PRELIMINARY GEOTECHNICAL AND DAM CONCEPT ASSESSMENT MATAWII WATER STORAGE RESERVOIR, KAIKOHE
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MATAWII WATER STORAGE RESERVOIR, KAIKOHE

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<table>
<thead>
<tr>
<th>Issue</th>
<th>Details</th>
<th>Date</th>
</tr>
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<tbody>
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<td>Preliminary Geotechnical Assessment and Dam Design</td>
<td>29 April 2020</td>
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<tr>
<td>2.0</td>
<td>Preliminary Geotechnical Assessment and Dam Design</td>
<td>8 July 2020</td>
</tr>
</tbody>
</table>
Appendices

Appendix A: RILEY Dwgs: 190272-761 to -766
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MATAWII WATER STORAGE RESERVOIR, KAIKOHE

1.0 Introduction

Riley Consultants Ltd (RILEY), along with Williamson Water and Land Advisory Ltd (WWLA) and other project partners, has been commissioned by Mid North Water Trust on behalf of Northland Regional Council (NRC) to prepare documentation to support a resource consent application for construction of the Matawii Water Storage Reservoir, located north-west of Kaikohe.

The site, one of several options identified by the Northland Water Storage and Use Project (NWSUP): Pre-feasibility Demand Assessment and Design Study, was recommended for rapid progression due to the multi-purpose benefits it would provide, including:

1. Irrigable water for up to 100ha of generally Maori land immediately south of Kaikohe to seed developments.
2. A reliable supply to the planned Ngawha Innovation and Enterprise Park (NIEP); and
3. To supplement existing municipal supply needs.

This report outlines typical design, construction, and operational considerations for the reservoir, outlined with reference to the New Zealand Society of Large Dams (NZSOLD) Dam Safety Guidelines.

The primary objectives of the geotechnical and dam concept assessment are to:

- Specify the geological context of the dam sites and reservoir basin, and how this influences structural safety, water retention and reservoir slope integrity.
- Determine if any prohibitive geological or geotechnical conditions exist that could prohibit safe and cost-effective dam construction and operation.
- Refine the most suitable dam type and appurtenant structures based on geological, geotechnical, ecological, hydrological, operational, and cost considerations.
- Gain an understanding of the potential failure modes and how defensive measures can be incorporated into the design to reduce the risk of dam safety incidents or failure.
- Evaluate the present design against recommended performance criteria that is commensurate with the hazard posed by the reservoir (potential impact classification).
- Outline additional recommendations for the progression of the project through final design investigations and detailed design.

At the time of writing this report, the final phase of intrusive geotechnical investigations (machine boreholes and laboratory testing) were about to commence when New Zealand entered Level 4 lockdown as a result of the COVID-19 pandemic. This report, therefore, provides an assessment based on present understanding using available information captured to date. The findings presented will need to be reviewed and updated once machine boreholes and laboratory testing can be completed.
2.0 Site Description and Topography

The proposed dam site is located at 5435 State Highway 12 (SH12), near Kaikohe, previously referred to as the Far North Holdings Ltd (FNHL) Project site, Mid North Storage and Distribution Site No.10 (MN-10). This site is considered favourable in terms of its location, land ownership, surrounding water needs, and future plans for the property.

A provisional dam site location has been selected on an unnamed tributary of the Kopenui Stream on the southern slopes of the Te Pua Volcano to the north-east of Kaikohe township and south-east of Lake Omapere.

The main embankment traverses a narrow point within a local valley and follows the natural ridgeline on the true right. The gully is approximately 20m wide at its base and approximately 120m wide at its highest point (RL 252.0). The right abutment is 4m lower than the left meaning the embankment will need to be constructed proud of the existing ground on that side to achieve the desired storage. The base of the gully comprises a small spring-fed stream, with swampy ground/wetland features at several locations within the reservoir.

A natural saddle is located beyond/upstream of the right abutment, within which a flood spillway could be located cut into natural ground.

Watercourses downstream generally pass through farmland and forestry areas. There are a number of road crossings, including SH 12 at two separate locations. River reaches further downstream consist of the Wairoro Stream to the east of Kaikohe township, the Punakitere River, the Taheke River and the Waima River. The Waima River discharges to the Hokianga Harbour. The proposed dam is located approximately 60km upstream from the harbour.

3.0 Downstream Effects and Potential Impact Classification

3.1 Background

A potential impact classification (PIC) assessment considers the consequences of an uncontrolled release of the reservoirs' contents as a result of a dam breach. PIC assessments are independent of the likelihood of a failure, which, for a suitably designed, constructed and operated dam, should be very low.

A comprehensive PIC assessment involves determining dam breach characteristics, and hydraulic modelling downstream of the dam.

Module 2 of the NZSOLD Dam Safety Guidelines (2015) outlines the consequence assessment and dam classification framework adopted in New Zealand. It considers three principal components, being:

1. Damage level.
2. Population at risk.

Dams are categorised as low, medium, or high PIC based on these components.
The NZSOLD Guidelines provide design criteria, construction and operation requirements for each PIC, with a high PIC dam having the highest criteria. Such a classification system ensures the dam performance requirements are appropriate for the hazard posed by the reservoir.

An initial PIC assessment is contained with the Concept Design Report (RILEY Ref: 190416-A). The assessment involved initial hydraulic modelling of a single dam breach scenario and concluded that the most appropriate PIC for the proposed dam was high. The assessment recommended that a comprehensive assessment be undertaken during detailed design to confirm that a high PIC applies and for dam safety management and emergency planning.

This work has now advanced, and the report will be made available to support this resource consent application within two weeks of lodgement. The following sections provide a summary of the outputs of that work.

### 3.2 Residential Houses

The NZSOLD Guidelines define destroyed as rendered uninhabitable but does not define uninhabitable. We note that the NZSOLD Guidelines make references to the following publications with regards to damage to residential houses:

- National Institute of Weather and Atmosphere (NIWA, 2010) – provides potential damage curves as a function of building type and flood depth, based on observed data from floods and tsunamis in New Zealand.

An alternative conservative approach is to consider the number of houses that are surrounded by greater than 0.5m of water (above surrounding ground levels). Such inundation could render a house uninhabitable (and there destroyed) due to static water damage.

We have used the latest building outline information from Land Information New Zealand (LINZ) and aerial imagery to assess the number of residential houses affected. We have made our best judgment on weather buildings are residential in nature (i.e. habitable). A total of six residential houses could be destroyed, with up to 14 other houses damage to at least some extent as highlighted on the drawings. The residential houses affected are predominately located in the area approximately 2km to 5km downstream of the dam. No affected residential houses have been identified upstream of Wallis Road.

### 3.3 Critical Infrastructure

The NZSOLD Guidelines state that critical or major infrastructure includes:

- a) Lifelines (power supply, water supply, gas supply, transportations systems, wastewater treatment, telecommunications (network mains and nodes rather than local connections)); and
- b) Emergency facilities - (hospitals, police, fire services); and
- c) Large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community; and
- d) The dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.
Table 1 presents the critical or major infrastructure we have identified downstream of the dam, via a review of aerial photography. Both the State Highway 15 (SH15) bridge and the SH12 road embankment provide the main connections from Kaikohe to State Highway 1.

Table 1: Critical or major infrastructure identified downstream of dam

<table>
<thead>
<tr>
<th>Infrastructure Type</th>
<th>Description</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge</td>
<td>State Highway 15 (Te Pua Road)</td>
<td>Likely to be damaged significantly due to significant overtopping of road.</td>
</tr>
<tr>
<td>Road/Culvert</td>
<td>State Highway 12</td>
<td>Likely to be damaged significantly due to significant overtopping of road.</td>
</tr>
<tr>
<td>Power Supply</td>
<td>High Voltage Transmission Towers</td>
<td>Some potential damage</td>
</tr>
<tr>
<td>Wastewater Treatment</td>
<td>Oxidation Pond</td>
<td>Some potential damage</td>
</tr>
<tr>
<td>FNDC Intake</td>
<td>Nearby SH12 Bridge</td>
<td>Likely to be have significant damage</td>
</tr>
</tbody>
</table>

We consider any damage to critical or major infrastructure is likely to take up three months to restore operation. We consider design, planning, resource consent, building consent, and political issues which could extend the time to restore operation are likely to be largely avoided.

Four bridges were also identified on Wallis Road, the old SH12 bridge, Quarry Road and Cumber Road, but are located on local roads and therefore do not appear to be critical or major infrastructure.

We understand the dam is likely to become a major component of the local economy with water supplied to the horticulture industry. We also understand that in drought conditions the dam may provide town water supply, although it will not be the main water supply during normal conditions. At this stage we have excluded the dam as major or critical infrastructure, however, this could be reviewed at a later stage.

3.4 Natural Environment

The effects of a dam breach on the natural environment downstream may include deposition of sediment and scour within the downstream watercourses, potentially impacting water quality and likely fish habitat. We consider that the majority of the sediment is likely to be deposited in the reach upstream of SH12.

We note that the dam breach flow passes in close proximity to the oxidation pond. The effects of the release of the oxidation pond contents into the natural environment could be significant.

3.5 Population at Risk

As outlined above, a number of residential houses appear to be located in areas where inundation depths are predicted to exceed 0.5m (above surrounding ground levels). The total number of houses meeting the 0.5m threshold is predicted to be 20. We note that a specific floor level survey house not been undertaken, which may reduce the number of houses affected.

As per the latest census in 2018, the population of 4,437 in Kaikohe was located within 1,299 occupied houses, which is approximately 3.4 people/house. Assuming an occupancy rate of 3.5 people/house, the population at risk (PAR) associated with the residential dwellings is 70.
We note that one of the affected residential dwellings appears to operate as an accommodation location (Top Trail Accommodation). For the purposes of this assessment, we have assumed that up to ten people are located at the property.

Community facilities include any permanent buildings other than residential. No existing facilities have been identified that will be affected by at least 0.5m depth of water.

We understand that an innovation and enterprise park is planned within the initial reach downstream of the dam. The proposed layout is outside of the dam breach inundation area.

We have assessed the potential locations downstream of the dam that may contain recreational users and have identified Rawiri Park and the Twin Coast Cycle Trail. We consider the PAR associated with recreational users is likely to be less than five.

The dam breach floodplain crosses several roads, although due to exposure times the PAR associated with road crossings is likely to be low. We consider the PAR associated with road crossings is likely to be less than five.

The PAR may vary considerably depending on the time of day and day of week of a breach. We consider Table 2 provides an appropriate summary of the PAR.

<table>
<thead>
<tr>
<th>Type</th>
<th>Population at Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Houses</td>
<td>70</td>
</tr>
<tr>
<td>Community Facilities</td>
<td>0</td>
</tr>
<tr>
<td>Recreational Users</td>
<td>0 - 5</td>
</tr>
<tr>
<td>Road Crossings</td>
<td>0 - 5</td>
</tr>
<tr>
<td>TOTAL</td>
<td>70 - 80</td>
</tr>
</tbody>
</table>

### 3.6 Potential Loss of Life

Assuming a PAR of 75 and fatality rate of 0.01, per the RCEM (2014) methodology\(^1\), the statistical potential loss of life (PLL) for the worst-case scenario (at night, no warning) is 0.75 and thus loss of life is possible, but not highly likely. For a daytime scenario, adequate warning is more likely than at night and loss of life would be unlikely in these circumstances.

Overall, the risk of loss of life appears low if adequate evacuation plans are in place. It is noted that the PLL calculation in this case does not affect the PIC, as it is already high irrespective of the PLL (i.e. the PIC can be no greater than high).

### 3.7 Discussion and Potential Impact Classification

A sunny-day dam breach will cause significant flooding downstream of the dam, with breach flows in excess of the 50-year flood flows for a distance at least 11km downstream of the dam. A number of residential houses are likely to be flooded, with some destroyed, and significant damage to both SH12 and SH15 is likely to occur. No commercial, industrial or high consequent features such as schools or hospitals have been identified within the dam breach floodplain. We note that the Twin Coast Cycle Trail crosses the floodplain. Overall, the population at risk is likely to be between 50 to 100.

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\(^1\) United States Bureau of Reclamation developed a methodology for estimating PLL entitled Reclamation’s Consequences Estimation Methodology (RCEM). RCEM provides a graphical approach giving the fatality rate as a function of the D x V and amount of warning time (based on measured fatality rates in actual dam breach events). A fatality rate of approximately 0.01 is indicated by the method for a D x V product of 3m³/s.
The PIC assessment is summarised within Table 3 (as taken from the NZSOLD Guidelines). Given that the damage level is major, the PAR is in the range from 11 to 100, the table indicates that the dam should have a High PIC.

Table 3: Determination of dam classification

<table>
<thead>
<tr>
<th>Assessed Damage Level</th>
<th>Population at Risk (PAR)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Catastrophic</td>
<td>High</td>
</tr>
<tr>
<td>Major</td>
<td>Medium</td>
</tr>
<tr>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>Minimal</td>
<td>Low</td>
</tr>
</tbody>
</table>

Notes:
1. With a PAR of five or more people, it is unlikely that the potential impact will be low.
2. With a PAR of more than 100 people, it is unlikely that the potential impact will be medium.
3. Use a medium classification if it is highly likely that a life will be lost.
4. Use a high classification if it is highly likely that two or more lives will be lost.

Consideration of a rainy-day, overtopping breach scenario will be undertaken in the detailed design phase of the project.

3.8 Design Standards

The NZSOLD guidelines recommend that a high PIC dam has an Inflow Design Flood (IDF) between the 10,000-year flood event, and the Probable Maximum Flood (PMF) with the selected design event dependent on the PLL, as outlined in Table 4.

Table 4: Recommended minimum inflow design floods (NZSOLD, 2015)

<table>
<thead>
<tr>
<th>PIC</th>
<th>PAR</th>
<th>PLL</th>
<th>IDF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0 to 10</td>
<td>0</td>
<td>100 to 1,000</td>
</tr>
<tr>
<td>Medium</td>
<td>0 to 10</td>
<td>0</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td>0 to 10</td>
<td>1</td>
<td>2,500</td>
</tr>
<tr>
<td></td>
<td>11 to 100</td>
<td>0 to 1</td>
<td>10,000</td>
</tr>
<tr>
<td>High</td>
<td>No limits</td>
<td>0 to 1</td>
<td>10,000</td>
</tr>
<tr>
<td></td>
<td>No limits</td>
<td>&gt;10</td>
<td>PMF</td>
</tr>
</tbody>
</table>

Notes:
1. PIC – Potential impact classification
2. PAR – Population at risk
3. PLL – Potential loss of life
4. IDF – Inflow design flood

We consider that the minimum IDF for the dam is the 10,000-year flood event (as presented within Table 4). However, we note that a higher design standard such as the PMF can be conservatively selected. We note that due to the relatively small catchment area, the higher standard is unlikely to have significant construction cost implications. The higher standard also future proofs the design.

A risk-based approach to the management of construction floods is recommended, as was applied during construction of the Hopua te Nihotetea Flood Detention Dam in Whangarei.

Seismic ground motions commensurate with a High PIC are outlined in Section 4.3.
An Emergency Action Plan (EAP) should be developed to assist with the evacuation of residents and temporary populations (associated with road crossings or recreational areas) in the event of a dam safety emergency. The potential for loss of life is low if evacuation can be undertaken effectively. We note that future development downstream of the dam may affect the population at risk.

4.0 Geotechnical Site Investigations and Laboratory Testing

4.1 Investigation Phases

Investigations of the dam site has evolved across three phases:

1. An initial review of broad geotechnical issues across the mid-north region as part of the Northland Water Storage and Use project: Pre-feasibility Demand Assessment and Design Study.
2. A site walkover assessment of the dam alignment and reservoir surrounds, five excavated test pits, and limited laboratory testing.
3. Comprehensive geotechnical investigations involving geomorphic field mapping, 13 excavated test pits, and seven cone penetration tests (CPTs). Additionally, four to five machine drillholes with in situ permeability (Lugeon/packer) testing and a suite of laboratory tests were programmed, but unable to commence due to the COVID-19 Level 4 restrictions being put in place.

Results of the investigations are discussed within Section 4.0 of this report.

4.2 Desktop Study and Initial Site Visit

A high-level review of available geotechnical information across the Kaipara and Mid-north was undertaken as part of the wider assessment. This looked into likely ground conditions and the potential variability across several reservoir sites, and for highlighting any known regional hazards that should be considered in the context of shortlisting and concept design for the water storage and distribution scheme. Information was obtained from the following sources:

- New Zealand Geology Webmap v.2.3 https://data.gns.cri.nz/geology/.
- New Zealand Active Fault Database v3.3 https://data.gns.cri.nz/af/.
- New Zealand Landslide Database v.4.1 https://data.gns.cri.nz/landslides/.
- Photoblique images captured in 2017 and 2018.
- Information relating to known recent or historic large dam projects nearby.
- Walkover of selected sites where access was available (MN-10 was visited).

4.3 Preliminary Geotechnical Investigation

Preliminary geotechnical investigations were undertaken by RILEY on 16 October 2019. The purpose of the investigation was to inform a concept design and provide indicative construction costings. It involved:
1. A site walkover and engineering geological inspection of the site and surrounding areas.

2. Five excavated test pits, all at the dam site (TP1 to TP5), two of which within possible landslide debris on the left abutment (TP2 and TP3); and

3. Laboratory testing of selected soil samples.

Test pits were excavated with a 20t hydraulic excavator operated by Far North Roading Limited, excavator under the guidance and supervision of RILEY. The test pits were generally extended to a depth of 5m. A RILEY engineering geologist inspected exposures within the test pits, logging the materials encountered, and any geological structures in general accordance with the New Zealand Geotechnical Society (NZGS) Guidelines.

Laboratory testing was undertaken by GeoCivil Ltd, a local IANZ accredited soil laboratory, on selected samples from the test pits as outlined in Table 1.

Table 5: Laboratory tests on retrieved samples

<table>
<thead>
<tr>
<th>Test Pit</th>
<th>Depth (m)</th>
<th>Test</th>
<th>Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP2</td>
<td>0.8</td>
<td>Atterberg Limits</td>
<td>NZS4402:1986 Tests 2.2, 2.3, 2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hydrometer</td>
<td>NZS4402:1986 Tests 2.8.4</td>
</tr>
<tr>
<td></td>
<td>4.9</td>
<td>Atterberg Limits</td>
<td>NZS4402:1986 Tests 2.2, 2.3, 2.4</td>
</tr>
<tr>
<td>TP4</td>
<td>1.5</td>
<td>Atterberg Limits</td>
<td>NZS4402:1986 Tests 2.2, 2.3, 2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hydrometer</td>
<td>NZS4402:1986 Tests 2.8.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Emerson Class</td>
<td>AS1289.C8.1-1980</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pinhole</td>
<td>ASTM D4647-13</td>
</tr>
<tr>
<td>TP5</td>
<td>1.0</td>
<td>Hydrometer</td>
<td>NZS4402:1986 Tests 2.8.4</td>
</tr>
</tbody>
</table>

Key findings were presented in a report titled Concept Design Report, Proposed Storage Reservoir, Far North Holdings Property, RILEY Ref: 190416, dated 14 November 2019.

4.4 Comprehensive Geotechnical Investigation

A multi-faceted approach to the investigation was adopted, comprising detailed site mapping, and subsurface investigations with test pits, CPTs, and machine boreholes.

The objectives of the investigation were to determine subsurface conditions within:

- The proposed dam footprints and abutments in regards to founding conditions and stability.
- The proposed borrow area in regards to the availability and suitability of the material for embankment fill material.
- The reservoir basin in regards to slope stability under a range of reservoir operating conditions.
- The proposed spillway area in regards to erosion resistance and ability to be excavated.

Although no fatal flaws were identified during this stage, several engineering challenges were identified requiring consideration as part of the detailed design phase.
4.4.1 Geological Mapping and Review of Existing Information

A walkover was undertaken by a senior engineering geologist from RILEY to optimise the location of test pits and machine boreholes on-site. A more comprehensive geomorphic mapping of gullies and steeper slopes was undertaken during field investigations, with surface exposure and subsurface information correlated to published geological map of the area. Results of geologic mapping are summarised in Section 4.0 and plotted on RILEY Dwg: 190272-761.

4.4.2 Test Pits

Thirteen test pits were excavated by Far North Roading using a 20t hydraulic excavator under the guidance and supervision of RILEY. Six of these test pits were excavated as part of the embankment foundation investigation (TP6 to TP11), six as part of the borrow area investigation (TP13 to TP18), and one at the indicative spillway location (TP12).

The test pits were generally extended to a depth of 5m. A RILEY geologist inspected exposures within the test pits, logging the materials encountered, and any geological structures.

4.4.3 Machine Boreholes

A series of four to five machine boreholes drilled into the abutments and dam foundation were programmed but put on hold as a result of the COVID-19 Level 4 lockdown. The boreholes are to be a combination of vertical and inclined drilled to an anticipated maximum depth of about 40 to 50m. Inclined drillholes are proposed in some of the locations to intercept any vertical joint sets in the rock. In-situ permeability (Lugeon) testing of the underlying Kerikeri Volcanic Group rock will be performed progressively throughout drilling.

Standpipe piezometers are planned in all of the machine boreholes for ongoing monitoring of groundwater levels.

4.4.4 Laboratory Testing

Bulk soil samples have been retained from the test pits. These, in combination with selected samples from the drillholes, will be delivered to an IANZ soil laboratory for them to perform a suite of tests to better understand material characteristics and behaviour. Such information will be used to inform detailed design of the reservoir including material suitability for dam construction, strength parameters, and construction processes.

5.0 Investigation Results

5.1 Geological Setting

Based on a review of the GNS Science 1:250 000 Geological Map of New Zealand (QMAP), the site is inferred to be predominantly underlain by Kerikeri Volcanic Group Late Miocene andesite of the Kaikohe - Bay of Islands Volcanic Field. This is bound to the south by younger Kerikeri Volcanic Group Pleistocene basalt of the Kaikohe - Bay of Islands Volcanic Field. It is considered likely that all the volcanoes in this field erupted in the last 300,000-years, and that many are likely younger than 100,000-years.
The site is located on the lower southern slopes of the Te Pua Volcano near the boundary with more recent basalt flows originating from volcanic plugs from the north-east. Due to the eroded nature of the Te Pua Volcano, it is assessed to be one of the older volcanoes in the field (Haywood, 2017). The southern slopes of Te Pua volcano is inferred to be made up by a number of flows emanating from crater the north. With the later flows overlying and overlapping with previous flows. This volcano having become significantly eroded from the time it was formed. Basaltic andesite lava typically having medium viscosity potentially erupting in a combination of explosive and effusive volcanism.

The Kaikohe – Bay of Islands field is considered to be dormant, and not extinct (Haywood, 2017).

The Kaikohe – Bay of Islands Volcanic Field is underlain at depth by structurally complex units of tectonically intercalated sandstone and mudstone of the Northland Allochthon. The materials of the Northland Allochthon are inferred to rest on basement rock of the Caples Terrain.

5.2 Observations from Site Walkover

A large roadcut to the west of the dam site on the flanks of the lava flow that is inferred to form the true right abutment is shown in Photo 1. This is seen to be overlain by a surficial layer of red and red-orange residually weathered basaltic material estimated to be up to 5m to 6m thick. This basaltic material is also seen exposed in roadcuts to the south of the embankment overlying andesitic material exposed at a lower elevation within the stream channel.
Photo 1: Looking east towards andesite lava flow exposed in road cut to the west of the proposed embankment. Surficial reddish and orange brown layer of inferred younger basaltic material sourced from the volcanoes sited to the east exposed near the crest of the exposure.

Less weathered material is exposed in the stream channel downstream of the proposed embankment where the stream forms a sharp bend. The less weathered andesitic material exposed on the outside of the bend is shown in Photo 2. This material broken disjointed rock associated with the lava flows and areas that have become brecciated. Within the andesitic lava the brecciated rock mass consisting of broken up bocks. The angular blocks are inferred to have formed by the fragmentation of an already solidified portion of the lava flow, the solidified blocks surrounded and cemented by the still molten lava. The blocks within the flow are angular, having a platy fabric consistent with an andesitic flow.
5.3 Seismicity

Whilst seismic hazard is generally low by national standards and there are no active faults mapped nearby, seismic aspects will be a design consideration. With reference to the NZSOLD Dam Safety Guidelines, it is considered that the dam will require seismic design with consideration of the following scenarios:

- Operating Basis Earthquake (OBE) – The earthquake for which a dam, appurtenant structure, and gate/valve system that fulfils a dam safety function is designed to remain operational, with any damage being minor and readily repairable following the event. It is considered that an annual exceedance probability (AEP) of 1 in 150 is appropriate for the OBE.
• Safety Evaluation Earthquake (SEE) – 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. It is considered that AEP of 1 in 10,000 is appropriate for the SEE based on the assessed high PIC.

• Controlling Maximum Earthquake (CME) – The maximum earthquake on a seismic source that is capable of inducing the largest seismic demand on a dam.

Due to the long recurrence interval design events, seismic parameters for use in design of the dam are normally established by a site-specific seismic hazard assessment by a technical specialist, using a probabilistic analysis (although this will be dependent on the assessed PIC). Uncertainties and site amplification effects should be addressed.

6.0 Geotechnical Hazards and Site Suitability Assessment

Precedence is a particularly useful indicator as to the likely challenges with building large storage reservoirs in a given area. In general, based on our experience there is some precedence for large storages in the Mid-North, particularly to the north and east in areas of more favourable geology (e.g. Kerikeri irrigation dams which are on volcanic or basement rock). We are not aware of any significant dam safety instances at those locations and have largely performed well. Some challenges have been observed at large dam sites owing to poor foundation conditions in recent geological deposits in and around the Whangarei Region, which have needed to be overcome by significant engineering works.

Whilst a number of potential geotechnical hazards have been considered, based on the investigations undertaken to date, we have not identified any specific natural geotechnical hazards that indicated the possible dam site is unsuitable. The following provides a summary of the primary geotechnical hazards that exist for the proposed dam.

6.1 Foundations and Abutments

A preliminary geological ground model is presented on Section A, RILEY Dwg: 190272-763. This generally comprises residually weathered basaltic andesite of the Kerikeri Volcanic Group. Figure 1 presents a long section through the embankment crest, illustrating the key geological layers within this unit.

On the left abutment, deeply weathered andesite of the Te Pua volcano is prevalent to moderate depth, below which older and more competent rock is inferred. Potentially more recent, intercalated weathered basalt (from the Waimmiti Volcano) and andesite with boulder inclusions is inferred on the right abutment and within the stream channel. Defining the location and composition of these geological contacts, and how these might influence the dam embankment design is a key focus of the drilling programme.

No potentially liquefiable materials, such as saturated loose sand and non-plastic silt soils, were encountered below the foundation and abutments areas of the proposed dam, albeit some soils with a propensity for minor strength loss due to strain-softening are possible. These aspects will be specifically investigated via analysis of the CPT soundings, borehole and laboratory testing information as part of detailed design.
6.2 Borrow Area

Test pits TP13 to TP18 were located in potential borrow areas within the reservoir basin. Soils predominantly comprises several meters of silt with minor clay and trace sand with slight to moderate plasticity, inferred to be residually weathered andesite. Results from earlier lab testing indicate a 20% to 50% clay content, moderate to high plasticity, and were non-dispersive. On that basis, the soils are expected to be suitable as low-permeability earthfill for dam construction (Zone1 Fill), however, may be sensitive to moisture changes during placement and compaction that will need to be considered in the design. Gravel and boulder-sized inclusions, such as those identified in some of the test pits a few meters below ground level, will require consideration in design.

Fill sources will be assessed further as part of the forthcoming laboratory testing.

6.3 Reservoir Leakage

Leakage beneath the reservoir and dam, and associated erosion of soil through open joints within the underlying rock, are seen as potentially the most significant geotechnical issue associated with the project.

Given that the site is underlain by inferred Basaltic-Andesite lava flows with the potential for ash horizons between flows there is the assessed to be a potential for high permeability layers. As there is the potential for explosive episodes, potential ash, lapilli, blocks and scoria layers between flows at depth cannot be discounted. Open joints and associated high permeability layers can potentially occur within andesitic lava flows. These may potentially be encountered within the fresh to highly weathered lava flow material. Open voids may also be potentially present within the lava flow.
Natural springs were observed both within the reservoir basin and on adjacent slopes. These may require local drainage and monitoring, or possibly natural earth lining.

The existing perennial stream indicates losses may be low, with most springs emanating at or above the full supply level. Further, there has been no obvious evidence observed in road cuts upslope or on the margins of lava slopes of potentially highly permeable materials. The majority of roadcuts exposing the upper completely to residually weathered material (refer Photo 3).

![Photo 3: Taken of road cut exposure upstream of proposed embankment in central portion of reservoir.](image)

### 6.4 Slope Stability

Geological hazard maps for the area have been reviewed.

A distinct landslide feature can be observed on the left abutment. Landslide debris was encountered TP2 to a depth of ~5m and had a distinct, highly plastic, clayey silt layer at residual strength near the base (possible failure surface). A similar layer was encountered to 0.7m in TP3 (downslope) but not in TP1 (upslope), suggesting it could be a localised feature which is consistent with the ground morphology, indicating a shallow debris lobe (refer Photo 4). This feature warrants further assessment during detailed design.
No obvious signs of further recent or past slope instability have been observed in historic aerial imagery, during the site walkover at the dam site, or immediately adjacent to the reservoir extent that would be formed by the proposed dam. Notwithstanding this, any further investigations should include assessment of the ground conditions and stability of all slopes surrounding the dam site and reservoir.

Generally, the dam concept does not involve any long-term slope toe excavations or slope surcharging. The soils strengths indicated from the in-situ shear strength testing, do not indicate any obvious slope instability hazard in the slopes adjacent to the reservoir site. Notwithstanding this, further consideration of stability for any permanent cut slopes required to form the spillway and reservoir basin are necessary, as well as stability of temporary excavations required for undercutting of soft unsuitable soils in the dam footprint.

6.5 Further Assessment

Information retrieved from the geotechnical investigations to date have provided a good initial picture of the ground model in the abutments and, to a slightly lesser extent, in the foundation. These aspects will be reviewed, and the design updated to reflect additional information obtained from the machine drilling and laboratory investigations.

7.0 Dam Design Considerations

7.1 Storage Requirements

A conceptual dam layout and elevation are shown within RILEY Dwgs: 190416-762 and -763, with a design dam crest level and flood spillway sill level of RL 256.0m and RL 254.5m, respectively. The proposed upstream dam heel and downstream dam toe levels are approximately RL 240 m and RL 232m respectively.
A long low embankment was required along the western reservoir edge to create enough storage; such a feature will be required for dam crests higher than about RL 252m, which would only provide say 400,000m³ of storage, albeit with less earthworks.

### 7.2 Key Dimensions and Features

The site topography in and outside of the gully, and the ground model, indicate conditions that are favourable for an embankment dam with a flood spillway formed above the true right abutment. We consider that the dam could be a zoned embankment dam with an internal chimney drain and underlying central blanket drain beneath the downstream half of the dam footprint. Filter protection to the outlet pipe via a filter diaphragm would also be incorporated.

Key features of the embankment structure include:

- Zoned earthfill construction utilising site-won residual volcanic soils.
- Maximum height of 24m as measured from the downstream toe to crest.
- 5m wide embankment crest and 3(H):1(V) up and downstream batter slopes.
- 1.5m operating freeboard.
- Vertical intercepting chimney drain with horizontal blanket drain outlet.
- Riprap to the upstream face for wave protection.
- 1.2m diameter main pipe conduit with internal pressure pipe (250mm) for operational flows, and dedicated residual flow pipe (150mm).
- Intake structure at the upstream end, fitted with an upstream control valve.
- Conduit utilised as a temporary construction diversion in combination with a 3m high coffer dam.
7.3 **Earthworks Volumes and Fill Material**

It is estimated that the dam embankment will require in the order of 87,500m³ of earthfill. Within this, specialist filter material for internal chimney and blanket drains, and riprap for upstream wave protection, will need to be imported from a nearby quarry. The bulk of the earthfill embankment should be constructed from cohesive material with a compacted permeability no greater than 10⁻⁷m/s. The silt and clayey soils outlined in Section 5.2 are considered suitable for this purpose subject to further testing.

In addition to the above, volumes related to replacement of foundation undercut and abutment stripping, estimated to be a further 26,000m³, and cut to form the spillway estimated to be in the order of 7,500m³.

7.4 **Foundation and Abutment Preparation**

Excavation down to the underlying unweathered rock for foundation purposes may not be feasible due to its depth. However, as noted earlier, the forthcoming drilling is intended to confirm the depth and composition of the underlying rock and therefore, any requirements for foundation preparation.

Stripping of the gully slopes to stiff natural ground for the purpose of exposing suitable abutments is anticipated to require nominal stripping. Dam fill could then be keyed in and compacted against the abutments.

Where potentially dispersive or high permeability soils are exposed in the abutments, it is envisaged this will be removed completely to the underlying cohesive horizon and benched into the abutment.

Further assessment on the lower abutment slopes will be required to evaluate temporary slope stability when the landslip and dam footprint is undercut. This undercut of unsuitable soft soils will temporarily remove some support to the toe of the abutment slopes before dam fill is placed and compacted.

7.5 **Static and Seismic Stability Requirements**

Table 6 outlines recommended target factors of safety from the NZSOLD Guidelines. Stability analyses will be performed as part of detailed design to demonstrate that these targets are met.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Scenario</th>
<th>Minimum Target</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of construction before reservoir filling</td>
<td>Upstream and downstream</td>
<td>FoS ≥ 1.3</td>
</tr>
<tr>
<td>Long-term steady state seepage, normal reservoir level</td>
<td>Downstream</td>
<td>FoS ≥ 1.5</td>
</tr>
<tr>
<td>Full or partial rapid drawdown</td>
<td>Upstream (likely critical)</td>
<td>FoS ≥ 1.3</td>
</tr>
<tr>
<td>Operating Basis Earthquake (OBE)</td>
<td>Downstream (likely critical)</td>
<td>FoS ≥ 1.0</td>
</tr>
<tr>
<td>Safety Evaluation Earthquake (SEE)</td>
<td>Downstream (likely critical)</td>
<td>Acceptable deformation</td>
</tr>
</tbody>
</table>
8.0 Flood Bypass Facilities

Two possible locations for an emergency overflow spillway were identified during the walkover. One on the north-western edge upstream of the embankment, and another beyond the left abutment. Both would discharge into neighbouring watercourses.

The preferred spillway location is on the right abutment, located within natural ground. The right abutment consists of a ridge separating the proposed reservoir from an adjacent unnamed tributary of the Kopenui Stream. The spillway will discharge to the tributary, which confluences with the Kopenui Stream approximately 1000m downstream of the spillway.

The design philosophy is to:

- Maintain appropriate freeboard to the dam crest during the IDF.
- Ensure that the reservoir is not comprised by velocity-induced erosion within the spillway during the IDF.

The PIC assessment concludes that the minimum IDF is the 10,000-year flood event. However, we note that a higher design standard such as the PMF can be conservatively selected given the relatively small catchment size.

A 1-hour PMF inflow hydrograph has been prepared as presented within Figure 3. The peak inflow is approximately 36m$^3$/s which is comparable to earlier estimates. Further analysis will be undertaken during detailed design to refine these estimates including an assessment of rainfall temporal distributions, and the critical rainfall duration for the reservoir.

![Figure 3: 1-Hour probable maximum flood inflow hydrograph](image)

The spillway has been designed to incorporate both a service and flood spillway as presented within the drawings. The service spillway will be designed to have a very low risk of erosion for the more frequent flood events; the emergency spillway can allow for some erosion, but not such that would allow the uncontrolled release of the reservoir.
The service spillway will have capacity to accommodate a minimum of a 20-year or 50-year event, or preferably up to a 100-year event.

The service spillway has been designed as concrete with a sill elevation of RL 254.5m and a sill length of 10m. The flood spillway sill level is 500mm higher at RL 255.0m, with an overall sill length of 40m. The downstream longitudinal gradient is 10%. Further details are presented within the preliminary design drawings located within the Appendices.

The service spillway may be lined with concrete or similar to provide erosion resistance, but also to potential fish passage for downstream migration of eels. The flood spillway will be grass lined, possibly reinforced subject to further assessment.

We have developed a 2D HEC-RAS (v5.07) model to simulate the design flood. The 2D flow areas extends from the spillway sill downstream to beyond Wallis Road. A Manning’s n value of 0.03 has been used to reflect a grassed lined spillway. A storage area has been used upstream of the 2D flow area to simulate the attenuation effect of the reservoir. The elevation storage curve is presented within the PIC assessment. A cross section of the weir structure that represents the dam within the model is presented in Figure 4. See below:

![Figure 4: Weir structure representing the dam within HEC-RAS](image)

Resultant flood maps are included within the Appendices. The results indicate that the peak velocities are less than 3m/s within the spillway chute. A well grassed spillway should experience only limited erosion at such velocities; however, reinforced grass would provide for greater assurance.

Some specific erosion protection design will be required at the interface between the spillway chute and the downstream natural channel such as riprap. We note that some erosion repair work may be required after extreme flood events when the emergency spillway operates.

Figure 5 presents the velocities and depths predicted by the model near the toe of the spillway for the modelled PMF event, which is reproduced from our Potential Impact Classification report (RILEY Ref: 190272-B, dated 8 July 2020).
The results also indicate that the peak reservoir level will be in the order of RL 255.59m.

The NZSOLD Guidelines recommend that the minimum IDF freeboard is the greater of:

a) 0.9m or
b) The sum of the wind set up and wave run up for the highest 10% of waves caused by a sustained wind speed, which is dependent on the fetch, with an AEP (1) of 1 in 10.

A small wave wall, say in the order of 0.6m, or similar could be included in the design or alternatively the spillway dimensions increased to achieve the required freeboard.

We note that a preliminary assessment indicates that the sum of the wind set up and wave runup is approximately 0.7m, and therefore, the 0.9m freeboard requirement is the controlling design criteria. A detailed assessment of wind set up and wave runup will be undertaken as part of detailed design.

Figure 5: Predicted velocities and depths near the spillway toe for the probable maximum flood event

9.0 Intake Details and Fish Passage

The following section includes details of the intakes and approach to fish passage. The Puhoi Stour report “Mid-North Water Supply Reservoir: Site No. 10 Assessment of Ecological Effects” outlines the potential effects from the proposed dam and the issues that need to be considered in the design of the dam in regard to fish passage.

The key issues to be addressed in the design are the following:

- Migration of eels (elvers) upstream. Both long and shortfin elvers required during peak migration periods (Summer). These elvers are <200mm in size (typically 100mm) and are good climbers even with minor flows. Because the distance upstream in the stream from the coast the migration period may be spread over more months.
• Excluding upstream passage for Gambusia, an exotic pest which is present in the lower reaches of Kopenui Stream.

• Consideration for downstream movement of migrant eels should, however, be included in spillway design to minimise the potential for injuries to occur.

• From the proposed Regional Plan water intakes will need screens with 3mm mesh and velocities into the screen of less than 0.12m/s based on Canterbury Guidelines.

• There are issues relating to construction which will be covered in a separate section.

9.1 Upstream Migration of Elvers

The challenge with upstream passage is that the dam is proposed to have approximately a 18m operating range. When the reservoir is full the barrier is 18m high for the elvers to climb to and the range from full to empty is challenging to design for. For a range of water levels an elver pass can include a floating intake at the reservoir. The operation of the reservoir shows that for more than 75% of the time the reservoir will be within the upper 4m of the range. An elver pass may be feasible with a floating intake to operate in the upper few metres of the range but is not considered feasible for the entire operating level of the dam. When the reservoir water level is below the operating level of the elver pass then a trap and transfer system could be utilised to manage the upstream migration of eel.

Figure 6: Interpretation of monthly simulated reservoir from 1972 to 2019

Alternatively, a trap and transfer of elver could be undertaken without the construction of an elver pass. There may be opportunity for iwi to be involved in or manage the trap and transfer process. The trap and transfer may only operate over peak migration, but adaptive management approach could be used in developing an efficient programme.
The nature of both an elver pass and trap and transfer are challenging, and it is likely that some modifications to the pass or the trap and transfer process will be required during operation. Monitoring of the effectiveness will need to be undertaken and where required modifications to resolve any issues implemented.

**Figure 7: Schematic of Elver trap option**

There are a variety of means to trap elvers. The figure shows a schematic of a concept which provides water from the dam to flow down an elver pass that ends in a trap. The concept is the flow down the elver pass ramp attracts the elvers and excludes other unwanted species. If this approach were adopted the supply pipe could be through the dam and into the trap. This would be located with a pass a minimal distance above the downstream water level to maximise the reservoir water level range it would operate over.

9.2 **The Exclusion of Gambusia**

This is relatively straightforward as either an elver pass or the trap and transfer of elver can ensure any unwanted species of fish cannot migrate upstream of the dam.

9.3 **Spillway Design for Downstream Adult Eel Migration**

The shaping of a spillway channel and downstream structures that are part of the spillway will consider what is required to minimise damage to eel. This will relate to depth of flows and any structures with the flow channel downstream and back to the river.

9.4 **Intake**

The dam will operate with a residual flow requirement of 14.7l/s and a peak supply flow of 60l/s. The intake for the supply flow will need to comply with fish screening requirements as this water is being removed for irrigation use. The elver pass and/or trap and transfer operation will provide some residual flow but likely this will be significantly less than 14.7l/s. The supply flow will need a pipe approximately 250mm diameter. This could be located at the base of the pond or alternatively a pipe could be laid down the side slope of the pond with multiple openings to enable water to be taken from different reservoir levels. The challenge with this approach will be meeting the fish screening requirements for a 3mm mesh and the velocity into the screen.
The residual supply pipe will need to be approximately 150mm in diameter and will require a means to dissipate the energy at the outlet back to the river downstream.

Both pipes will require a valve and flow meter to control and measure the flows released.

9.5 **Screens on Intakes**

The intake to the supply pipe will need to include a screen to comply with proposed regional plan to keep fish in the stream and also to avoid impingement onto the screen. This includes a requirement of a 3mm mesh screen. Given the small gaps in the screen there is a risk of the screen blocking and therefore, likely that a cleaning system will also be required.

If the intake is a single intake located at the invert of the pond, then a rotary screen may be used (refer to schematic and photo). The example below is a rotary screen which has a propeller to turn the screen and a set of brushes on the inside and out keep the screen clean. For the option of an intake that can take water at different levels within the reservoir a pipe with 3mm mesh screened openings may require a means for manual cleaning of the mesh.

![Figure 8: Rotary screen example - schematic](image)

![Figure 9: Rotary screen example - photo](image)
10.0 Construction

10.1 Overview

Construction of the dam embankment and reservoir basin will involve site clearance combined with earthworks and stockpiling of material. Approximately 14ha of ground will be disturbed during the construction process. A tributary of the Kopenui Stream runs through the proposed construction site, and hence erosion and sediment control measures are needed to prevent the discharge of uncontrolled suspended sediment to the waterway during the construction process.

The main earthworks activities include:

- Clearing and stripping of topsoil from the dam, spillways and borrow area.
- Sub-excavation of unsuitable foundation materials and placement in a designated unsuitables area.
- Excavation of the spillways and borrow area and placement of fill within the dam (or the unsuitable area).

The earthworks will entail cut depths of up to about 5m or more in the borrow area, and fill depths of up to about 24m for the dam embankment. With the following volumes:

2. Cut for foundation strip and landslip remediation: 26,000m$^3$.
3. Cut for spillway: 7,500m$^3$.

10.2 Indicative Construction Staging

The following provides an indicative construction sequence for the proposal. Note this will be subject to review once detailed design is completed, and construction management plans are prepared by the contractor.

10.2.1 Stage 1 – Site Establishment

The sequence of work anticipated is as follows:

a) Initial establishment for the site offices, storage areas, and mobilisation of equipment. This may occur progressively as plant is required because bulk earthworks can only commence when services diversion is complete.

b) Form stabilised construction entrance(s) utilising silt fences or similar as required.

10.2.2 Foundation Preparation and Stream Diversion

a) Construct permanent culvert in dry conditions on left abutment, including concrete works and outlet structures (and downstream energy dissipation).

b) Excavate temporary stream diversion leaving each end in place. Limit clearing to allow construction of diversion channel only. Line with fabric. Bund off upstream and downstream ends and divert stream into temporary channel. Protect bund from overflows using fabric or rock.

c) Install surface water diversions and silt control measures for unsuitable disposal area and main borrow area.
d) Construct cofferdam at upstream face to provide flood protection.

During this phase it is envisaged that fill will be sourced from the main borrow area.

10.2.3 Bulk Filling Stage

a) Install additional silt control works for dam construction as necessary (mainly decants/supersilt fences).

b) Sub-excavate for dam foundations in stream area (sub-excavate and backfill) initially.

c) Continue foundation preparation over dam footprint. Place fill in dam to final crest level.

d) Stabilise borrow area and unsuitable disposal area as soon as no longer required (topsoil/grass).

e) Topsoiling/grass of dam faces.

f) Decommission/remove silt controls at dam and other areas.

During this bulk filling stage, it will be necessary to commence bulk excavation of the spillways and other cut areas as needed. The relative sequencing of excavation work for the spillways and borrow area will depend on the contractors preferred methodology. The recommended staging of works within the spillway is described below:

a) Initially strip topsoil and excavate upper sections of spillway only. Prior to stripping, install silt fences and/or supersilt fence.

b) Strip and excavate lower sections in stages, placing further supersilt fences. Maintain riparian vegetation buffer beside stream if required. Maintain vegetation on steep slopes (particularly left abutment) until stripping is carried out in that area.

b) As soon as practical, install concrete works on upper sections and then reinforced grass. Stabilise progressively downslope as excavation/trimming work completed in each section.

d) Clear vegetation beside stream and complete works at spillway/stream junction (excavation, trimming, and rock placement). Undertake works in low flow period only. Minimise period of exposure of works within the stream itself.

10.3 Erosion and Sediment Control

As part of the earthworks contract, the contractor will be responsible for providing adequate sediment and erosion control measures to protect downstream environments from excessive sedimentation and water quality degradation.

Planning of erosion and sediment control measures will occur prior to any earthworks commencing. The key objectives of sediment control are to minimize stormwater runoff, protect the land from erosion and prevent sediment from leaving the site. The measures shall ensure that:

- The diversion of clean stormwater away from the site is maximised.
- All contaminated water is intercepted and treated.
- All control measures and discharges shall be undertaken so as not to cause erosion.
The effects of weather and the use of heavy earthmoving equipment will have the potential to generate sediment runoff until the site has been stabilised (i.e. top soiled and grassed, revegetated, or gravelled). The staging and sequencing of earthworks activities are, therefore, important to minimise adverse environmental effects from construction. In particular, the staging of specifically designed erosion and sediment control devices prior to, and during, construction is fundamental to the success of any preventative measures.

A comprehensive Erosion and Sediment Control Plan (ESCP) in accordance with Auckland Council’s Erosion and Sediment Control Guide for Land Disturbing Activities in the Auckland Region Guideline Document 2016/005 (GD05) will be prepared prior to construction to ensure the construction contractors will implement robust practices for the prevention and management of sediment laden water discharges. This will detail the type, size, location, operation and maintenance of erosion and sediment control measures at particular work areas within the site.

11.0 Limitation

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

Recommendations and opinions in this report are based on data from limited test positions. The nature and continuity of subsoil conditions away from the test positions are inferred, and it must be appreciated that actual conditions could vary considerably from the assumed model.
APPENDIX A

Drawings
TYPICAL CROSS SECTION

INLET AND FISH PATH DETAIL

FILTER DIAPHRAGM DETAIL