study. The project involves a 90m high water supply dam for restoring the river environment in the Kathmandu Valley, and a secondary power house. Identified an upstream alternative with 50% less volume.

Sovi Dam Feasibility Study, Water Authority of Fiji, Fiji (2013-2015) – Project manager/specialist hydropower engineer for a feasibility study. Led a team of international and local engineers and scientists and coordinated all the field investigations. Responsible for the liaison with all the relevant stakeholders and agencies associated with the project. The project involves a 93m high water supply dam for the Suva area, and a secondary power house. Client satisfied by the outcome of the study with the possibility to supply water by gravity from the reservoir.
Nam Pha Gnai Dam, DSK Group, Laos (2015) – Dam specialist. Detailed design for a 65m high concrete gravity dam with a surface spillway for 8000m³/s.

Baram HPP, SEB, Malaysia (2015) – Design review manager for the review of the tender design for the Baram 1 HEP. The project comprises a 160m high roller compacted concrete (RCC) dam, 1,200 MW four unit power station, power intake with surface penstocks, twin 13m diameter by 800m long diversion tunnels and 23,700m³/s gated overflow spillway.

Hela Hydropower, LR Group, Papua New Guinea (2013-2014) – Project manager/specialist hydropower engineer for a feasibility study. Led a team of international and local engineers and scientists and co-ordinated all field investigations. Responsible for liaison with all the relevant stakeholders and agencies associated with the project. This project consisted of a diversion dam, a 4km long headrace tunnel in karstic limestone and an underground power house with 160MW installed capacity.

Mongi Bulum, PNG Power, Papua New Guinea (2013-2014) – Project manager/specialist hydropower engineer for pre-feasibility study involving management, co-ordination team and liaison with relevant stakeholders. This project consists of two diversion dams on the Mongi and Bulum rivers, a 23km long headrace tunnel and a surface power house of 120MW.

2005 – 2012 TRACTEBEL ENGINEERING FRANCE (trading as Coyne et Bellier), France

Specialist Hydropower Engineer

Quitaracsa 1, Enersur, Peru (2011-2012) – Design review manager. Duties included approval of the pre-feasibility study, final design study and assistance during construction. This project consisted of a 15m high concrete gravity deviatory dam in a high-mountain environment (access through roads at 4300m asl), an offstream water supply storage of 400,000m³, a 5km long high pressure unlined tunnel and an underground powerhouse with 116MW and two Pelton units with 850m head.

Dos Mares, Suez Energy Central America, Panama (2008-2011) – Technical director for owner’s engineer for approval of detailed design study and assistance during construction. The project includes three powerhouses in cascade with 20kms of HDPE lined channels and two deviatory dams. The upstream powerhouses are equipped with two S-type Kaplan Turbines for 125m³/s and the downstream powerhouse is equipped with two vertical Kaplan turbines for 155m³/s. The total installed capacity is 118MW.

Tocoma, Edelca, Venezuela (2006-2007) – Technical assistance design manager. Duties included the approval of a detailed design study and assistance during construction. The project includes an 8km long dam with a gated spillway (28,750m³/s of discharge capacity through nine gates of H=15m and L=12m) and a powerhouse with 2,160MW installed capacity (10 Kaplan’s turbine units). Also included hydraulic model studies of the spillway energy dissipation.

Caruachi, Edelca, Venezuela (2006) – Project manager/specialist hydropower engineer for a vibration study. The project includes a 5km long dam with a gated spillway (30,000m³/s of discharge capacity through nine gates of H=15m and L=12m) and a powerhouse with 5000m³/s design discharge. Conducted studies of the vibration caused by the spillway energy dissipation on the adjacent powerhouse.

Ouldjet Mellegue, Agence Nationale des Barrages, Algeria (2005) – Project manager/specialist hydropower engineer for the Tender Design study and co-ordination of investigations. The project includes a RCC dam of 50m height with a surface spillway (10,000m³/s of discharge capacity).

Dumbea, Mairie de Dumbea, France (2002-2005) – Project manager/specialist hydropower engineer. Duties included risk assessment and mitigation, safety review, surveillance, monitoring, reporting, deficiency assessment, upgrades, and remedial works of the spillway. The project includes a 35m high arch dam executed in 1954, with a surface spillway of 1400m³/s discharge capacity.

1997 – 2004 COYNE ET BELLIER, France
Senior Hydropower Engineer

Koniambo, Falconbridge and Hatch-Technip, New Caledonia (2001-2004) – Deputy project manager/senior hydropower engineer for the feasibility and Tender Design study and coordination of investigations. The project includes a 50m high roller compacted concrete dam with a surface spillway (4500m³/s of discharge capacity).

Gurara, Abuja City Council, Nigeria (2003) – Deputy project manager/senior hydropower engineer for Tender Design study. The project includes a 75km long pipeline, ø 3m, discharge 14m³/s, for the water supply of Abuja using the Gurara reservoir.

Soubella, Agence Nationale des Barrages, Algeria (2002-2003) – Project manager/senior hydropower engineer for this feasibility study. The project includes a CFRD dam of 50m height with a morning glory spillway (500m³/s of discharge capacity).

M’Djedel, Agence Nationale des Barrages, Algeria (2002-2003) – Project manager/senior hydropower engineer for this feasibility study. The project includes a fill dam of 35m height with surface spillway (1000m³/s of discharge capacity).

Adjarala, Communauté Électrique du Bénin, Benin (2002-2003) – Project manager/senior hydropower engineer for Engineering and Procurement Contract technical assistance. The project includes an earth and rockfill dam with surface spillway (3900m³/s of discharge capacity), and a powerhouse of 95MW.

Chicoasen Powerhouse, Alstom Power, Mexico (2001) – Structural engineer for a detailed design study. The project includes the extension of the existing underground hydroelectric power plant (5 x 310MW) which capacity has been raised by three new units developing 3 x 310MW.

Rocha Grande Small HPP, Colas, France (2000) – Project manager/senior hydropower engineer for a pre-feasibility study. The project includes a 7.5MW river hydroelectric power plant equipped with bulb turbines.

Potrerillos Dam, CEMPPSA, Argentina (1999-2000) – Deputy project manager (site)/senior hydropower engineer for detailed design. The project includes a 117m high Concrete Face Rockfill Dam on 70m of alluviums with a cut off wall. Maximum Design Earthquake of 1.05g. Morning glory spillway (2,000m³/s of discharge capacity), and powerhouse with an installed capacity of 123MW. Headrace tunnel 4500m long. Dam volume = 6.4Mm³.

Pamuk HPP, Dolsar, Turkey (2000) – Senior hydropower engineer for an assessment study. The project includes a 3.6km long power tunnel and a powerhouse with an installed capacity of 22MW.

Wala Dam, Hashemite kingdom of Jordan, Jordan (1998) – Hydropower engineer for a stability analysis. The project includes a 49m high Roller Compacted Concrete gravity dam with a free surface spillway (design flood 2,000m³/s) on problematic foundation.

Lakhwar Dam, Government of Uttar Pradesh, India (1997-1998) – Hydropower engineer for feasibility study. The project includes a 200m high arch-gravity dam on problematic foundation. Concrete volume = 2.5Mm³. Gated spillway with design flood 8000m³/s.

Berke Dam, Çukurova Elektrik A.S., Turkey (1997) – Hydropower engineer for design studies and supervision of the works. The project includes a 201m high arch dam with a gated spillway (2000m³/s of discharge capacity) and two underground spillways, a grouting curtain of 500,000m², an underground powerhouse with an installed capacity of 510MW.

1994 – 1997 STUCKY CONSULTING ENGINEERS, France Dam Engineer

Nancy Dam, VNF, France (1995-1997) – Hydropower engineer for assistance on the supervision of the work. Study office for a group of contractors for the conception and realisation of a flap valve dam on the Meurthe river at Nancy (L = 95m, H = 7m, three flops of 20m large).

Sainte Cecile d’Andorge Dam, Mairie de Sainte Cecile d’Andorge, France (1996) – Hydropower engineer for site supervision. The project consisted of restoration of the asphaltic upstream face of the Sainte Cécile d’Andorge dam; The dam is 40m high and was built in 1967; Milling of the damaged part of the membrane (slope 1,7/1) and implementation of the new clear membrane.

1992 – 1994 COYNE ET BELLIER, France
Hydro Power Engineer

Berke cofferdam, Çukurova Elektrik A.S., Turkey (1992) – Graduate hydropower engineer with duties including assistance for site supervision. Project includes a 40m high RCC cofferdam.

Berke Dam, Çukurova Elektrik A.S., Turkey (1992-1994) – Hydropower engineer for the design studies and supervision of the works. The project includes a 201m high arch dam with a gated spillway (2,000m³/s of discharge capacity) and two underground spillways, a grouting curtain of 500,000m², an underground powerhouse with an installed capacity of 510MW.

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**LANGUAGE CAPABILITY**

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<td>French</td>
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</tr>
<tr>
<td>Spanish</td>
<td>Spoken – Fluent; written – Fluent; reading – Fluent</td>
</tr>
<tr>
<td>Portuguese</td>
<td>Spoken – Fair; written – Fair; reading – Good</td>
</tr>
</tbody>
</table>
PHILIPPE CAZALIS DE FONDOUCE - Designer’s Representative

Nationality: French
Year of Birth: 1956
Profession: Civil Engineer
Specialisation: Hydropower, Dam, and Major Hydraulic Structures
Position in Group: Major Projects Director – Hydropower
Year of joining Group: 2016

KEY QUALIFICATIONS

Mr. Cazalis has 36 years of experience in dams and hydropower engineering. He acted as Project Director, Project Manager, and Chief Design Engineer, responsible for the design and construction supervision of a large variety of dams, hydropower plants and large hydraulic infrastructures, in different parts of the world, including several years in Asia and Latin America. His responsibilities have included significant periods of time as resident engineer during both design and construction stages at numerous HPP construction sites.

EDUCATION AND PROFESSIONAL STATUS

MSc Federal Institute of Technology Zurich, Civil Engineering 1977 – 1981 Switzerland
Member of CFGB, the French Committee on Dams and Reservoirs of ICOLD
Member of ICOLD

EXPERIENCE RECORD

2016 – present MOTT MACDONALD GROUP

1987 – 2016 TRACTEBEL ENGINEERING FRANCE (trading as Coyne et Bellier), France
Dams and Hydropower Engineer

Batang Toru HPP – Indonesia (2018) Project Director. Due Diligence carried out for Genting Lestari Energi Pte Ltd. on the 530MW hydropower project developed by North Sumatera Hydro Energy and constructed under EPC contract by Sinohydro. See description below.

Waimea dam – New Zealand (2017-2018 ongoing) Dam specialist providing technical advice and support to the designer Tonkin and Taylor for the review of the detailed design and assistance during the ECI process. The project consists in a 52m high CFRD built on the Waimea river for irrigation and water supply purposes. The dam, located in a highly seismic environment, is also classified as a high Potential Impact Category Project, and designed in accordance with NZSOLD 2000 Dam Safety guidelines. The project is being developed by Waimea Water Augmentation Committee.

El Quimbo dam – Colombia (2017-2018 ongoing) Dam CFRD specialist providing technical expertise and support to ENEL-EMGESA. The 151m high El Quimbo dam (CFRD – 7.5 million m3 of alluvium gravel fill) has been completed in 2015 but the reservoir could never be filled to its full supply level due to excessive dam deformations and settlements. In 2017, Mott MacDonald has been awarded the contract for the numerical modelling, stability calculations and recommendation for final reservoir impoundment. The 3D dynamic calculations are presently ongoing.
Nzilo 2 HPP - Democratic Republic of Congo - (2017 ongoing). Project Manager for the prefeasibility study of a cascade project on the Lualaba river in Katanga Region and including two Run of River plants cumulating 100MW capacity. The Project is developed by Congo Infra.

Nachtigal HPP– Cameroon (2017 ongoing) Project Manager and Dam Specialist. Independent Technical and ESIA Assessment for IFC of the 420 MW Nachtigal HPP on the Sanaga river developed by EDF and the Govt of Cameroon. The project construction performed under two main EPC contracts is scheduled to start in 2018. The project includes a low RCC dam 1455m long a 3.3km headrace canal rated at 980m³/sec, a surface powerhouse housing 7x60MW Francis units, and a 50 km long 225kV HV line and associated substations

Kidston PSP – Australia (2017 ongoing) Team Leader Civil works Technical Feasibility Study and ECI Management for the developer Genex Power for a 250MW underground pump storage plant with two reversible pump/turbines. Incorporated in the international team for reviewing concepts and optimizing projects performances and revenues while minimizing costs and risks. Our connected thinking resulted in real cost savings for the project—around $78 million—which confirmed the commercial viability of the project.

Tilougguit I and II HPPs - Morocco (2017) Dam and hydropower specialist. Due Diligence for Platinum Power of two hydropower projects located on the Assif Ahançal River. Tilougguit I HPP includes a 41m high concrete gravity dam with gated spillway and adjacent powerhouse housing 2 Francis units cumulating 7.2MW of installed capacity. Tilougguit II HPP includes a 24m high concrete gravity dam with free ogee spillway, a 6.5km headrace tunnel 4.3m diameter connected to the surface powerhouse housing 2 Francis units cumulating 30 MW of installed capacity

Asahan1 HPP Due Diligence – Indonesia (2017) Team leader - Asahan 1 is a 2x90 MW hydropower station located on Asahan river downstream of Lake Toba in Sumatra. The plant was commissioned in 2010. The International Finance Corporation (IFC) and the developer of Asahan 1 are working together to find a better long-term project funding structure for Asahan 1. IFC and together with the developer had agreed to appoint Mott MacDonald as the independent engineer to conduct a technical due diligence on Asahan1.

Karuma and Isimba HPP Audit – Uganda (2017) – Team Leader for the technical, financial and contractual audit of two large HPP developed by UEGL and under construction by Chinese contractors. Karuma HPP-600MW, Hₙ=60m, Qₙ=1128m³/sec EPC Contractor Sinohydro and Isimba HPP-183MW, Hₙ=15.1m, Qₙ=1375m³/sec EPC Contractor CWE.

Suki Kinari Hydropower Project – Islamic Republic of Pakistan (2017 ongoing) – Chief Engineer (civil) for Owner Engineer’s design review of all technical documentation prepared by the EPC Contractor China Gezhouba Group Company Ltd. The review included the identification of alternatives for optimizing projects performances and revenues while minimizing costs and risks. The Suki Kinari HPP is a 870MW high head project operating under 911m gross head with 21km long HRT being constructed in a tectonically highly active region with complex geological background.

Semangka Hydroelectric Power Project - Sumatra, Indonesia (2016-ongoing) – Project Director for an Independent technical advisor’s role to the project owners Korea Midland Power Co. Ltd in the development of a 55.4 MW hydropower project including a 25m high dam and a 6.7km long power canal rated for 58m³/sec.

Batang Toru Project - Republic of Indonesia - (2016) - Project Manager for the review and control of all technical documentation prepared by the EPC Contractor Sinohydro before submission to the developer North Sumatera Hydro Energy (NSHE). The Batang Toru hydropower project is located in North Sumatra. It includes a 70m high concrete arch gravity dam, a 12km long and 8m diameter headrace tunnel and a 530MW hydropower plant housing 4 Francis turbine units. From seismic risk point of view, the vicinity of the project to the great Sumatra subduction fault makes it particularly challenging.
Budhi Gandaki Hydropower Project – Nepal - (2013-2015): Resident Project Manager for the 40 months of the Feasibility and Detailed Design Studies of the Budhi Gandaki Hydropower Project. The project includes a Power plant of 1200MW, a very large reservoir of 4500 hm$^3$ capacity created by a 263m high double curvature arch dam. The high seismic risk (SEE:1.2g & OBE: 0.6g) prevailing in Nepal makes the design of the project particularly challenging. The Project implies also a significant component for the Environmental and Social Impact Assessment and the related Environmental and Social management and mitigation plans.

Bui Hydroelectric Power Project – Ghana - (2012) - Senior Adviser Civil Works - Review of the Construction Methodologies and support to site construction supervision for Ghana Bui Power Authority. The 400MW Bui HPP built on the Black Volta River by SinoHydro includes a 110m high RCC dam.

Tocoma Hydroelectric Power Project – Venezuela - Resident Technical Director (2012) - Senior Adviser to the Project Director (2007) - CORPOELEC is building a 2160MW HPP on the Rio Caroni. Within the frame of the specialized consulting services contract awarded to the JV Decoyne, with Coyne et Bellier as leading partner, the joint venture is providing experts in different fields to assist CORPOELEC in planning, designing and supervising all works during the 7 years construction of the Tocoma HPP. The project includes a 8km long composite dam incorporating the power house aligning 10 Kaplan’s turbine units and also a major gated spillway rated at 28,750m$^3$/sec.

Rehabilitation project of 15 dams – French Polynesia - (2008-2011) - Senior Adviser to Electricité de Tahiti (EDT), GDF Suez Group - EDT is operating 15 mini and micro hydropower plants in Tahiti cumulating 46MW and providing about 30% of the island electricity demand. EDT has launched in 2007 a large project of dam’s rehabilitation and renovation to cope with the French regulation and the CFGB/ICOLD recommendations. Fifteen earth fill dams ranging from 15 to 30m in height are concerned by this systematic review. The role of the adviser includes planning and supervising investigations, design review, tendering and contract preparation, procurement of hydro mechanical equipment’s, works supervision and reporting.

Vaiiha hydropower project – French Polynesia - (2008-2009) - Senior Adviser to EDT, GDF Suez Group – EDT is planning the construction of a 10MW hydropower project on the Vaiiha river. This project includes a small regulation reservoir and 4 runoff river intakes directly connected to the power plant by a ramified penstock. This unusual configuration and the specific hydrological and digital simulation models developed for power capacity and energy generation, has been granted a special price at the 2009 Initiative Innovation Trophy from GDF Suez Group. The role of the adviser to EDT was the project management and coordination of the design studies and field investigations with a special emphasis on the design adaptation to mitigate the impact of the project in a valley of exceptional biodiversity.

Karahnjúkar Hydroelectric Power Project – Republic of Iceland - (2003-2006) - Project Manager Dams - Landsvirkjun has developed a 690MW HPP in East Iceland to supply power for the Alcoa Aluminium smelter under construction as well. The Construction Supervision covering several Contracts for the realization of 70km of tunnels (involving 3 TBM’s), a 100m high power intake, two saddle dams 25m, 60m high and a major dam (CRFD) 198m high, 8.5 million m$^3$ of rock fill, and including the role of Owner Representative has been awarded to a Joint Venture led by Mott MacDonald with Coyne et Bellier’s leadership for the dams.

Post Panama Locks Conceptual Studies – Republic of Panama - (2002-2003)- Resident Engineer and Project Coordinator - The Panama Canal Authority (ACP) has awarded to a European Consortium led by Tractebel Development Engineering the conceptual studies of a new set of locks on the Pacific side of the Panama Canal. Each lock step has 427m in length, 61m in width and 18 m in depth. The height between the Pacific level and the Gatun Lake level raise to +26m. The conceptual study includes three configurations with one, two and three steps locks including adjacent water saving basins. This concept has been selected by ACP and applied as well on the Atlantic locks. This project has been awarded the Grand Prix National de l’Ingénierie - Paris - October 2011.
Chicoasen HPP - Mexico - (2001) - Chief Design Engineer - The project includes the extension of the existing underground hydroelectric power plant (5 x 310MW) which capacity has been raised by three new units developing 3 x 310MW.

Multipurpose Potrerillos BOT Project - Argentina (1999-2001) - Project Manager resident - detailed design studies and technical assistance - Concrete Face Rock Fill Dam (river gravel fill CFRD) h=116m, cut-off wall h=70m, headrace tunnel 4500m, 123MW new powerhouse and refurbishing & upgrading to 62MW of an existing hydro power plant.

Dul Hasti Hydroelectric Project – India - (1998) - Resident Chief Engineer - Technical assistance to National Hydro Power Corporation (NHPC) for the construction supervision of the Dul Hasti Hydroelectric Project (concrete gravity dam h = 70 m, underground powerhouse 390MW, 800 MUSD construction contract (see description below).

Tossaye Dam and HPP – Mali - (1997-1998) - Chief Design Engineer - The Project includes a 30 m high Embankment Dam on the Niger River in Mali with a gated spillway (15,000m³/s of discharge capacity), an Outdoor power plant on the river with an installed capacity of 30 MW and a HV transmission line 70 km long. Responsible for feasibility study of all aspects of the Project and ensuring timely and quality engineering services in accordance with the Contract.

Birecik Hydropower Scheme, Turkey - (1995-1998) - Chief Design Engineer - Responsible for the review and control of the Final Design and construction drawings for the 65m high Birecik concrete, earth and rockfill dam, spillway, power plant and substation (720MW) and a HV transmission line 200 km long. The total volume of fill and concrete are 10 million and 2 million of m³ respectively. Build Operate and Transfer Contract.

Gojeb Hydropower Scheme, Ethiopia - (1996) - Team leader for the pre-feasibility studies of the Gojeb Hydropower Scheme.

Gomal Zam Multipurpose Project, Pakistan - (1994-1995) - Project manager - feasibility studies for a 136m high Roller Compacted Concrete gravity dam and underground power house (20 MW), for irrigation and hydropower purposes. Responsible for the civil design aspects of the Project.

Dul Hasti Hydropower Scheme, India - (1990-1993) - Resident Engineer - Coordinator and consultant’s representative in New-Delhi, review of design, preparation of the detailed design, construction drawings and technical assistance during construction for DUL HASTI hydroelectric power scheme on the Chenab River in Kashmir including a 80m high concrete dam with discharge capacity of 8,000m³/s, a series of multi-level intakes, two underground sand traps, a 10.6km long headrace tunnel (TBM excavated diameter = 8.3 m), a 90 m high 20m dia. surge shaft, a 160m high pressure shaft partly steel lined (dia. 6.5m), a 390MW installed capacity underground power station and a 200 km long 400 kV HV transmission line. The Project was developed and built by Dumez/Sogea/Borie & Cegelec on a turnkey basis contract.

La Touche-Poupard dam, France - (1989-1990) - Civil Works Chief Design Engineer - Responsible in liaison with the Team Leader for the design review of the 36m high Roller Compacted Concrete Dam Project (diversion works, hydraulic structures, drainage and grouting works, etc.) at the Feasibility stage and then Team Leader at the Detailed Design Stage in charge in particular of the preparation of all Design Documents and Contractual documents related to the Civil Works portion of the Project including the General and Special Conditions, the Technical Specifications and Information’s to Tenderers and elaboration of the Contracts awarded to the successful Contractors.

CHARPAL dam, France - (1988-1990) Project Manager - Design for the 6m raising of an existing 40m high masonry gravity dam built in 1913, upgrade of hydraulic structures, drainage and grouting works, etc. and in charge in particular, at the Detailed Design Stage, of the preparation of all Design Reports related to the Civil Works of the Project.

Barrages du Gard (1987-1988) Project Manager – Dam safety assessment, instrumentation monitoring and yearly reporting for 4 dams ranging from 20 to 40m in height (rockfill and concrete gravity types) owned and operated by the Ministry of Equipment.

Madagascar - (1988) - Civil work expertise and rehabilitation projects of 7 existing HPP in Madagascar. Energy One WB Project

Tahiti (French Polynesia) - (1983-1987) for SEDEP - Feasibility, detailed design construction design and drawings, and Construction Supervision of 5 mini / micro power plants and associated rockfill dams (50 - 6000 kW)

Civil Service 1981-1982 in the Civil Engineering unit of the French Polynesian Public Administration.
LANGUAGE CAPABILITY

French : Mother tongue
English : Spoken – Fluent; Written – Fluent; Reading – Fluent
Spanish : Spoken – Excellent; Written – Excellent; Reading – Excellent

PUBLICATIONS


AWARDS

1st Prize Tractebel Engineering Awards – January 2009 for the Vaiiha mini hydro project studies Tahiti French Polynesia
Initiative Innovation Initiative Trophy GDF SUEZ – June 2009 for the Vaiiha mini hydro project studies Tahiti French Polynesia
Appendix K: Intake fixing design summary
1.1 Intake Rails and Fixings design summary

1.1.1 Methodology

The following loads were considered for the design of the rail and fixings:

- \( G_p \)  Self weight of pipe and saddle supports
- \( G_w \)  Weight of the water in the pipe
- \( G_i \)  Weight of the intake screen
- \( E_p \)  Seismic inertia load of pipe
- \( E_w \)  Seismic load of water inside and outside pipe
- \( E_i \)  Seismic inertia load of intake screen
- \( F_w \)  Wave load on intake screen
- \( F_m \)  Load on rails due to movement of section of intake pipe or intake screen
- \( U \)  Buoyancy load on empty pipe.
- \( F_t \)  Pipe thrust load from hydrostatic pressure test

Ultimate limit state load factors are applied to the above loads and different combinations are considered in accordance with AS/NZS 1170. The combination and load factors are tabulated below:

<table>
<thead>
<tr>
<th>Combination</th>
<th>( G_p )</th>
<th>( G_w )</th>
<th>( G_i )</th>
<th>( E_p )</th>
<th>( E_w )</th>
<th>( E_i )</th>
<th>( F_w )</th>
<th>( F_m )</th>
<th>( U )</th>
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<tr>
<td>1</td>
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<td>1.35</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>Static - intake screen</td>
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<tr>
<td>2</td>
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<td>-</td>
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<td>Seismic - intake screen (dry)</td>
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<td>5</td>
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<td>-</td>
<td>0.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Wave load (with reduced self-weight)</td>
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<td>6</td>
<td>1.2</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.5</td>
<td>-</td>
<td>-</td>
<td>Moving intake screen + top section of pipe</td>
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<td>7</td>
<td>0.9</td>
<td>-</td>
<td>0.9</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>Pipe buoyancy case</td>
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<td>8</td>
<td>0.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.5</td>
<td>-</td>
<td>Pipe thrust from hydrostatic test</td>
</tr>
</tbody>
</table>

1.1.2 Assumptions

A brief summary of the assumptions used to calculate and analyse the loadings are below:

- Downslope component of the pipe weight is resisted by the rail fixings.
- Intake pipe is DN1200 and 10mm thick.
- Pipe saddle supports are fabricated from 10mm thick plates.
- For rail and fixing design, safety evaluation earthquake (10,000 year return period) seismic loading is considered.
- Slope of the dam face is horizontal to vertical = 1.5: 1. Hence the angle of rail to the horizontal is 33.7° or 0.588 radians.

1.1.3 Rail Design Results

**Combination 1 – Intake Screen static case**

In accordance with the general theory of the beam on elastic foundation, the ultimate limit state bending stress in the rail under leg load was found to be 1.52 MPa and the concrete bearing stress...
was 0.19 MPa. By using the Andree-Fricke method (“Design of Hydraulic Gates”) the ULS bending stress in the rail was determined to be 1.70 MPa. Furthermore, a sensitivity check was conducted on the concrete Young’s Modulus, $E_c$; NZS3101 $E_c$ value of 22958 MPa was used and ULS bending stress was 1.84 MPa.

By inspection the rail bending stress and concrete bearing stress is very low and therefore design is acceptable.

**Combination 2 – Intake pipe static case**
- ULS bending stress in the rail under the leg load: 2.11 MPa
- Concrete bearing stress: 0.22 MPa
- Andree-Fricke method ULS bending stress: 2.36 MPa

By inspection the rail bending stress and concrete bearing stress is very low and therefore design is acceptable.

**Combination 3 – Intake pipe seismic case**
Use of method in section 6.2 of “Design of Hydraulic Gates”:
- ULS bending stress in the rail under the leg load: 7.34 MPa
- Concrete bearing stress: 0.77 MPa
- Andree-Fricke method ULS bending stress: 8.22 MPa

By inspection the rail bending stress and concrete bearing stress is very low and therefore design is acceptable.

Further check was conducted on the rail under tension leg load. By applying twice the tension leg load as a single point load to the rail, midway between fixings, the maximum spacing of rail fixings to avoid bending failure was determined to be $L_{max} = 9554$ mm. Hence bending of rail is unlikely to govern spacing of fixings.

**Combination 4 – Intake screen seismic case**
- ULS bending stress in the rail under the leg load: 13.69 MPa
- Concrete bearing stress: 1.43 MPa
- Andree-Fricke method ULS bending stress: 15.32 MPa

By inspection the rail bending stress and concrete bearing stress is very low and therefore design is acceptable.

Further check was conducted on the rail under tension leg load. The tension leg load was applied as a single point load to the rail, midway between fixings, and the maximum spacing of rail fixings to avoid bending failure was determined to be $L_{max} = 5327$ mm. Therefore bending of rail is unlikely to govern spacing of fixings.

**Combination 5 – Wave load on intake screen**
The wave load and overturning moment is compared with combination 4 as shown in the table below. By inspection, the wave load is not critical for the design of the rails or fixings.

<table>
<thead>
<tr>
<th>Combination</th>
<th>$F_h$ (kN)</th>
<th>$M$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>149.0</td>
<td>458</td>
</tr>
<tr>
<td>5</td>
<td>20.9</td>
<td>83.3</td>
</tr>
</tbody>
</table>
**Combination 6 – Moving intake screen**

No additional downward load in rail compared to the static case, therefore no rail check required.

**Combination 7 – Pipe Buoyancy**

By applying tension leg loads from saddles as point loads to the rail, the bending strength of the rail with the proposed fixing spacing of 1 m was found to be 202 kNm. Hence bending strength of the rail is adequate.

**Combination 8 – Pipe thrust**

In determining the pipe thrust from hydrostatic pressure test \( (F_t) \) it is assumed the intake structure will not be in place at time of hydrostatic test. That is, the thrust load will be transferred to the saddles not the intake structure anchors.

The axial load on pipe was found to be 785 kN along the pipe. By inspection, the design is adequate.

**Check rail in bending under horizontal loads**

Based on the chosen rail fixing spacing of 1000 mm, a check is conducted on the rail bending and shear under horizontal loading.

**Seismic loads from intake pipe**

The horizontal seismic load per saddle is calculated as shown below:

\[ E_w = 82.5 \text{ kN per saddle} \]
\[ E_p = 11.7 \text{ kN per saddle} \]

Total = 94.2 kN per saddle

There are 2 rails hence each load per rail is 47.1 kN per saddle.

The above load per rail is input into Microstran model to determine bending moment with a saddle spacing of 2000 mm. Loads were placed in various locations to find the worst case scenario. The results are shown below:

\[ M^* = 8.41 \text{ kNm} \]
$V^* = 44.3\, \text{kN}$

**Figure 1. Bending moment diagram of rail under horizontal loads**

**Figure 2. Shear force diagram of rail under horizontal loads**

**Properties of 250UC73:**
- $Z_y = 306000\, \text{mm}^3$ (elastic section modulus – weak axis)
- $F_y = 300\, \text{MPa}$
- $\phi = 0.8$
- $\phi M_n = 82.62\, \text{kNm} > 8.41$

Hence rail has sufficient bending capacity.

- $A_w = 1125.9\, \text{mm}^2$
- $f_y = 320\, \text{MPa}$
- $\phi V_v = 216.2\, \text{kN} > 44.3$

Hence rail has sufficient shear capacity.

**Seismic loads from intake screen**

The horizontal seismic load from intake screen is 149.0 kN in total, hence each leg of rail is carrying 37.2 kN. By inspection, this horizontal load is less critical when compared to the intake pipe.

**1.1.4 Fixing Design Results**

An initial fixing spacing of 1 m along each rail was chosen. It is assumed that 2 fixings per leg will resist intake screen leg shear and tension loads. The ULS loads per fixing for combinations previously defined are tabulated below:
\( V_x' \) = Shear in fixing, parallel to rail

\( V_y' \) = Shear in fixing, transverse to rail

\( N_z' \) = Tension in fixing, perpendicular to rail. Compression loads are ignored for fixing design

\( V_r \) = Resultant shear

<table>
<thead>
<tr>
<th>Combination</th>
<th>( V_x' ) (kN)</th>
<th>( V_y' ) (kN)</th>
<th>( N_z' ) (kN)</th>
<th>( V_r ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.97</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>12.9</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>3A (Horizontal)</td>
<td>16.7</td>
<td>47.1</td>
<td>52.8</td>
<td>50.0</td>
</tr>
<tr>
<td>3B (Vertical)</td>
<td>7.38</td>
<td>0</td>
<td>11.1</td>
<td>-</td>
</tr>
<tr>
<td>4A (Horizontal)</td>
<td>7.75</td>
<td>18.6</td>
<td>78.9</td>
<td>20.1</td>
</tr>
<tr>
<td>4B (Vertical)</td>
<td>9.30</td>
<td>0</td>
<td>13.9</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>By inspection, wave loading not critical for design of rails or fixings, see previous section Combination 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>3.59</td>
<td>1.86</td>
<td>-</td>
<td>6.06</td>
</tr>
<tr>
<td>7</td>
<td>11.6</td>
<td>-</td>
<td>17.4</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>67.3</td>
<td>-</td>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>

Design fixings in accordance with NZS3101 for the above ULS combinations

Number of bolts per fixing: 4 no. stainless steel
Bolt diameter: 20 mm
Embedment depth of base plate: 95 mm
Bolt spacing – y direction: 170 mm
Bolt spacing – x direction: 250 mm
Concrete strength \( f_c \): 21 MPa

The strength reduction factors used (NZS 3101 Clause 17.5.6.4):
Steel elements \( \phi = 0.75 \) (tension) \( \phi = 0.65 \) (shear)
Concrete failure \( \phi = 0.65 \) (tension) \( \phi = 0.65 \) (shear)

The following results were determined from above parameters:

Tension:
- Steel Strength \( \phi N_s = 514.5 \) kN for the fixing
- Concrete breakout strength \( \phi N_{cb} = 82.7 \) kN
- By inspection, tension pull-out strength of anchor will not govern due to large cast-in bearing plate.
- By inspection, side face blowout strength is not critical.
- **Design strength in tension \( \phi N_r = 82.7 \) kN.**

Hence tension strength is adequate.
Shear

- Steel strength $\phi V_s = 267.5$ kN for the fixing
- Breakout in shear both parallel and perpendicular to edge will not be critical due to anchors being far from any concrete edge.
- Pryout of anchor in shear $\phi V_{cp} = 165.3$ kN for the fixing.
- **Design strength in shear $\phi V_s = 165.3$ kN.**

Hence shear strength is adequate.

Combined shear and tension check

$$\frac{N'}{\phi N_H} + \frac{V'}{\phi V_H} \leq 1.2 \quad NZS3101 (Eq. 17 - 5)$$

<table>
<thead>
<tr>
<th>Combination</th>
<th>Combined check</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A (Horizontal)</td>
<td>0.94</td>
</tr>
<tr>
<td>3B (Vertical)</td>
<td>0.18</td>
</tr>
<tr>
<td>4A (Horizontal)</td>
<td>1.08</td>
</tr>
<tr>
<td>4B (Vertical)</td>
<td>0.22</td>
</tr>
<tr>
<td>7</td>
<td>0.28</td>
</tr>
<tr>
<td>8</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Hence design passes combined check.

A further check is conducted to examine the twisting moment of the fixings on the rail generated by transverse seismic loads ($V'_y$). The maximum transverse load is seen in combination 3A with a value of 47076 N. By applying the maximum transverse load at the top flange of the rail along with the leg tension load, $V'_y$ will generate a moment at the base of the rail.

- Maximum $V'_y = 47.1$ kN
- Tension force $N'_z = 52.8$ kN
- Height of rail = 254 mm
- Moment = 12.0 kNm

Assuming length of compression block in the $x'$ direction $b = 350$ mm. Try $a = 5$ mm (Using length of compression block in the $y$ direction = $a$)

$$0.885ab\phi' = 31.2 \text{ kN} = C$$

$$N'_z + C = T$$

Therefore tension force on the tension side of the fixing is determined to be 84.1 kN. The number of bolts resisting tension force is 2 bolts, and the spacing of tension bolts is 250 mm.

Concrete breakout strength in accordance with NZS3101 clause 17.5.7.2, is found to be $\phi N_{cb} = 51.8$ kN for the fixing where

$$N_{cb} = \phi_1 \phi_2 \phi_3 \frac{A_n}{A_{nc}} N_b.$$  

- $\phi_1 = 1$ (*no eccentricity*)
- $\phi_2 = 1$ (*assume no edge distance effects*)
- $\phi_3 = 1$ (*assume concrete is cracked due to shrinkage*)
- $k = 10$ (*cast in inserts*)
- $\lambda = 1$ (*normal weight concrete*)

By providing 2 fixings per rail per saddle, the concrete breakout capacity will be $\phi N_{cb} = 51.8 \times 2 = 103.6$ kN > 84.1 kN. Hence having 2 fixings per rail per saddle is adequate.