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APPENDIX K

Hydraulic and Geotechnical Assessment – Riley Consultants Limited

MOTUKAWA RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT MOTUKAWA HYDRO-ELECTRIC POWER SCHEME

Engineers and Geologists

Note: Since the lodgement of the resource consent applications for the Motukawa Hydro-Electric Power Scheme in November 2021 (being the application to which this technical assessment relates), the proposal by Manawa Energy has been amended to retain the consented maximum water take from the Manganui River as 5.2 m^3/s. The Assessment of Environmental Effects lodged with the resource consent applications has been amended to reflect this change, but the technical assessments associated with the application (including this one) have not been amended. However, all effects on the environment will either be the same or less than previously assessed in the lodged technical assessments.

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MOTUKAWA RACE
HYDRAULIC AND GEOTECHNICAL ASSESSMENT
MOTUKAWA HYDRO-ELECTRIC POWER SCHEME

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Contents

Appendices

MOTUKAWA RACE
HYDRAULIC AND GEOTECHNICAL ASSESSMENT
MOTUKAWA HYDRO-ELECTRIC POWER SCHEME

1.0 Introduction

Riley Consultants Ltd (RILEY) has undertaken this hydraulic and geotechnical assessment at the request of Trustpower Limited (Trustpower). The report details an assessment undertaken for the Motukawa Race (the Race) and Lake Ratapiko, which form part of the Motukawa Hydro Electric Power Scheme (HEPS). The report is intended to support a resource consent application for the reconsenting of the Motukawa HEPS.

A core deliverable in undertaking and presenting the results of this assessment is to examine how the presence and operation of the scheme may have potential adverse effects arising from hydraulic conditions, on the surrounding environment. The assessment then examines how the potential effects can be managed.

The scope included:

- Development of a calibrated two-dimensional hydraulic model of the Race to estimate water levels and velocities within the Race under normal operating and flood conditions.
- Assessment of potential hydraulic improvements to the Manganui Weir and Race to support a flow take of 7.5m^3/s.
- Site inspections.
- Assessment of race freeboard.
- Assessment of potential flooding effects on adjacent properties.
- Assessment of mitigation options to minimise flooding effects on adjacent properties.
- Assessment of likely erosion issues within the Race and management option to mitigate these issues.
- Assessment of the likely geotechnical issues within the Race.

The scope did not include:

- Any detailed design of race or fish passage improvements.
- Any physical geotechnical investigations.
- The power conveyance aspects of the scheme, being the tunnel, penstocks, powerhouse, and discharge from the scheme.

1.1 Report Structure

This report utilises the following structure.

Section 2: Description of Catchment and Setting. This section describes the physical setting, features, and catchment characteristics relevant to this assessment. It then discusses how the scheme interacts with these characteristics and features.

Section 3: Scheme Description. This section describes the scheme components relevant to this assessment and how they operate.

Section 4: Assessment Methodology. This section describes how this assessment has been undertaken and the adopted performance criteria against which assessment results have been compared.

Section 5: Race Flow Capacity Assessment. This section examines the capacity of the Race to convey flow.

Section 6: Performance Assessment. This section examines race flow under several scenarios including normal and flood operation. This is compared against the performance criteria defined in Section 4.0, to determine where there is potential for adverse effects to arise.

Section 7: Management of Potential Adverse Effects. This section examines how the potential adverse effects, identified in Section 6 can be effectively managed.

Section 8: Summary. This section summarises the core outcomes from the assessment and the associated recommendations.

The report also contains a number of Appendices that include greater detail and supporting technical information. The contents of these appendices are summarised as follows:

Appendix A: Review of available time series data.

Appendix B: Review of the catchment and delineation into appropriate sub-catchments.

Appendix C: Development of a HEC-HMS (Hydrologic Engineering Center – Hydraulic Modelling System)[1] hydrological model to estimate various sub-catchment runoff hydrographs.

Appendix D: Development of a HEC-RAS (Hydrologic Engineering Center – River Analysis System) hydraulic model to simulate the passage of water along the Race from the Manganui River through to the lake, and further downstream.

Appendix E: Selection of appropriate calibration events.

Appendix F: Calibration to observed flood events and selection of the most appropriate catchment parameters.

Appendix G: Assessment of design rainfall using a range of methods, comparison of the results, and selection of the most appropriate rainfall.

Appendix H: Determination of the inflow design flood and comparation of the results with results from other flood estimation methods.

Appendix I: Assessment of erosion within the Race.

Appendix J: Hydraulic Model Result Maps

[1] US Army Corps of Engineers, Hydrologic Engineering Center

2.0 Description of Catchment and Setting

The following section briefly describes the local landform and associated catchment. Further detail is provided in Appendix B. RILEY Dwg: 18MTK/ENH-201 presents the scheme location in a regional context and Table 1 provides a summary of the catchments.

Table 1: Catchment Area Summary

Catchment	Area (km^2)	Source
Salisbury Drain at Race	1.5	RILEY[1.]
Mangaotea Stream at Race	9.0	RILEY[1.]
Mako Stream at Dam	12.4	RILEY[1.2.]
Manganui River at Weir	81	NIWA
Manganui River/Mangaotea Stream Confluence	86/12	NIWA
Manganui River/Waitara River Confluence	295/939	NIWA
Waitara River Mouth	1139	NIWA

[1]Derived as part of this assessment using LiDAR.
[2]Includes catchment areas intercepted by the race. Excludes Mangaotea catchment.

2.1 Landform and Drainage

The landform and associated drainage pattern within which most of the Motukawa HEPS is located consists of a broad floodplain. A low watershed separates the western portion from a shallow basin to the east.

2.1.1 Western Portion

The western portion of the land on which the scheme is located is dominated by near flat farmland, largely within the Mangaotea floodplain. The Manganui River forms an irregular boundary along the north-western side of the plain.

This plain is bounded to the south by low hills with flow generally draining in a north-westerly direction. The main natural stream that drains the plain is the Mangaotea Stream, a tributary of the Manganui River, which in turn drains to the Waitara River. An extensive man-made drainage network that collects surface and groundwater directs flow to the Mangaotea Stream. A second much smaller tributary (the Salisbury Drain, which has been extensively modified), lies to the west of the Mangaotea Stream. The dominant land use within the floodplain is dairy farming.

2.1.2 Eastern Portion

The eastern portion consists of a shallow basin surrounded to the north, east, and south by low hills. This basin forms the headwaters of the Mako Stream (a tributary of the Waitara River). The Mako Stream penetrates the low hills to the south of the basin, draining the associated catchment in a south-easterly direction.

The dominant land use within the basin is dairy farming.

Beyond this basin to the east, after passing over the low hills that bound the basin, the landform drops away into the Waitara River valley that flows in a north-westerly direction to the river mouth on the northern Taranaki Coast.

2.2 Scheme Interaction with Landform and Catchment

The hydraulic components of the scheme are formed within the landform and associated drainage system (man-made and natural) discussed above and as presented within RILEY Dwg: 18MTK-ENH-202. Therefore, the scheme components interact with these landform and drainage features. The following sections describe how the scheme interacts with these landform and drainage features.

The two most significant hydraulic components within the scheme, and relevant to this assessment, are the Ratapiko Dam and the Race.

2.2.1 The Race

The Race conveys water from an intake on the Manganui River to Lake Ratapiko (which has formed behind the Ratapiko Dam). It flows in an easterly direction and is approximately 4.6km long. The Race is predominately located within, and passes through, rolling to flat pastureland within the Mangaotea floodplain before passing through a low watershed into the basin at the headwaters of the Mako Stream catchment.

As the Race crosses the Mangaotea floodplain, it passes over the Mangaotea Stream via an aqueduct. The Mangaotea Stream is the most significant watercourse that bisects the Race. Under normal flow and operating conditions, the Mangaotea Stream flows pass under the aqueduct. However, during extreme flood events, the Mangaotea Stream has been observed to overtop its banks and flow into the Race.

The Race intersects a significant local catchment area (in addition to the potential overflow from the Mangaotea Stream during flood events). The majority of the catchment that drains into the Race is located to the south of the Race. During flood events, stormwater runoff from the local catchment enters the Race.

The two largest tributaries that feed directly into the Race are at chainage (CH) 1300m (refer to Photo 1, 1.3km from the intake, Salisbury Drain immediately upstream of the Salisbury Road Culvert) and at CH 3400m (refer to Photo 2, 3.4km from the intake, unnamed drain upstream of Tunnel 2). Prior to the Race construction and modifications to the drainage system for farming purposes, these two tributaries were likely to have discharged to the Manganui River and the Mangaotea Stream, respectively.

Historic land drainage and farming in general is likely to have resulted in the modification of the drainage system surrounding the Race. The land drainage system includes farm drains, which drain to the Race or to the Mangaotea Stream. During flood events the Race becomes the main conveyance system for flow derived from the local catchment. Within the Mangaotea floodplain, the Race has the potential to overtop during extreme flood events once its capacity is exceeded and discharge into the floodplain (even with the river intake gates closed). Such flood waters will confluence with the Manganui River further downstream or drain back into the Race once levels in the Race recede.

Photo 1: CH 1300m Tributary as viewed from just upstream of the confluence with the Race (June 2020).

Photo 2: CH 3400m Tributary as viewed from left bank of the Race across to the right bank (June 2020).

Because the Race is largely situated within a floodplain, it interacts with localised flooding in two main ways. Firstly, it intercepts local catchment flow either directly or through the minor tributaries and drains that discharge into the Race. Within the capacity limits of the Race, this means the Race will largely reduce flood risk downstream of its alignment.

Secondly, through water abstraction from the Manganui River, it has the potential (if not actively managed through scheme controls) to add more flow into the local catchment than is derived directly from that local catchment. If the combination of the diverted and local flow exceeds the capacity of the Race, there is the potential that the extent of flooding on adjacent land is increased.

2.2.2 Ratapiko Dam

The dam embankment was constructed across Mako Stream in 1927 and impounds Lake Ratapiko. The resulting reservoir sits within the basin from which the headwaters of the Mako Stream originate.

Several modest tributaries of the Mako Stream feed into Lake Ratapiko predominantly to the north and east of the reservoir. The presence of the dam and associated reservoir captures the flow from these tributaries during normal flow conditions. During large floods in the catchment, flow unable to be held within the reservoir is spilled over the Ratapiko Dam spillway to re-join the Mako Stream.

3.0 Scheme Description

The Race was constructed in the 1920s and is mainly located within cut, with three short tunnel sections and one aqueduct at the Mangaotea Stream (Mangaotea Aqueduct).

The Race is typically 3m to 8m wide (at normal operating water levels) and is concrete-lined in places. The Race has a relatively steep longitudinal gradient with an average gradient of 0.3% (1:300), although the gradient varies significantly in places. The narrower and steeper sections are more common in the earlier (western) portions of the Race, with the wider and shallower gradient sections more common in the later (eastern) portions.

The most significant structures/components within the Race and lake are summarised within Table 2 and are discussed further as follows.

Table 2: Race and Lake Structures/Components

Component	Race Chainage (m)	Comment
Manganui River	0	Upstream end of race.
Intake Structure	90	Controls abstraction from the Manganui River into race.
Tariki Weir	720	Weir assists race flow measurements.
Drop Structure	1270	Race invert level falls by approximately 1m over a step.
In-Race Generator	2220	Mini hydroelectric power scheme within race.
Mangaotea Aqueduct	2850	Aqueduct over the Mangaotea Stream.
Lake Ratapiko	4630	Downstream end of race.
Ratapiko Dam/Spillways/Penstock Intake	-	Dam, spillways, and penstock intake control water within lake.

The Manganui Weir is located on the Manganui River, immediately downstream of the Race forebay. During flood events, the weir is designed to overtop. There are two fish passes around the weir, one on the left abutment, and one on the right. The required minimum compensatory flow is 400L/s.

The current maximum consented take from the Manganui River into the race is 5.2m^3/s. The rate at which flow is abstracted from the river is controlled by two vertical gates at the intake structure. The two gates are identical in size.

The Tariki Weir is located approximately 600m downstream of the intake structure. The weir provides a stable upstream water level to allow water level readings and gaugings to be undertaken. The record of abstracted flow is derived from these water level readings and gaugings.

The In-Race Generator was constructed in 2005/2006, with a design head of approximately 6m. The level of Lake Ratapiko influences the Race water level as far upstream as the In-Race Generator.

A spillway from the Race is located at the Mangaotea Aqueduct. The spillway consists of a vertical gate and concrete-lined channel, which discharges to the Mangaotea Stream.

Along the length of the Race, it is crossed by several bridges and passes through short culverts and tunnel sections. There are:

- Eight bridges, including two road bridges;
- Three culverts; and
- Three tunnels.

The Race discharges into Lake Ratapiko. Lake Ratapiko is separated by the Ratapiko Road embankment with culverts connecting the two portions of the lake. The consented maximum normal operating water level at Lake Ratapiko is RL 198.70m.

The main spillway for Lake Ratapiko consists of a concrete overflow weir and is located near the southernmost limit of the reservoir. The Ratapiko Dam that forms Lake Ratapiko is a short distance to the east of the main spillway and consists of an earth embankment. A second auxiliary spillway is located near the dam’s right abutment.

The local Mean Sea Level vertical datum (Taranaki 1970) has been used within this assessment, to provide consistency with other ongoing measurements taken at Lake Ratapiko (i.e., lake water levels, monitoring instrumentation, and deformation surveys).

3.1 Scheme Operation

The primary operational control on abstracted flow into the Race, and subsequent reservoir, are the Race intake gates located upstream of the Manganui Weir. The intake gates control inflow to the Race in order to achieve several criteria:

- To ensure that the required residual flow, via the fish pass, is able to be maintained (subject to the hydrology of the Manganui River).
- Control inflow to ensure compliance with the consented maximum abstraction (existing maximum = 5.2m^3/s).
- Control inflow to ensure that unnecessary spill from Lake Ratapiko is avoided.
- Control inflow to limit the potential for the scheme to contribute to localised flooding.

The third and fourth points above are the main operational considerations relevant to this assessment, as they relate to flood management.

The other main area of operational control for the scheme is the rate at which water is drawn from the reservoir for electricity generation. In broad terms this equates to:

- When the reservoir is at or near maximum level, the station will operate at or near maximum available output.
- When the reservoir is at or near minimum level, the station will not operate or only operate at or near minimal output.
- When the reservoir is within normal operating levels, the station operates on a variable basis to meet energy and market demands.

While provided here, these operational output controls associated with generation are not relevant to this assessment.

3.1.1 Normal Operations

For the purpose of this assessment the following definition has been adopted for normal operations.

- Water levels within the scheme are at, or below, maximum operational levels. There is therefore no spill occurring at any point within the scheme.
- Modulation of the intake gates at the Manganui River is only occurring for the purpose maintaining the water level behind the weir. Modulation is not occurring to manage flows to avoid flooding and spill.
- There are no significant faults within the scheme that would impact the ability to convey and manage flow.

3.1.2 Operation During Floods

The historical operating regime for the intake gates consists of:

- During higher river flows, including small floods, the gates are typically only partly open to ensure that the existing consented maximum abstraction of 5.2m^3/s is not exceeded.
- During significant flood events, the gates are typically closed so that inflow to the Race is limited to runoff from the local catchment area.

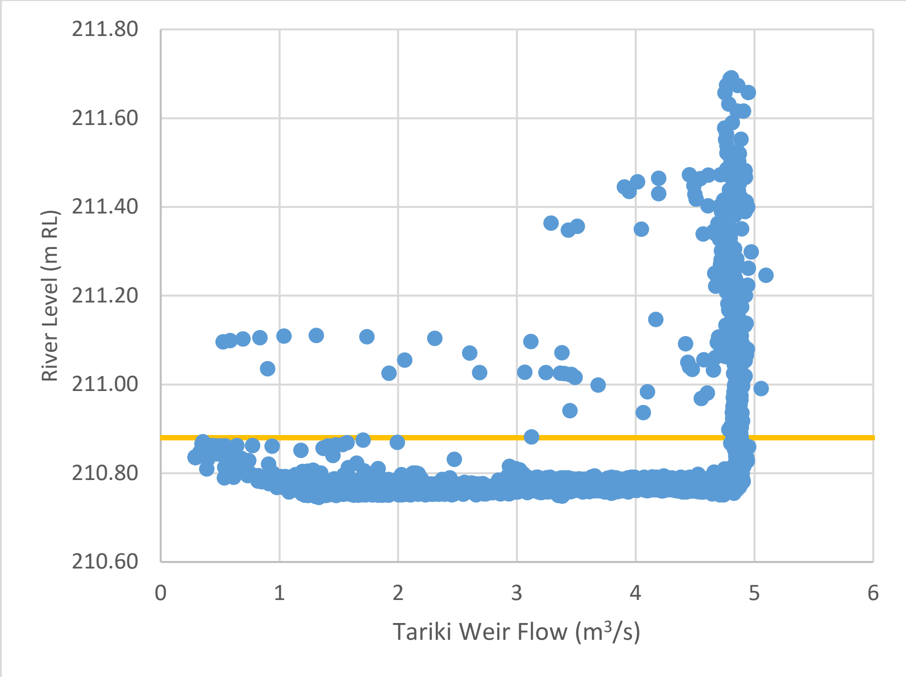
Furthermore, should the specified Maximum Control Water Levels along the Race be exceeded (refer to Table 3, Section 4.0) due to local catchment flood inflow, the intake is shut.

We understand through discussions with Chris England (Trustpower) that the gates:

- Are typically only partly opened to limit abstraction to approximately 5.0m^3/s (or less).
- Are typically not clear of the water level when passing approximately 5.0m^3/s.

Figure 1 demonstrates that Trustpower typically abstracts a maximum of 4.8m^3/s from the Manganui River, up to a river level of around RL 210.80m. At higher river levels/flows, Trustpower restricts the abstracted flow to less than 5.0m^3/s by partly closing the intake gates. The maximum recorded flow abstraction, shown within Figure 1, is 5.1m^3/s.

Figure 1: January 2018 Data (Calendar Month)



4.0 Assessment Methodology

4.1 Site Inspections

Two site inspections of the Race and Lake Ratapiko were specifically undertaken as part of this assessment. The initial site inspection was undertaken on 4 February 2019. At the time, river flow was low and approximately 0.5m^3/s was abstracted into and flowing within the race. The intake gates were only partly open.

A second site inspection was undertaken on 8 June 2020. At the time, the Race flow was in the order of 4.9m^3/s and the lake level was approximately RL 198.4m (0.3m below the consented maximum normal operating water level). While not directly observed, given the flow was close to the operation limit utilised by Trustpower, it can be inferred that the intake gates were partially closed to control inflow.

Further relevant details from the site inspections are referred to throughout this report.

4.2 Performance Criteria

To understand and assess the potential for the scheme components and operation to induce potential adverse effects, it is important to define the performance criteria for the hydraulic conveyance systems and associated structures. Current and future operational scenarios can then be assessed against these performance criteria to give confidence that potential adverse effects are being appropriately avoided or controlled.

4.2.1 Freeboard to Race Crest

One of the main performance criteria for the Race is the provision of adequate freeboard to prevent overtopping during normal operating conditions. Performance criteria relevant to flooding conditions are discussed in Section 4.2.4. We note that no part of the Race meets the definition of a large dam as defined by the Building Act. The Building Act defines a large dam as a dam that has a height of four or more metres and holds 20,000 or more cubic metres volume of water or other fluid. The largest fill embankment on the Race (and potentially the only fill embankment) is located immediately upstream of the In-Race Generator. The fill height in this area appears to be no more than 3.3m (based on a review of 2018 LiDAR information).

During normal operation, (refer 3.1.1) we consider that, for a race of the scale and characteristics of the Race, a 300mm to 500mm freeboard from the water level to the crest is typically adequate. As a comparison, the Deep Stream Canal as part of the Waipori HEPS has a design freeboard of 300mm, however, we note that this has been augmented with a number of spillways. As discussed in Section 3.1, flow abstraction to the Race during normal operation is controlled by the intake gates reducing the risk of the Race level over filling. The Race is also predominantly in cut, not fill, so there is minimal risk that the Race embankment would erode, and breach should overtopping occur.

In summary, given the ability to control flow by use of the intake gates, and the low risk of race breach, we consider the lower end of the range given above, of 300mm minimum freeboard during normal operating conditions, is appropriate.

4.2.2 Freeboard to Bridges/Culverts

As for the Race embankments, there is minimal need for significant freeboard around bridges and culverts. There is minimal potential for significant floating debris to be entrained into the Race flow that might pose a risk to bridges and culverts. As such, a freeboard of no less than 300mm for bridges is also considered adequate. Culverts can safely operate fully surcharged (no freeboard) provided they are designed to do so.

4.2.3 Erosion

Two possible erosion/slumping processes/mechanisms have been identified that could occur along the Race. These are as follows:

1. Erosion/scour due to high flow velocities.
2. Slumping due to frequent variations in water levels (i.e., independent of velocities).

In general, the longitudinal gradient of the Race is steeper upstream of the In-Race Generator than downstream, and therefore higher velocities occur upstream of the In-Race Generator.

The Culvert Manual (Ministry of Works and Development, 1978) indicates that a velocity of 1.4m/s is at the upper limit of non-scouring velocities in clays or organic clays, the type of which generally form the banks of the Race (where formal lining does not exist). A flow velocity of 1.4m/s has therefore been adopted as a limit above which there may be potential for erosion.

Water level in the Race is influenced by intake flow and local catchment inflow during inclement weather. Lake levels also have an influence on race water levels as far upstream as the In-Race Generator. The most likely situation for relatively rapid changes is race water level rises resulting from local inflow during high intensity rainstorm events. In these events, the Race water level will largely respond in a similar manner to the local drains and tributaries. It is also during such events that some of the higher flow velocities are anticipated to occur and hence we consider that flow velocity is the dominant contributor to potential erosion and slumping mechanisms.

4.2.4 Flooding

As discussed in Section 2.2, the Race interacts with localised flooding in two main ways:

- By intercepting local catchment flow and thereby potentially reducing the risk/extent of flooding on adjacent land.
- By contributing additional water, abstracted water from the Manganui River, and therefore potentially increasing the risk and extent of flooding on adjacent land.

The core performance criteria for flooding is, therefore, that the abstraction of water from the Manganui River into the Race does not increase the risk or extent of flooding on adjacent land. A range of water level monitoring exists along the Race in order to monitor the Race against the operational limits.

The existing resource consents contain several maximum race water level limits (refer to Table 3) to manage the potential for the Race flow to exacerbate flood risk to adjacent land. This assessment has not sought to apply these limits directly (i.e., the model allows these levels to be exceeded), but instead the limits have been used in a comparative manner to indicate when the risk of potential adverse effect may increase.

Table 3: Control Water Levels defined in Existing Resource Consents

Location	Maximum Control Water Level (m RL)
Salisbury Road Bridge	205.20
Mangaotea Road Culvert	199.30
Mangaotea Aqueduct	199.25
Lower Mangaotea	199.15

4.3 Modelling

The analysis and modelling, undertaken as part of this assessment, has been conducted with consideration of the various performance criteria provided in Section 4.2. Technical details of the modelling undertaken are provided within the Appendices (Appendix A to J).

In the various scenarios assessed, we have made the following assumptions within the HEC-RAS hydraulic model:

- A steady compensatory flow of 400L/s is released downstream of the Manganui Weir.
- For each race abstraction flow assessed, the unsteady flow upstream boundary condition within the Manganui River has an additional 400L/s included to account for the compensatory flow (e.g., a race abstraction of 7.5m^3/s requires a river flow of 7.9m^3/s).
- A static lake level of RL 198.70m (i.e., the maximum normal operating level) during normal operating conditions.
- An initial lake level of RL 198.70m (i.e., the maximum normal operating level) during flood conditions.
- During flood conditions, electricity generation at the power station continues, with a steady flow abstraction of 7.0m^3/s.

It is noted that during very large flood events (in excess of the 100-year event), operation of the power station may not be possible, as presumed in the last point above. In such events, it is also expected that abstraction from the Manganui River will have ceased.

5.0 Race Inflow Capacity Assessment

We have used the calibrated model to estimate the possible abstraction flow, for a range of Manganui River flows, assuming that the gates are 100% open. Therefore, this portion of the assessment defines a likely upper limit of what could be conveyed into the Race from the river, without modifications in the form of capacity improvements to the conveyance system. Table 5 provides a summary of the results.

The results show that the flow take is sensitive to the Manning's 'n' value, as presented within Table 5. The Manning's 'n' value represents the roughness of the Race, which effects the water velocity and level for a given flow. In simple terms, if the Race is rougher, the flow velocity is reduced, and the water level correspondingly increases to pass the flow.

Chow 1959 provides a guide for selection of Manning's 'n' value, using a description to describe the channel type. Table 4 provides a summary of channel descriptions from Chow 1959 that we consider are most appropriate for the race.

Table 4: Manning's n Summary

Description	Minimum	Normal	Maximum
Concrete-lined, finished, with gravel on bottom.	0.015	0.017	0.020
Excavated channel, earth, straight, uniform clean (after weathering)	0.018	0.022	0.025

From this table, and also based on our experience, we would anticipate that the Manning's 'n' value would be in the range of 0.015 to 0.025, along the length of the Race. For this assessment, we have assessed both a Manning's 'n' value of 0.015 and 0.025 for the Race section, noting that the value is most likely to vary along the length of the Race. We note that the calibration process, where modelled results are compared to actual measured information, indicates that the value is likely to be towards 0.015 within the concrete-lined intake area (i.e., the Race reach which controls the flow take).

Table 5: Existing Modelled Flow Take (Gates 100% open)

Manganui River Flow (m^3/s)	n=0.015		n=0.025	
	Modelled River WL (m RL)	Flow Take (m^3/s)	Modelled River WL (m RL)	Flow Take (m^3/s)
5.6	210.57	5.2	210.88	5.2
7.9	210.90	7.3	210.98	5.7
10.0	210.98	7.77	211.04	6.03

Note: Manganui Weir Crest RL 210.88m

This portion of the analysis indicates that the abstraction capacity of the existing Race is broadly in the range of 5.2 and 7.3m^3/s when the Manganui Weir is not spilling (i.e., the water level behind the weir is below RL 210.88). Abstraction above this range would only occur due to an elevated water level behind the weir during floods, helping to drive more flow through the intake gates. In these times, the weir will be spilling.

5.1 Potential Race Hydraulic Improvement

Because the Race is relatively steep and has several hydraulic structures along its length, it can be largely separated into sections in terms of options for improving hydraulic performance.

5.1.1 Upstream of Tariki Weir

This upper section of the Race is the most important in terms of flow capacity as it defines how much water can be abstracted from the river. There is little value in improving the hydraulic performance of the Race further downstream if inflow is constrained within this section.

Observations on-site indicate that the Tariki Weir has a backwater effect well upstream of the weir. The weir currently acts as a flow control device to assist with upstream flow gauging. Removal of the weir would lower the water level upstream at a given flow. The weir has been removed previously on a temporary basis to lower the water level within the Sediment Pond (also known as Ayling's Pond, located upstream of the weir) during maintenance works. We understand the weir consists of a steel plate and can be lifted out with a Hiab (a crane truck). Photo 3 shows the water level drop over the weir.

Photo 3: Flow over Tariki Weir (February 2019).

Modelling indicates that the modification or removal of the Tariki Weir will increase the flow take. Table 6 summarises the improved flow takes, with a river flow of 7.9m^3/s. Again, the model results indicate that the flow take is sensitive to the Manning's 'n' value. If the Manning's 'n' value is close to 0.015, removing the Tariki Weir will be sufficient to allow a flow take of 7.5m^3/s, when the river flow is 7.9m^3/s, assuming that the fish pass passes a steady flow of 400L/s.

Table 6: Modelled Flow Take (Gates 100% open)

Manganui River Flow (m^3/s)	Scenario	n=0.015		n=0.025	
		Modelled River WL (m RL)	Flow Take (m^3/s)	Modelled River WL (m RL)	Flow Take (m^3/s)
7.9	Proposed – Tariki Weir removed	210.84	7.5	210.97	5.9
7.9	Existing – Tariki Weir in place	210.90	7.3	210.98	5.7

Note: Manganui Weir Crest RL 210.88m

If the weir is permanently removed (or modified), flow gauging will need to be reinstated at an alternative location. Ideally, any new gauging site would be established prior to removal of the existing weir to ensure monitoring continuity.

If the hydraulic characteristics of the Race above the Tariki Weir tends toward the rougher end of the likely range (i.e., n = 0.025), then additional improvement works may be required to increase abstraction capacity beyond approximately 5.9m^3/s. Such improvements could include lining the Race with smoother material such as concrete, enhancing vegetation and sediment maintenance, and localised trimming of the sides to reduce roughness.

5.1.2 Downstream of the Tariki Weir

While the Race below the Tariki Weir can be hydraulically separated into several sections, in terms of potential for hydraulic improvement, the same options largely apply to all sections:

- Reduce roughness by trimming to a smoother profile or lining with smoother material.
- Modify hydraulic structures (e.g., tunnels, bridges, culverts, drop structures) to provide more efficient flow characteristics.
- Locally raise the sides of the Race to provide greater flow depth.
- Locally widen the Race to provide greater flow width.
- Combination of one or more of the above.

6.0 Assessment of Performance

Having established that up to 7.5m^3/s could be effectively conveyed into the Race system, the remaining focus of the assessment is to determine the potential for such flow to induce adverse effects.

The following section presents the results from the assessment against a range of criteria. The performance criteria described in Section 4.2 are applied to determine if the potential for adverse effects might arise. Where a potential for adverse effects has been identified, options for managing these effects are provided in Section 7.0.

6.1 Race Freeboard Performance

For comparison purposes two flow scenarios are presented: 5.2m^3/s, equivalent to the maximum abstraction provided in the existing resource consents; and 7.5m^3/s, the maximum feasible abstraction into the Race without requiring major hydraulic improvements as discussed in Section 5.0. It is noted that the modelling and corresponding results provided below presume removal of the Tariki Weir, which reduces the water level upstream of the weir as discussed in Section 5.1.1.

Table 7 and 8 provide a summary of modelled water levels covering the range of anticipated roughness coefficients and corresponding losses (n = 0.015 & n = 0.025). The water level results are also presented in the appended RILEY drawings. Also shown for reference is the current operational water level limits defined in the existing resource consents.

Table 7: Water Level Modelling Results (Lower Race Losses, n = 0.015)

Location	Existing Maximum Race Control Levels (m RL)	Modelled WL @ 5.2m^3/s (m RL)	Modelled WL @ 7.5m^3/s (m RL)	Difference (m)
Manganui Weir	-	210.49	210.84	0.35
Intake Structure	-	210.48	210.84	0.36
Sediment Pond	-	210.42	210.75	0.33
Tariki Weir	-	210.38	210.71	0.33
Salisbury Road Bridge	205.20	204.70	204.93	0.23
Mangaotea Road Culvert	199.30	199.06	199.26	0.20
Mangaotea Aqueduct	199.25	199.02	199.20	0.18
Lower Mangaotea	199.15	198.88	199.05	0.17
Upper Lake Ratapiko	-	198.73	198.77	0.04

Table 8: Water Level Modelling Results (Higher Race Losses, n = 0.025)

Location	Existing Maximum Race Control Levels (m RL)	Modelled WL @ 5.2m^3/s (m RL)	Modelled WL @ 7.5m^3/s (m RL)	Difference (m)
Salisbury Road Bridge	205.20	204.91	205.36	0.45
Mangaotea Road Culvert	199.30	199.27	199.57	0.30
Mangaotea Aqueduct	199.25	199.22	199.49	0.27
Lower Mangaotea	199.15	199.01	199.30	0.29
Upper Lake Ratapiko	-	198.73	198.77	0.04

Note: Results are not presented upstream of Salisbury Road Bridge as 7.5 m3/s abstraction is not possible with higher race losses in the upper race, Refer Table 6.

The model predicts that the Race water levels will increase by less than 0.45m from the existing maximum flow take of 5.2m^3/s, to the proposed maximum flow take of 7.5m^3/s.

For abstracted flow up to the existing consented flow of 5.2m^3/s, the adopted performance criteria for freeboard is fully met. For abstracted flow above 5.2m^3/s, there reaches a point where performance criterion for freeboard is not met if no improvements are made to hydraulic performance.

Table 9 presents locations with potentially less than 300mm freeboard, with the proposed maximum 7.5m^3/s flow take. These areas are located within the influence of downstream tunnels or culverts that tend to slow the water flow velocity and consequentially cause the level to increase. Modelling and/or site observations indicate that although these areas may have inadequate freeboard (less than 300mm), overtopping is not predicted to occur with the proposed maximum 7.5m^3/s flow take.

There is also the potential for the higher 7.5m^3/s abstraction to exceed the existing Maximum Race Control water level limit at the Mangaotea Road Culvert site.

Table 9: Locations with Potentially Inadequate Freeboard

Chainage (m)	Comment
1800 – 1850	Within reach upstream of Tunnel 1. June 2020 inspection indicated that existing freeboard is in the order of 400mm at a low point on the true left bank, over a length of approximately 50m (refer to Photo 4). This low point is also evident from the LiDAR. The paddock beyond the bank is lower than the bank and therefore, consideration will need to be given to farm drainage if the Race bank is raised at this location.
2570 – 2740	Within reach upstream of Mangaotea Road culvert. June 2020 inspection indicated that race bank stability works have been undertaken in the time since the LiDAR (used in the hydraulic modelling) was undertaken. These stability works have modified some of the ground levels adjacent to the Race and improved freeboard to typically 900mm (refer Photo 5). Some further filling behind the new timber wall may be required to improve security of freeboard.
3040 – 3400	Within reach upstream of Tunnel 2 on true right side. June 2020 inspection confirmed that the true right side appeared to have a freeboard of as low as 600mm to the top of the bank at the time of the inspection (refer to Photo 6). We note that the lake level was approximately RL 198.4m at the time (300mm below maximum normal operating level), and that the proposed 7.5m^3/s flow take will increase water levels in the order of 300mm. Ground levels do rise slightly further into the paddock, so any overtopping issues are likely to be localised to the edges of the Race/paddock.

Photo 4: View downstream from CH 1830m (June 2020). The freeboard at the time of the inspection was estimated to be 400mm on the true left bank. The paddock to the left of the farm track is lower than the track. The low point in the track is evident in the lower to centre part of the photo. The vehicles are parked at the high point on the track.

Photo 5: 900mm freeboard at CH 2660m on true left bank (June 2020). Note that the timber wall appears to extend above ground levels behind. Timber board are 150mm deep and the concrete blocks in the order of 600mm high.

Photo 6: View downstream from CH 3400m (June 2020). The Race freeboard at the time of the inspection was estimated to be as low as 600mm on the true right bank. The ground levels in the paddocks beyond the right bank rise slowly. The true left bank (farm track) is significantly higher.

Apart from the localised areas provided in Table 8, the model results and the June 2020 inspection indicate that the Race will have a minimum freeboard of 300mm along its remaining length with an abstraction flow of $7.5m^3/s$.

6.2 Bridges Freeboard Performance

The modelled freeboard at the bridges is presented within Table 10 for the 7.5m^3/s maximum abstraction scenario. The minimum freeboard is generally at or better than 500mm, which exceeds the adopted performance criteria.

The one exception is at the Lower Mangaotea Bridge, where the modelled freeboard is approximately 290mm for the lower race losses scenario (refer to Photo 7). This is slightly less than the adopted performance criteria of not less than 300mm but is well within modelling tolerances. We note that, during the June 2020 inspection, a freeboard of 700mm was measured with the lake level at RL 198.4m. These site observations align with the increased flow model results and a lake level at RL 198.7m.

If race losses were closer to the high end (n = 0.025) the results indicate some work is likely to be required in the vicinity of the Lower Mangaotea Bridge if a flow of 7.5m^3/s is to be achieved.

Photo 7: Lower Mangaotea Bridge with freeboard of 700mm (June 2020, lake level RL 198.4m).

Table 10: Bridge Freeboards (Tariki Weir Removed)

Name	Estimated Underside of Bridge Level (m RL)	Modelled WL at 7.5m^3/s (m RL)		Freeboard (m)	
		Lower Race Losses	Higher Race Losses	Lower Race Losses	Higher Race Losses
Upper Silt Pond Bridge	211.4	210.75	n/a	0.65	n/a
Lower Silt Pond Bridge	213.2	210.73	n/a	2.47	n/a
Tariki Road Bridge	210.6	210.14	210.48	0.46	n/a
Salisbury Road Bridge	206.3	204.94	205.35	1.36	0.95
In-Race Generator Bridge	205.2	204.84	204.85	0.36	0.35
Upper Mangaotea Bridge	201.1	199.26	199.57	1.84	1.53
Lower Mangaotea Bridge	199.37[1.]	199.08	199.35	0.29	0.02
Thompsons Bridge	201.3	198.84	198.90	2.46	2.40

[1]Surveyed 16 April 2019

6.3 Culvert Freeboard Performance

The model results presented within Table 11 indicate that, for the low losses scenario, the Mangaotea Road culvert will come close to being surcharged with a maximum abstraction flow of $7.5m^3/s$ (and a lake level of RL 198.7m). The other two culverts along the Race meet performance criteria. During the June 2020 inspection (refer Photo 8), we observed a freeboard of 350mm to the culvert obvert[2] (lake level RL 198.4m). At full supply lake level of RL 198.7m (i.e., 300mm higher), site observations confirm that the culvert is likely to become surcharged with any increase in flow above the existing maximum of $5.2m^3/s$.

Photo 8: Mangaotea Road Culvert entrance (June 2020).

For the higher race loss scenario, the Mangaotea Road Culvert can expect to become surcharged if no works are undertaken to improve hydraulic performance.

Table 11: Culvert Summary

Culvert	Modelled WL at $7.5m^3/s$ (m RL)		Culvert Obvert Level (m RL)	Freeboard to Culvert Obvert (m RL)	
	Lower Race Losses	Higher Race Losses		Lower Race Losses	Higher Race Losses
Tariki Road culvert	210.71	n/a	211.5	0.8	n/a
Salisbury Road culvert	207.68	207.79	207.8	0.1	0.0
Mangaotea Road culvert	199.26	199.57	199.3	0.0	-0.3

[2] Obvert, the interior top (or highest) elevation of pipe.

6.4 Erosion Performance Assessment

Progressive erosion and slumping of the cut slopes adjacent to the Race has been ongoing, necessitating repairs and upgrades. We understand from Trustpower that erosion and slumping is progressive over time and flood events do not necessarily cause increased erosion.

From observation, the Race generally appears to be located within cut consisting of clays and organic clays (noting that a physical geotechnical investigation has not been undertaken).

In general, the longitudinal gradient of the Race is steeper upstream of the In-Race Generator than downstream and therefore, higher velocities occur upstream of the In-Race Generator.

To determine expected performance in terms of erosion and slumping, we have separated the Race into eight sections, with four sections (1 to 4) upstream of the In-Race Generator, and four sections (5 to 8) downstream of the In-Race Generator. These sections are detailed in Appendix I and summarised below. Observations presented are based on the June 2020 inspection.

Velocity maps included within Appendix J present the modelled velocities for the two abstraction flows (for the lower race losses). The model velocity results generally match observations made on site during the June 2020 inspection. No direct flow velocity measurements were made on site; however, velocity estimates were made using a stopwatch and observing floating debris within the main current, over an estimated short distance.

Table 12 summarises the typical velocities observed in each of the sections, and how this might be expected to increase with a greater flow abstraction. We have only presented results for the low loss scenarios as these produce the most conservative velocities. Any significant discrepancies are highlighted in the following sections.

Table 12: Typical Velocities (Low Losses, n = 0.015)

Section	Predominant Lining	Typical Velocity at 5.2 m^3/s (m/s)	Typical Velocity at 7.2 m^3/s (m/s)	Typical Velocity Increase (m/s)
1	Concrete	1.2	1.5	0.3
2	Concrete	>1.4	>1.4	0.3
3	Unlined	1.6	2.0	0.3
4	Unlined	<1.0	<1.4	0.3
5	Unlined	1.8	2.1	0.3
6	Unlined	<1.0	<1.4	0.3
7	Unlined	1.5	1.8	0.3
8	Unlined	<1.0	<1.0	0.3

Note: The Section Numbers shown relate to the following report sections

6.4.1 Section 1 – Manganui River to Tariki Weir, CH 0m to 725m

This section is predominantly concrete-lined with no observed significant existing erosion. Modelled flow velocity is typically 1.2m/s, increasing to 1.5m/s at a maximum abstraction of 7.5m^3/s.

While the modelled flow velocity of 1.5m/s exceeds the desired 1.4m/s performance criterion, significant erosion or prevention/remedial works is not anticipated given the added resistance provided by the concrete lining. Water levels may increase above the existing concrete lining in places at an abstraction of 7.5m^3/s and therefore, some erosion may occur above the liner in these areas. Refer to Section 7.3 for further discussion on the proposed erosion management plan.

6.4.2 Section 2 – Tariki Weir to End of Concrete Lining (CH 725m to 1075m)

Section 2 is approximately 350m long and appears to be concrete lined along its entire length, with no observed significant existing erosion issues.

Modelled velocities in this section typically range from 1.5m/s to 3.5m/s at a flow of 5.2m^3/s, increasing to 1.6m/s to 3.7m/s at 7.5m^3/s. Due to the concrete lining, it is not anticipated that any significant increase in the risk of erosion will occur needing prevention/remedial works. We note that we have not undertaken a specific inspection of the concrete lining.

As with the previous section, water levels may increase above the concrete lined sections of the Race and therefore some erosion in these areas may occur.

6.4.3 Section 3 – CH 1075m to 1600m

Section 3 is approximately 525m long and extends from the end of the concrete lining at CH 1075m, to the end of the steep section at CH 1600m. This section has a relatively steep longitudinal grade (including a drop structure) and has variable geometry, lining and flow conditions/velocities, with some observed approximate velocities of 2.5m/s.

Modelled velocities in this section typically range from 0.4m/s to 1.6m/s at a flow of 5.2m^3/s, increasing to 0.5m/s to 2.0m/s at 7.5m^3/s. We note that there is a 200m stretch between CH 900m and CH 1100m, where modelled velocities are typically greater than 2.0m/s in both scenarios (and a similar increase of approximately 0.3m/s is observed between the two).

No significant slumping or erosion was identified; however, some erosion remedial works have been undertaken in the past. There appears to be a moderate risk of further erosion and slumping that could be exacerbated by an increase in maximum flow and associated flow velocity.

6.4.4 Section 4 – CH 1600m to 2225m (In-Race Generator)

Section 4 is approximately 625m long and extends from the end of the steep section at CH 1600 to the In-Race Generator. Velocities in this section achieve the performance criterion of less than 1.4m/s at all locations up to the maximum flow modelled (7.5m^3/s). Some localised repairs undertaken in the past are evident. Erosion risk in this section is considered to be low, with the need for repairs and maintenance likely to be limited to localised areas.

6.4.5 Section 5 – In-Race Generator to Upper Mangaotea Bridge (CH 2225m to 2550m)

Section 5 is approximately 325m long and had no observed significant existing erosion issues. However, the uneven appearance of the Race banks in the reach downstream of the In-Race Generator, indicates that some historical erosion may have occurred through this reach. Some mass concrete block remedial works have also been undertaken in the past further downstream.

During the June 2020 inspection, we observed velocities of up to 2.0m/s immediately upstream of the bridge. Model results indicate velocities at this location could reach 2.1m/s for a maximum abstraction flow of 7.5m^3/s. Overall, there appears to be a moderate risk of further erosion and slumping requiring periodic remedial repairs.

6.4.6 Section 6 – Upper Mangaotea Bridge to Mangaotea Aqueduct (CH 2550m to 2850m)

Section 6 is approximately 300m long and extends from the Upper Mangaotea Bridge to the Mangaotea Aqueduct. The section is located within the broad Mangaotea floodplain and is cut within soft, highly organic material prone to slumping due to frequent variations in water levels (i.e., independent of velocities). Water levels can also be affected by lake levels within this section. Observed flow velocities are less than 1.0m/s. Site personnel confirmed that within this section slumping does not appear to occur due to the low water velocity.

Tree stumps are visible when the Race is dewatered. Site personnel have observed that in places, these tree stumps cause eddies to occur which can also lead to erosion.

Through this section, extensive erosion remedial works have been undertaken over recent years. The repairs consist of mass concrete blocks and a timber retaining wall section. We understand the retaining wall was constructed instead of the mass concrete blocks due to settlement issues relating to the low strength of the organic founding material. To date, the remedial works appear to be performing adequately.

The model results indicate that velocities will typically remain less than 1.0m/s at a maximum abstracted flow of 7.5m^3/s. Irrespective of maximum flow, we consider that there is a risk of ongoing slumping due to low ground strengths, which will need periodic remediation along the lines of past repairs.

6.4.7 Section 7 – Mangaotea Aqueduct (CH 2850m to 3000m)

This section was not inspected during the June 2020 inspection; however, we understand from Trustpower Site Personnel that existing erosion remediation works consisting of mass concrete blocks are performing adequately.

The model results indicate that typical velocities in the section may increase from 1.5m/s to 1.8m/s with an increased abstraction flow from 5.2m^3/s to 7.5m^3/s. However, based on the February 2019 site inspection and comments from site personnel, we consider that there is a low risk of further erosion protection works being required.

6.4.8 Section 8 – Lower Race CH 3000m to Lake Ratapiko

This section is over 1600m long and extends along the remainder of the Race to the lake. Velocities were observed to be low (less than 1.0m/s), and it has been confirmed by site personnel that velocities remain low through to the lake. The model results also indicate that velocities remain less than 1.0m/s up to a maximum abstracted flow of 7.5m^3/s. The Race banks appear to be well grassed, with no evidence of recent erosion observed.

We consider that the risk of erosion related issues due to the proposed increase in flows is low to moderate within this section. The risk is mainly associated with the increase in water levels causing localised slumping.

6.5 Flood Conditions

The Race has the potential to reduce or increase flooding adjacent to it, depending on the amount of abstracted flow during localised flood events. We have approached the potential for flooding by firstly modelling no abstraction (intake gates closed) allowing the Race to operate as a drain. Only if this scenario indicates no flooding, would abstraction then be considered.

We have modelled the mean annual flood and the 100-year flood over a range of durations from 1-hour through to 12-hours. Further detail is provided in Appendix H.

We note that the results are potentially sensitive to the soil infiltration rate within the catchment at the time of the rainfall event.

6.5.1 Intake Gates Closed

The first scenario assessed is with the Manganui River intake gates closed. This means flow is only derived from the local catchment. Results of this scenario are provided in Table 13 at selected locations along the Race. The two largest tributaries to the Race are at CH 1300 (immediately upstream of the Salisbury Road Culvert) and at CH 3400 (upstream of Tunnel 2).

For this model scenario, the following parameters have been adopted:

- An initial race flow of 5.2m^3/s along its full length.
- An initial lake level of RL 197.9m (based on a review of typical lake levels i.e., average lake level 2001 – 2018 and average lake level for 2017 calendar year).
- The race intake gates are closed immediately in response to the local flood inflows.
- The powerhouse remains operational throughout the event with a constant flow of 7.0m^3/s.
- Higher race losses (race n =0.025 and higher losses at hydraulic structures).

Only the results for the higher race losses are presented as this scenario provides the higher flood levels.

Table 13: Peak Results with Intake Gates Closed (Higher Race Losses)

Location	Mean Annual Flood			100-Year Flood		
	Flow (m^3/s)	Level (m RL)	Critical Duration (hr)	Flow (m^3/s)	Level (m RL)	Critical Duration (hr)
Salisbury Road Bridge	6.2	205.10	3	14.7	207.19	3
Mangaotea Road	6.0	199.67	3	n/a[1.]	200.46	>12
Mangaotea Aqueduct	6.0	199.56	3	n/a[1.]	200.46	>12
Lower Mangaotea	9.9	199.47	3	11.1	200.46	>12
Lake Ratapiko (flood spillway)	n/a	198.85	12	n/a	199.52	6

[1]Not applicable as spill flow is occurring into and out of the race at this location.

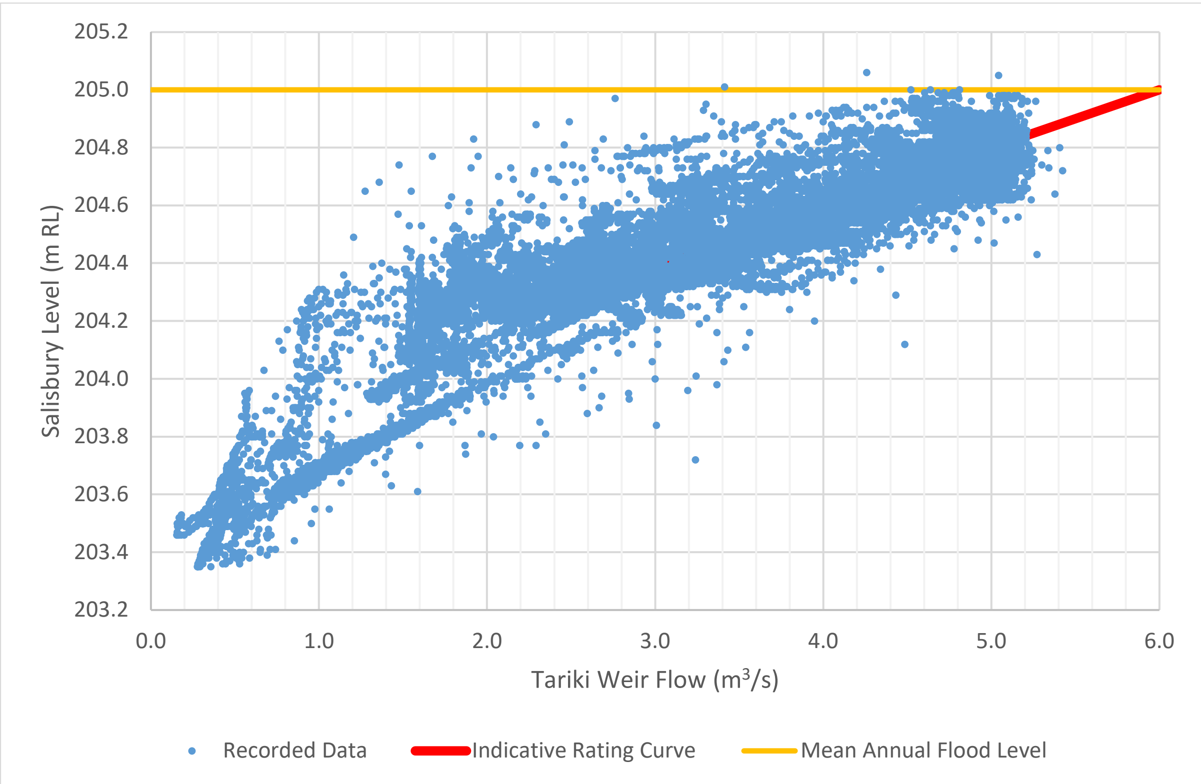
Table 14 provides a comparison between the recorded and modelled mean annual flood levels. The recorded levels were determined by averaging the maximum recorded water level for each calendar year from 2003 to 2018, inclusive. Years where there was inadequate data were not included within the analysis. We also excluded data from 2003 and 2004 for the Salisbury gauge (i.e., prior to the construction of the In-Race Generator). The table indicates that the calibrated model provides appropriately conservative results (i.e., model results typically 100mm to 200mm above recorded levels).

Table 14: Mean Annual Flood Level Comparison

Location	Recorded Mean Annual Flood Level (m RL)	Modelled Mean Annual Flood Level (m RL)	Difference in Mean Annual Flood Level (m)
Salisbury Road Bridge	205.00	205.10	0.10
Mangaotea Road	199.50	199.67	0.17
Mangaotea Aqueduct	199.40	199.56	0.16
Lower Mangaotea	199.22	199.47	0.25
Lake Ratapiko (flood spillway)	198.84	198.85	0.01

The Salisbury Road Bridge gauge provides a useful comparison of mean annual flood levels and mean water levels. As presented within Figure 2, with recorded Tariki Weir flows of approximately 5.0m^3/s, the mean level at the Salisbury Road Bridge gauge is approximately RL 204.8m. The mean annual flood level at the Salisbury Road Bridge gauge is RL 205.00m. This comparison indicates that the mean annual flood at the gauge is approximately 6.0m^3/s (noting that some of this flow may be diverted flow from the Manganui intake). We note that the calibrated model indicates that the critical duration mean annual flood flow at the gauge is 6.3m^3/s. Therefore, we consider that the calibrated model provides appropriately conservative flood flows at the gauge.

Figure 2: Salisbury Road Bridge Gauge (2017 Calendar Year)



Mean Annual Flood

The results indicate that even with the intake gates closed, the peak flows within the Race during the mean annual flood are greater than the existing maximum flow take of 5.2m^3/s.

Consistent with existing operational practice, and in the absence of any hydraulic improvements to the Race we consider ceasing abstraction at the intake should continue to occur during events in excess of the mean annual flood within the local race catchment. As such, modelling including some abstracted flow (gates partially open) was not considered necessary.

Events smaller than a Mean Annual Flood can continue to be managed through operational modulation of the intake gates.

100-Year Flood

The larger 100-year flood scenario indicates significant flooding of land, including adjacent to the Race, with flow into and out of the Race, particularly within the Mangaotea Stream floodplain. Flood inundation maps for this event are included within Appendix J. We note that the Manganui River flows have not been modelled and therefore, the mapped results do not reflect the 100-year flood situation within the river. However, this does not impact the extent of flooding adjacent to the Race.

It is impractical in the model to account for minor features like the farm tracks and culverts within the catchment. These elements are likely to have minor and typically localised effects on flood levels.

Because the derived flood flows exceed the practical capacity of the Race over the majority of its length (i.e., well in excess of 7.5m^3/s), any flow abstracted from the intake would potentially exacerbate flood risk and extent. Closure of the intake would therefore be a necessity in such an event.

7.0 Management of Potential Effects

The following section describes recommendations for managing the potential risk of adverse effects arising from flows within the Race including any increase in maximum abstraction rate. The overall recommendation is to adopt a performance management approach, utilising observational verification against the performance criteria described in Section 4.0. This is consistent with current operations but also includes performance verification should the maximum abstracted flow be increased.

7.1 Observation Performance and Verification

The following describes a staged approach that is recommended to be implemented when increasing abstracted flow in the Race. The staged approach would be accompanied by observational verification of Race behaviours and characteristics for the purpose of monitoring the performance of the Race and identifying and/or managing potential adverse effects. Any additional recommendations associated with specific aspects of the Race are discussed in the subsequent sub-sections.

Because the scheme has operated for many years, and has an extended performance history, there can be confidence that performance can continue to be managed going forward. The main consideration is associated with a potential change in performance if Trustpower abstracted and conveyed a higher flow in the Race than the existing consented maximum of 5.2m^3/s.

While this assessment indicated that the risk and magnitude of potential effect is low, if an increasing abstracted flow were to occur there is the potential for increased risk of adverse effects that might need managing though physical or operational controls. It is appropriate therefore to adopt a staged approach to increasing the maximum abstraction flow, with performance verified before advancing to the next increase in the maximum abstraction flow. Once each increase in maximum flow is verified through performance monitoring it would form the new operational maximum subject to normal performance management criteria such as maintaining freeboard. This should take the approach of:

- Increasing maximum abstracted flow in increments of no more than $0.5m^3/s$ from $5.2m^3/s$ with performance verified before subsequent increases are permitted.
- The new increased maximum abstraction flow is maintained until the Race has operated at the new increased maximum for at least seven cumulative days unless adverse effects are observed. While the seven cumulative days (168 hours) does not need to be consecutive, it should include at least one period of six consecutive hours.
- The new increased maximum abstraction flow should coincide for a period of several hours with a high lake level (within upper 0.5m) to verify performance with the influence of high lake level.
- Performance at the new increased maximum is verified against the performance criteria discussed in Section 4.0.
- If performance is not verified, the maximum abstraction flow is reduced to the previous maximum to allow time for the consequence to be determined and any mitigation/remedial measures for the Race defined.

The above should be encapsulated in a Flood and Race Flow Management Plan (FRFMP) prepared before any increase in maximum abstracted flow is advanced. This plan would also provide operational procedures discussed in some of the following sections.

7.2 Managing Freeboard

This assessment has determined that freeboard is sufficient for the existing maximum abstracted flow of $5.2m^3/s$. For abstracted flow approaching $7.5m^3/s$, there may be a need to undertake some work to minimise the risk of localised adverse effects arising from insufficient freeboard. The freeboard could be improved through a combination of localised hydraulic improvement in the Race (e.g., smooth lining or profile improvements) or minor raising of the Race banks.

Irrespective of whether any physical improvement works are undertaken, the staged approach with observational performance verification, described in Section 7.1, is appropriate for verifying freeboard performance under any increase in maximum abstraction flow above $5.2m^3/s$.

If needed, physical works to increase freeboard or improve hydraulic performance could be undertaken to enable raising the flow to the next step. A maximum abstracted flow capacity more than $7.5m^3/s$ is not recommended unless significant additional analysis and physical works are undertaken to offset race capacity constraints.

The same staged approach with observational performance verification is equally appropriate to verify bridge freeboard performance. If freeboard is deemed to be insufficient, there are options available to mitigate any adverse effects including:

- The bridge could be raised to increase freeboard. Any bridge raising should also consider the 100-year flood levels.

- Hydraulic improvement within the Race would reduce water levels and hence improve freeboard under the bridge.
- Maximum Race flow could be trimmed back when the level in Lake Ratapiko is high, inducing greater backwater effect in the lower reaches of the Race.

For culverts and the potential for surcharge, the same staged approach with observational performance verification is also appropriate to verify bridge freeboard performance. In addition, discussions with New Plymouth District Council (NPDC) should be held to discuss the implications of the increased water levels for the Mangaotea Road culvert. As for bridges potential management options, to limit adverse effects on culverts would include:

- Hydraulic improvement within the Race and approaches to the culverts to reduce water levels and improve hydraulic performance.
- Maximum Race flow could be trimmed back if surcharge levels were deemed unacceptable.

7.3 Slumping and Erosion Management

For the existing maximum abstraction flow of 5.2m^3/s, the assessment has determined that there is a low to modest likelihood of ongoing erosion and slumping in the Race and that any such erosion and slumping is likely to be localised. As such, the risk of erosion and slumping is low which is consistent with historical performance.

This risk is increased in some areas with a higher 7.5m^3/s maximum abstraction flow. This increase in risk is due to an increased likelihood that erosion or slumping will occur rather than the nature or scale of these effects. The nature and extent of erosion and slumping is not expected to change.

Trustpower has previously used an observational approach to slumping and erosion related issues within the Race, under the existing flow take conditions. Remedial works that have taken place appear to have been effective in managing erosion and slumping and provide an example of what might be required in the future. As these past remediation measures are likely to have addressed the most vulnerable areas of the Race (on an as needed basis), irrespective of increased flow, it is unlikely that erosion or slumping risk will be measurably more than has occurred in the past.

We anticipate that some increase in localised and minor erosion related issues may occur if flows are increased beyond the historical maximum of 5.2m^3/s. We consider that the staged and observational approach is equally applicable to manage potential erosion issues.

We recommend that an erosion management plan is developed for the Race. This would largely be based on the staged and observational performance verification approach described in Section 7.1. Such a document could include details such as specific monitoring locations, the staged process for increasing flow takes over time, and potential repair options that may be required.

If erosion related issues become evident over time~~,~~ remedial options could involve erosion protection (such as mass concrete blocks as successfully used in the past), or localised amendments to the Race cross section or longitudinal profile.

An erosion management plan would as a minimum provide detail around:

- The frequency of monitoring (to include monitoring following any significant events).
- Classification of the extent and severity of any erosion of slumping features.

- Any immediate response requirements if needed (e.g., reducing back to the previous maximum abstraction flow where performance was previously verified).
- Guidance and performance criteria for potential remedial options.

7.4 Geotechnical Stability Considerations

We note that the only significant fill embankment located within the Race was constructed as part of the In-Race Generator project. We have not observed any visual instability issues with the embankment during Intermediate Dam Safety Review[3] (IDSR). Four toe drains are located at the toe of the embankment and are monitored on a monthly basis. The drains demonstrate seasonal but stable trends.

We note that a piping failure has occurred previously within the Race approximately 10 years ago. We understand that excavation of a farm drain adjacent to the Race occurred immediately prior to the breach. We do not consider that the increased risk of a piping type failure through the Race banks is significant, and therefore we do not consider that any piping assessments are necessary.

However, we suggest that a detailed review of tunnel condition is undertaken, along with a geotechnical review of the Race (to include an assessment of the In-Race Generator embankment). This would provide baseline information for use in implementing any erosion or slumping repairs, and design of any hydraulic performance improvements. The detailed review should be undertaken prior to the first increase in maximum abstracted flow as discussed under Section 7.1.

7.5 Operation During Normal Flow

The flow abstracted into the Race is managed through the operation of the intake gates (refer Section 3.1). The gates can be closed to prevent any flow take, or they can be opened to varying vertical positions to manage the flow take with varying river flows. The operation of the gates also manages the water level behind the Manganui River weir to ensure compliance with compensatory flow and fish passage operation requirements.

The intake gates will be operated to maintain the maximum abstraction flow take at the Manganui River to less than the maximum allowed within the resource consents, including any interim maximum during a staged and observational performance approach. The operational control logic for the gates is equally applicable to any maximum abstraction flow limit.

The historical operation discussed in Section 3.0 shows that the gates have been effectively managed such that abstracted flow is typically less than $4.8m^3/s$ ($0.4m^3/s$ below the maximum allowed in the resource consents). This is because compliance is currently based on instantaneous flow, which is particularly challenging to manage in river systems like the Manganui River where flow can change very quickly.

This assessment has shown that the Race has capacity beyond $5.2m^3/s$ and as high as $7.5m^3/s$, where the potential risk of adverse effects can be effectively managed. The $0.4m^3/s$ buffer, provided in the historical control system, therefore represents a conservatism that has been adopted by Trustpower to ensure compliance with the consent limit rather than a measure to avoid potential adverse effects.

[3] IDSRs are undertaken annually for the Motukawa Scheme.

As shown in Table 15, there is potential that at a maximum abstraction flow of 7.5m^3/s, there is a risk that the Maximum Control Water Levels specified in the existing resource consents could be exceeded at all locations if Race losses were toward the conservative upper end (n = 0.025). Table 15 compares the modelled water level results with the existing maximum consented levels. The recommended staged approach utilising observational performance verification outlined in Section 7.1 is considered appropriate to verify performance against any specified water level controls.

Table 15: Consented Water Levels

Location	Modelled WL at 7.5m^3/s (m RL)		Existing Maximum Consented Level (m RL)	Difference (m)	
	Lower Race Losses	Higher Race Losses		Lower Race Losses	Higher Race Losses
Salisbury Road Bridge	204.93	205.36	205.20	0.27	-0.16
Mangaotea Road Culvert	199.26	199.57	199.30	0.04	-0.27
Mangaotea Aqueduct	199.20	199.49	199.25	0.05	-0.24
Lower Mangaotea	199.05	199.30	199.15	0.10	-0.15

Irrespective of what maximum abstracted flow is ultimately allowed for in the resource consents, it is recommended that compliance associated with flow abstraction be based on two performance criteria rather than a single instantaneous flow limit, subject to any compensation flow requirements being met. The performance criteria are:

- Instantaneous abstracted flow does not exceed the Maximum Consented Flow plus 0.5m^3/s; and
- The average abstracted flow over any rolling three hour period does not exceed the Maximum Consented Flow.

The first provides flexibility to manage the intake during periods of rapidly changing river flow, whilst the second ensures that the abstracted flow does not induce increased risk of potential adverse effects associated with the operation of the Race. These criteria would be included in the FRFMP.

7.6 Operation During Floods

In addition to operational management under normal flow conditions (Section 7.5), operation also needs to effectively manage inflows from the local race catchment. The recommended staged approach detailed in Section 7.1 is only partially relevant to management during flood events because of the infrequent nature of floods.

The intake gates should continue to be operated to minimise the potential for any flooding effects on neighbouring properties. This is consistent with historical operation where intake gate operation is linked to the measured race water levels. This mechanism has been effective in managing the risk of flooding to adjacent land but is relatively coarse as the gates simply shut if any of the Race water level limits are exceeded. This assessment indicates that the current Control Water Levels specified in the resource consents may be conservative in terms of risk of flooding on adjacent land.

We recommend a gate operation procedure is developed within the FRFMP that refines the current mechanism but avoids the need to simply shut the gates during flood events less than a Mean Annual Event. This assessment has shown that the current operational response of fully closing the intake gates is required to be retained for local flood events equivalent or larger than an annual event. This permits the Race to act solely as a drain, partially reducing flooding adjacent to the Race. For smaller local floods, full closure of the intake gates is not required, and the gates can be modulated to manage Race water levels.

Once the maximum abstracted flow is confirmed through the staged approach, the verified performance under normal operating conditions can be adopted into the hydraulic model to refine flood response operations. The current mechanism of simply shutting the gate can remain in operation until the maximum abstracted flow is increased above 5.2m^3/s. Prior to increasing the maximum abstracted flow above 5.2m^3/s, an updated procedure for intake gate control and race level management plan should be developed within the FRFMP.

As a minimum, this procedure within the FRFMP would:

- Confirm or modify the existing water level control locations and limits. This would include consideration of more than one control level per monitoring site, for example:
    - A lower trim water level that if exceeded the intake gates are closed in increments.
    - A higher flood limit water level where the intake gates are fully closed (i.e., as per the current situation).
- Identify any additional water control locations required.
- Specify any other operational response to water level limits being exceeded.

During extreme flood events, although the intake gates will be closed, the maximum consented water levels may still be exceeded due to local inflows entering the Race. This may induce localised flooding on adjacent land, however, this is not considered to be because of the existence or operation of the scheme.

8.0 Summary

We summarise the main points from this assessment as follows:

- The modelling undertaken to assess the performance of the Race under a range of flow scenarios has been purposely calibrated to be slightly conservative in terms of modelled water levels.
- Work undertaken for this assessment indicates that the Race has the capacity to operate at up to 7.5m^3/s under normal conditions with minimal need to modify the Race infrastructure. However, removal of the Tariki Weir (flow monitoring site) and localised hydraulic improvements would be required to achieve a maximum abstraction flow of 7.5m^3/s without spill occurring at the Manganui Weir.
- Under normal operating conditions, with an increase in maximum abstracted take from 5.2m^3/s to 7.5m^3/s, model results indicate water levels within the Race will increase by less than 0.3m.
- Any increase in abstraction above the historical maximum of 5.2m^3/s should be undertaken in a staged approach with associated performance verification. This same approach can be applied to verify performance against other potential effects.

- This assessment has determined that there are some localised areas where the risk of potential adverse effects could be increased due to an increase in maximum abstracted flow. These areas will need to be modified or monitored as part of the performance verification process detailed in the recommended management plans. These include:
    - Some adjustments to the historical maximum consented race water levels specified along the Race if it is shown that no adverse effects are induced in doing so.
    - In some locations the Race may have less than 300mm freeboard for a flow of 7.5m^3/s. This may require additional mitigation measures, which can be confirmed as part of the performance verification process such as minor increases to the Race embankment levels and hydraulic improvements in the Race to lower water levels.
    - The Lower Mangaotea Bridge may have insufficient freeboard. Again, this can be confirmed as part of the performance verification process.
    - The Mangaotea Road culvert will become surcharged with the increase in design flows.
- Ongoing slumping and erosion are evident within the Race. Effective remedial works have been undertaken by Trustpower using an observational approach to issues.
- We anticipate that some increase in the likelihood of erosion or slumping related issues may occur if the maximum abstracted flow is increased. It is considered that remedial measures similar to historical remediation measures that have taken place in the Race will be sufficient for managing potential erosion increases.
- We consider that it is appropriate to implement an observational approach to erosion and freeboard issues that may arise in association with a staged increase in flows over a number of months.
- There is benefit in undertaking a detailed review of tunnel condition along with a geotechnical assessment of the Race. Details from these would help inform response planning for any erosion of slumping repairs and any hydraulic modifications to improve flow performance.
- Significant inflows into the Race from the local catchment occur during flood events. Closure of the Manganui Intake is necessary in flood events at or above a Mean Annual Event. This leaves the Race to act more as a drain to reduce the impact of local flooding. During smaller flood events, modulation of the intake gates should continue, as is current practice, to manage Race water levels.

We recommend the following:

- The intake gates should be closed during significant rainfall events (as Trustpower currently does), to minimise downstream flooding effects. Full closure of the intake gates is required for local flood events equal to or exceeding an annual event. As an interim, the historic operation of fully closing the gates is considered conservative and should be continued. An operational procedure for incremental gate closure should be developed as part of a Flood and Race Flow Management Plan (FRFMP) before the maximum abstracted flow is increased.
- Any increase in maximum abstraction from the intake should be subject to a staged process with associated performance verification as outlined in Section 7.1. This process should be provided within the FRFMP.
- Discussions with NPDC should be held to discuss the implications of the increased water levels for the Mangaotea Road culvert.

- A visual observation monitoring plan should be developed for the Race. This would include an erosion management plan that is also based on an observational performance verification approach.

9.0 Limitation

This report has been prepared solely for the benefit of Trustpower Limited 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.

The hydrological and hydraulic analyses and recommendations contained in this report are based on our understanding and interpretation of the available information. The recommendations are therefore subject to the accuracy and completeness of the information available at the time of the study. Should any further information become available, the analyses and findings of this report should be reviewed accordingly.

APPENDIX A

Available Time Series Data

Available Time Series Data

Trustpower records water levels at numerous locations within the Race and lake. These are summarised within Table 1, in order from upstream to downstream. The existing consent has a maximum consented water level for some of the instruments as presented within Table 2.

Table 1: Water Level Instrument Summary

Name	Location	Previous Name	Identification
Manganui Weir	Upstream of weir	-	MTK_H14LT_WEIRL
Intake Structure	Downstream of structure	-	MTK_H14LT_010
Sediment Pond	Upstream of pond	-	MTK_US_ASL_POND
Tariki Weir	Upstream of weir	-	MTK_H14LT_011
Salisbury Road Bridge	Downstream of bridge	Coxhead Bridge	MTK_H14LT_002
Mangaotea Road Culvert	Upstream of culvert	-	MTK_H15LT_003
Mangaotea Aqueduct	On aqueduct	-	MTK_H15RAQUADUC
Lower Mangaotea	Upstream of Tunnel 2	Berrymans Bridge	MTK_H14LT_004
Penstock	Lake Ratapiko	-	MTK_H15LT_006
Flood Spillway	Lake Ratapiko	-	MTK_H13LT_007
Service Spillway	Lake Ratapiko	-	Harvest Unit

Table 2: Water Level Instrument Details

Name	Maximum Consented Level (m RL)	Data Period	Type
Manganui Weir	n/a	2005 - Current	Pressure Transducer
Intake Structure	n/a	2001 - Current	Ultra-sonic
Sediment Pond	n/a	2017 - Current	Ultra-sonic
Tariki Weir	n/a	2001 - Current	Pressure Transducer
Salisbury Road Bridge	205.20	2001 - Current	Ultra-sonic
Mangaotea Culvert	199.30	2001 - Current	Ultra-sonic
Mangaotea Aqueduct	199.25	2008 - Current	Pressure Transducer
Lower Mangaotea	199.15	2001 - Current	Ultra-sonic
Penstock	198.70	2001 – Current	Not sighted
Flood Spillway	198.70	2001 - Current	Pressure Transducer
Service Spillway	198.70	2013 - Current	Pressure Transducer

The Sediment Pond instrument is the most recently installed instrument. We understand the datums for all the instruments have been accurately surveyed.

The harvest unit at the service spillway only records raw water depths without a datum applied to the readings. For the purposes of this assessment, we have estimated that the harvest unit water level recorder has a datum of RL 197.80m.

Trustpower (as per conversations with Mr Bruce Walpole) has advised that the recorded water levels are likely to be recorded using daylight saving times. We have, therefore, adjusted any lake level data during day light saving periods by subtracting one hour from the recorded times.

Flow is/has been measured at two locations within the Race as presented within Table 3. The data provided from Trustpower for the In Race Generator ends in 2014, and therefore, we have not used this data within this assessment. Trustpower also records the flow discharged through both fish passes and the percentage opening of each gate at the intake structure. Trustpower records the penstock intake flow but does not record the estimated spillway discharge from the lake.

Table 3: Flow Instruments

Name	Location	Data
MTK_S1RACE_Q	Tariki Weir	2001 - Current
MTK_MT2_RGQ	In Race Generator	2007 - 2014
MTK_H16PEN_FLOW	Penstock	2001 - Current

We understand the main purpose of the Tariki Weir is to allow measurement of the Race flow (based on the measurement of the water level, MTK_H14LT_011).

We have identified four existing rainfall gauges which appear relevant to the catchment, as presented within Table 4. A historical rainfall gauge at Inglewood has a record extending from 1909 - 1992.

Table 4: Rainfall Instruments

Name	Source	Data	Identification
Inglewood @ Oxidation Ponds	Taranaki Regional Council	1999 - Current	941203
Boyd (Mangaotea Aqueduct)	Trustpower	2001 - Current	MTK_H15FQT_002A
Service Spillway	Trustpower	2013 - Current	Harvest Unit
Jet Boat Club (Lake Ratapiko)	Trustpower	2001 - Current	MTK_H17FQT_003A

Locations of key measurement stations are indicated in RILEY Dwg 18MTK-ENH-10.

APPENDIX B

Catchment Description

Catchment Description

The catchment is located within a broad plateau with generally flat to moderate slopes and is generally covered in pasture. The maximum elevation within the catchment is approximately RL 240m, with the reservoir at approximately RL 200m. There are no named tributaries flowing into the reservoir.

The catchment has a large storage capacity because it is relatively flat. Although there are no other large dams within the catchment, storage upstream of various culvert embankments is potentially significant (particularly at Tributary 1 and 2). The race capacity is also constrained at various culvert and tunnels (particularly Tunnel 2).

We have measured the catchment area utilising the LiDAR survey data (1m contours) within ARCGIS and CAD. A total catchment area of 21.38km^2 was derived. Sub catchments were also derived as summarised within Table 1 and 2 and are presented in RILEY Dwgs: 18MTK/ENH-1 to -3). We note that the Lower Mangaotea and Tunnel 3 catchments are flat, and flow channels may primarily be controlled by farm drains. We have assumed that the upper reaches of this flat area drain into the Lower Mangaotea catchment, through the network of farm drains.

The catchment drawings show that the lake is partly within the Upper Lake catchment and partly within the Lower Lake catchment, with a total area of approximately 0.28km^2. We have not specifically allowed for the water surface within the catchment model.

The lag times for each sub catchment were derived using four different methods. The Bransby-William's equation, US Soil Conservation equation, the Ramser-Kirpich equation and the Auckland Region TP108 equation. Lag times are discussed further within the calibration section. We note that the TP108 method resulted in the longest lag times (a CN of 39 was used to assess the upper limit of possible lag times).

Table 1: Catchment Summary

Sub Catchment	Area (km^2)	Shape	Steepness	Weighted Average Channel Slope (%)	Predominant Land Use
Tariki	0.12	n/a	Flat	0.1	Pasture
Salisbury	1.54	Wide	Moderate	1.5	Pasture
Mangaotea Road	0.26	n/a	Moderate	2.0	Pasture
Lower Mangaotea	1.40	Wide	Flat	0.5	Pasture
Tunnel 3	0.60	n/a	Flat	0.5	Pasture
Upper Lake	1.85	n/a	Moderate	1.0	Pasture
Lower Lake	1.95	n/a	Moderate	1.1	Pasture
Tributary 1	3.10	Wide	Flat	0.6	Pasture
Tributary 2	1.56	Wide	Moderate	1.1	Pasture
Mangaotea	9.00	Wide	Flat	0.3	Pasture
TOTAL	21.38				

Table 2: Catchment Lag Times

Sub Catchment	Area (km^2)	Longest Flow Path (km)	Weighted Average Channel Slope (%)	Bransby-Williams (minutes)	TP108 (minutes)
Tariki	0.12	0.8	0.1	20	53
Salisbury	1.54	1.3	1.5	24	52
Mangaotea Road	0.26	0.8	2.0	15	46
Lower Mangaotea	1.40	3.3	0.5	68	128
Tunnel 3	0.60	1.0	0.5	27	61
Upper Lake	1.85	1.4	1.0	25	62
Lower Lake	1.95	1.3	1.1	23	56
Tributary 1	3.10	4.0	0.6	76	142
Tributary 2	1.56	2.0	1.1	35	176
Mangaotea	9.00	6.0	0.3	122	242
TOTAL	21.38	-	-		-

The geology comprises generally low to moderate permeability overlaying brown ash.

The catchment area to the race intake has not been included within the assessment. During extreme flood events, we anticipate that some flow from the Manganui River will enter the race. We have assumed at this stage that these overflows do not enter the lake. Future assessments could consider this assumption further.

The race is prone to overtopping during extreme flood events (i.e., the race capacity is exceeded) and therefore during extreme flood events not all flood waters which enter the race reach the lake. The race tunnels potentially affect the race capacity (and therefore contribute to race overtopping). The race tunnels also potentially provide some attenuation.

APPENDIX C

HEC-HMS Model Description

HEC-HMS Model Description

1.0 General

A HEC-HMS model (v4.5) has been developed to model catchment responses to recorded rainfall for calibration events or to design rainfall. A subbasin has been used for each sub-catchment as detailed in the catchment description.

The resulting flow hydrographs were used as inputs to the HEC-RAS hydraulic model.

2.0 Precipitation Loss Method

HEC-HMS offers a number of methods to model the precipitation loss due to infiltration. As part of this assessment, we have selected the Initial and Constant method. This method uses the following variable input parameters:

- Initial Loss (mm)
- Constant Loss (mm/hr)

Initial and constant loss values vary depending on vegetation cover, ground type, and the state of the ground prior to the rainfall event. Typically, constant loss values range from 1 to 20, with lower constant loss values resulting in higher runoff flows throughout a rainfall event. We have used the same initial and constant loss values within each sub-catchment for each modelled event. We consider this is appropriate, given the relatively small catchment sizes and the similar geology within each sub-catchment.

The initial and constant loss values determine the excess rainfall.

3.0 Transform Method

The SCS Unit Hydrograph transform method has been used within the HEC-HMS model for all sub-catchments. Lag time and graph type (Peak Rate Factor (PRF)) are required input parameters for the SCS Unit Hydrograph transform method.

Lag times affect the timing of the peak direct flow and the peak direct flow magnitude. A higher lag time delays the time to the peak direct flow and reduces the peak direct flow.

Peak rate factors affect the peak direct flow magnitude and the distribution of the flow on either side of the peak (without significantly effecting the time of the peak direct flow). A higher PFR increases flow prior to the peak and increase the peak direct flow. PRF options within HMS vary from 100 to 600, with lower PRF generally appropriate for flatter catchments and higher PFR for steeper catchments.

4.0 Baseflow

Baseflow is significant within the catchment during extreme rainfall events.

The linear reservoir baseflow method has been used. The method results in direct flow (derived from the excess rainfall) and baseflow (derived from the rainfall losses). We note that the linear reservoir baseflow method does conserve mass (rainfall depth) within the catchment).

This method uses the following variable input parameters:

- Initial Discharge (m^3/s) or ($m^3/s/km^2$)
- Baseflow Fraction
- Baseflow Coefficient (hr)
- Steps

Baseflow fractions of 0.0 to 1.0 can be selected. A fraction of 1.0 results in the entire infiltrated/loss rainfall depth becoming baseflow. If the fraction is less than 1.0, the remaining fraction is lost and does not become baseflow (i.e., the runoff rainfall depth will be less than the rainfall depth).

Higher baseflow coefficients result in lower peak baseflows, as the baseflow volume is distributed over a longer period.

The attenuation can also be managed with the use of the steps value. A higher step value (possible range 1 – 99) provides more attenuation.

APPENDIX D

HEC-RAS Model Description

HEC-RAS Model Description

1.0 General

The following sections describe the development of the HEC-RAS hydraulic model. HEC-RAS enables the modelling of spill flow from the race, which can occur either due to high local inflows to the race or due to high lake levels (i.e., downstream controlled). A schematic of the HEC-RAS layout (RILEY Dwg: 18MTK/ENH-200) is included within Appendix I.

In summary, the model consists of:

- A 2D flow area for the race and the Mangaotea floodplain.
- Various structures within the race and within Mangaotea Stream.
- Storage areas for the Upper Lake, Lower Lake, Tributary 1 and Tributary 2.
- Culvert connections linking the Upper Lake, Tributary 1 and Tributary 2 to the Lower Lake.
- A 2D flow area covering the service and flood spillways and the initial reach downstream of the dam.
- Service and flood spillway connections linking the Lower Lake to the downstream reach.
- Inflow hydrographs at suitable locations.
- An outflow hydrograph to represent penstock outflow.
- Downstream boundary conditions (i.e., model outflows) downstream of the dam and the Mangaotea Stream culvert.

Further details are included within the following sections.

2.0 Terrain

A LiDAR survey was undertaken on 4 July 2018 by Landpro. At the time of the survey the lake level was approximately RL 198.3m. A photogrammetry survey of the lake area was also undertaken in April 2019 by BTW to take advantage of lower lake levels, and to gain further lake storage information. At the time of the second survey the lake level was approximately RL 195.0m.

The terrain was developed from a 0.1m DEM, based on the 2018 LiDAR information and from a 1.0m DEM based on the 2019 photogrammetry. The survey data does not include bathymetric information below water surface at the time of the survey. Therefore, a bathymetric surface was incorporated into the terrain along the length of the race.

A breakline was provided as part of the LIDAR survey information and additional survey, identifying the interface between the race/lake water edge and the ground above the water line. By visual inspection of the aerial photographs taken at the time of the LiDAR, the breakline generally follows the interface between the water surface and the ground. We have, therefore, used this breakline along the majority of the race length, to define the boundary of the bathymetric surface. One area of exception is upstream of the Tariki Weir, where the breakline appears to underestimate the race width.

Based on measurements taken during the site inspection and a survey undertaken by BTW Company Ltd (April 2019), we have conservatively assumed that the race upstream of Tariki Weir has:

- 2.725m base width.
- 20:1 (v:h) side slopes.
- A constant positive slope for the initial 90m (i.e., to upstream of the intake structure), with invert level falling from RL 208.15m to RL 207.24m. Over the next 15m (i.e., through the intake structure), a constant negative slope increases the invert to RL 208.68m. The invert remains constant at this level until the weir.

We also amended the terrain immediately upstream and downstream of the Tariki Weir, to ensure the race is wider than the weir width.

Downstream of Tariki Weir, race invert levels were obtained from a historical Works Consultancy Services drawing. The date of the survey is unknown. A copy of the historical drawing is included at the rear of this appendix.

3.0 Race

3.1 Race 2D Flow Area

The race has been modelled as a 2D flow area extending from Tariki Weir through to the Upper Lake.

The Manning's 'n' value represents the roughness of the terrain, which effects the water velocity and level for a given flow. Based on our experience, we would anticipate that the Manning's 'n' value would be in the range of 0.015 to 0.025, along the length of the race. For this assessment, we have assessed both a Manning's 'n' value of 0.015 and 0.025 for the race section, noting that the value is most likely to vary along the length of the race. In particular, the value is likely to be towards 0.015 within the concrete lined areas.

We have used a grid size of 1m for the race area.

3.2 Tunnels

The three race tunnels generally have a box-type shape, with an arched roof. Tunnel dimensions were not able to be measured during the site inspection due to high race water levels. However, the 16 April 2019 survey included tunnel invert levels and inlet dimensions. Within the model we have conservatively used smaller dimensions than that surveyed as the inlets are larger than the internal dimensions, as presented within Table 1.

Table 1: Tunnel Cross Section Dimensions

Name	Surveyed		Modelled	
	Height (m)	Width (m)	Height (m)	Width (m)
Tunnel 1 (Coxhead Tunnel)	2.750	3.150	2.0	2.5
Tunnel 2 (Lower Mangaotea)	3.208	1.739	3.0	1.5
Tunnel 3 (Toppings Tunnel)	3.633	2.750	3.5	2.5

The tunnels presented within Table 7 were included within the model as internal structures. The terrain was modified (to allow the inclusion of the structures) by forming a channel along the tunnel alignment through the terrain.

Tunnel 1 is located within the water level influence of the In Race Generator Weir, and both Tunnels 2, and 3, are located within the influence of Lake Ratapiko. The locations of these structures are shown on RILEY Dwg: 18MTK/ENH-200.

Table 2: Tunnel Summary

Name	Length[1.] (m)	Upstream Invert Level (m RL)	Downstream Invert Level (m RL)	Height (m)	Width (m)	Entrance Loss Coefficient	Manning's
Tunnel 1 (Coxhead Tunnel)	67	202.351	201.995	2.0	2.5	0.7	0.030
Tunnel 2 (Lower Mangaotea)	11	196.196	195.789	3.0	1.5	0.7	0.030
Tunnel 3 (Toppings Tunnel)	50	194.865	194.349	3.5	2.5	0.7	0.030

[1.] Length measured within ARCGIS from 2018 Aerial Photography

3.3 Upstream Structures

The Manganui Weir, Intake Structure, and the Tariki Weir have been included within the model as internal connections.

We note that there is some discrepancy between a recent Energy Surveys Limited survey (ESL 2017) and an earlier Trustpower drawing, as presented within Table 3. For the purposes of this assessment, we have assumed that the ESL 2017 levels are correct.

Table 3: Survey Comparison

Structure	Elevation (m RL)	Source
Manganui Weir crest	210.82	Trustpower drawing MTK-3G-0200
	210.88	ESL 2017
Tariki Weir crest	209.57	Trustpower drawing MTK-3G-0200
	209.59	ESL 2017

Input parameters for the structures are summarised within Tables 4, 5, and 6.

Table 4: Manganui Weir Parameters

Element	Value	Source/Comments
Crest (m RL)	210.88	ESL 2017
Width (m)	33.5	Historical drawing and checked via measurement on aerial photographs
Weir coefficient	1.7	Typical engineering value

Table 5: Intake Structure Parameters

Element	Value	Source/Comments
Available opening height (m)	2.0	Trustpower drawing.
Individual opening width (m)	1.3	Trustpower drawing show as 1.35m. Measured on-site (04/02/2019) as 1.38m. 1.3m used conservatively.
Number of openings/ gates	2	-
Upstream Invert (m RL)	207.22	BTW Company Ltd (16/4/19)
Downstream Invert (m RL)	208.7	Based on depth to Race invert measured (04/02/2019) at intake structure water level instrument (surveyed by ESL 2017). Also surveyed by BTW Company Ltd (16/4/19) at RL 208.66m.

Table 6: Tariki Weir Parameters

Element	Value	Source/Comments
Crest (m RL)	209.59	ESL 2017
Width (m)	3.7	Trustpower drawing and confirmed on-site (04/02/2019).
Weir coefficient	1.7	Typical engineering value

Trustpower records gate opening percentages, however, Trustpower was not able to provide vertical gate opening heights. We attempted to measure gate openings during the initial site inspection, however, due to the low river flows, and resource consent conditions, we were not able to successfully measure the gate openings. During the second site inspection in June 2020 some measurements were possible, however full gate openings were not possible again due to resource consent constraints. Measurements are presented within Table 7.

Table 7: Gate Opening Measurements

Case	Gate 1 (True Left)		Gate 2 (True Right)	
	Opening Percentage (%)	Opening Height (m)	Opening Percentage (%)	Opening Height (m)
Measurement	-1.3	0.00	0.0	0.00
Measurement	44.2	0.62	44.5	0.89
Assuming Linear Relationship	100	1.4	100	2.0

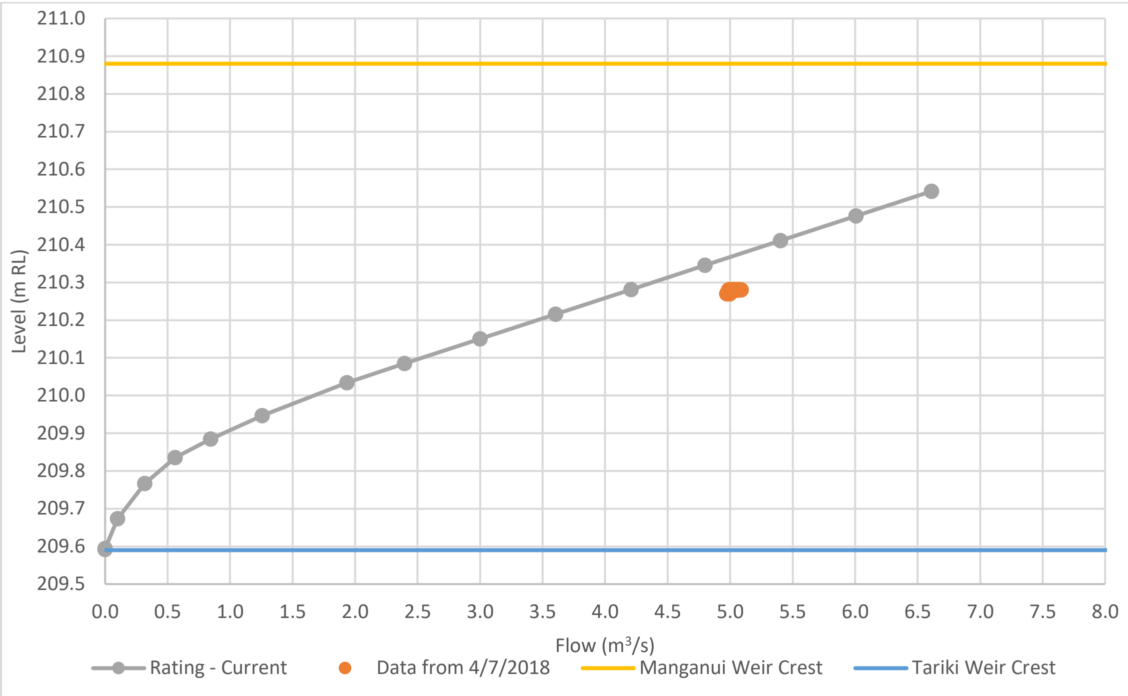
Assuming a linear relationship between opening percentage and opening height, it appears that Gate 2 opens to the full 2.0m available opening available, however Gate 1 may not. We also note that the soffit of the opening appears to be below the Manganui Weir crest. We recommend further on-site investigations are undertaken to ensure the geometry and the hydraulic performance of the structure and gate is fully understood.

The rating curve for Tariki Weir (provided by Trustpower during this assessment) is presented within Figure 1. We understand the rating has been derived from recent gauging's upstream of the Tariki Weir. The figure also includes recorded Citect data from a selected date of 4 July 2018. There is an apparent discrepancy between recorded Citect data, and the rating provided. We understand through discussions with Trustpower that the Citect rating is not available at this time.

This discrepancy is significant. At a water level of RL 210.28m Citect records a flow of approximately 5.0m^3/s, however, the rating provided, only suggest a flow of 4.2m^3/s at the same water level. For the purposes of this assessment, we have assumed that the Citect recorded flows are correct. We note that the water level instrument may be located within the drawdown zone of the Tariki Weir.

We recommend that this flow measurement discrepancy is further investigated during future assessments. In addition, there appears to be a need for an improved transition from site gauging's to the Citect rating.

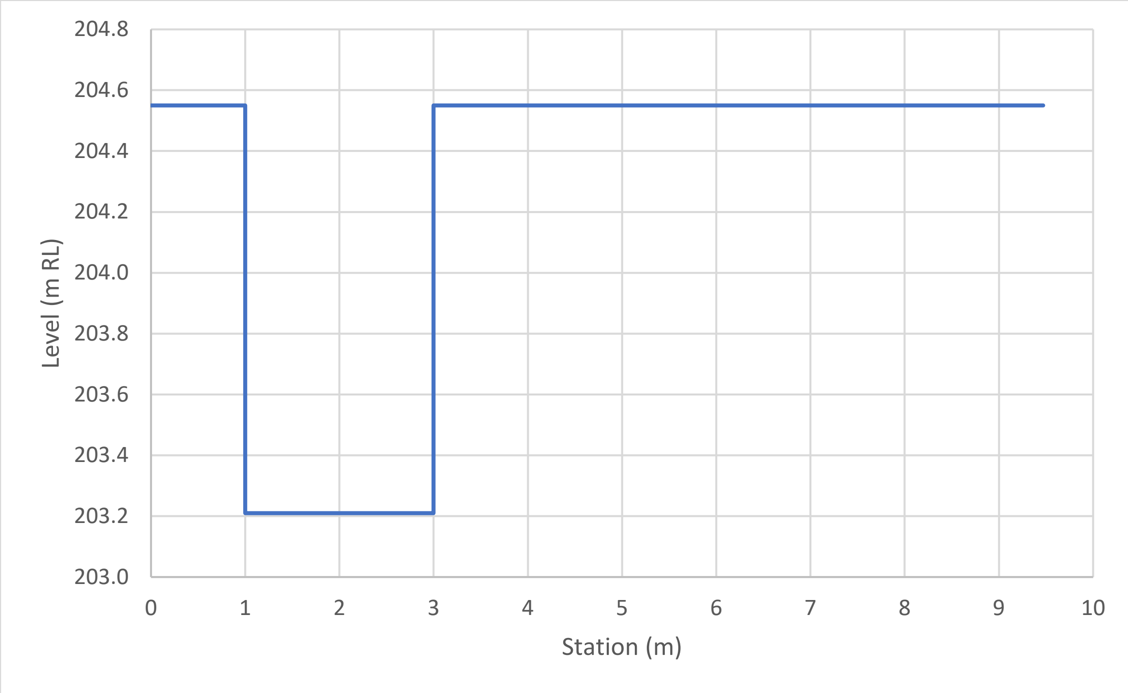
Figure 1: Tariki Weir Flow - Level Relationship



3.4 In Race Generator

As-built drawings for the In Race Generator, indicate that the weir crest is RL 204.55m (crest of the timber flashboards), with a 2.0m wide bypass channel. Figure 2 presents the weir used within the model.

Figure 2: In Race Generator Weir Section



We have not been able to confirm the design upstream water level at the In Race Generator, while in generation. We have therefore, assumed that the weir controls the water level at the In Race Generator. We recommend that the influence of the In Race Generator is further investigated during future assessments.

We note that the race water levels downstream of the In Race Generator are affected by Lake Ratapiko water levels.

3.5 Mangaotea Aqueduct

The Mangaotea Aqueduct crosses over the Mangaotea Stream. A vertical spillway gate is located on the true left side with a width of 5.45m (measured during the site inspection). We understand from Chris England (Trustpower) that the gate is generally not used, even during flood events. We also understand that during extreme flood events, the banks upstream and downstream of the spillway gate are overtopped.

ESL 2017 measured a race invert level at the aqueduct of RL 197.75m. During our site inspection we measured a vertical distance from the race invert to the top of the gate of 1.97m, which equates to a spillway gate crest of RL 199.72m. We note that this survey mark (MR18G) is located immediately adjacent to the spillway gate on a raised concrete block. The level of the survey mark is RL 199.90m (ESL 2017). The gate crest visually appears to be the lowest point of the race bank along this section of the race.

The LiDAR information is lower than the gate crest (i.e., due to the size of the gate, the LiDAR survey was unable to capture the gate within the survey results). We have therefore, modified the terrain on the true left side of the aqueduct, to raise the true left side of the race to RL 199.72.

We have modelled the Mangaotea Aqueduct bypass as an internal connection. The input parameters are presented with Table 8. The bypass consists of four separate box channels, separated by concrete walls. The size of each box channel was measured during the site inspection as 1250mm wide by 630mm high. For the purpose of this assessment, we have modelled the bypass as a single box culvert with a width of 5.00m. Within some model simulations we have assumed some blockages of the bypass by reducing the culvert width.

Photo 1: Mangaotea Aqueduct and Spillway Gate (February 2019)

The invert level of the bypass has not been surveyed. We have assumed an upstream and downstream invert level of RL 196.8m (i.e., 950mm below the race invert). Some modifications to the terrain were required at the entrance and exit to the box culvert to ensure that the terrain was lower than the assumed box culvert invert levels.

Table 8: Mangaotea Aqueduct Bypass Summary

Parameter	Value
Dimension (H x W) (m)	0.63 x 5.00
Upstream Invert (m RL)	196.8
Downstream Invert (m RL)	196.8

We have assumed that the Little Mangaotea Stream culvert under the race is fully blocked, and therefore have not included the culvert within the model. At the time of the site inspection the culvert entrance and exit were overgrown, and the culvert dimensions were unable to be measured.

3.6 Race Culverts

A number of public (New Plymouth District Council (NPDC)) culverts are located along the race. The culverts pass race flow and provide public access across the race. The culverts are summarised within Table 9.

The culverts have been modelled as internal connections. We were unable to source any drawings for the race culverts, and therefore, we have used dimensions as measured during the site inspection. Culvert invert levels were either estimated using measurements taken during the site inspection, and level information from the LiDAR survey or surveyed during the 16 April 2019 survey. The estimated invert levels are likely to have an order of accuracy of 200mm.

Chris England (Trustpower) identified during the site inspection that the Mangaotea Road culvert restricts flow during higher flows.

Table 9: Culvert Summary

Structure	Culvert Type	Dimension (H x W) (m)	Upstream Invert (m RL)	Downstream Invert (m RL)	Road Level (m RL)	Entrance Loss Coefficient	Manning's
Tariki Road Culvert	Box	2.8 x 2.7	208.7	208.7	212.3	0.5	0.025
Salisbury Road Culvert	Box	1.8 x 2.7	206.152[1.]	206.105[1.]	209.3	0.5	0.025
Mangaotea Road Culvert	Box	1.9 x 2.7	197.312[1.]	197.438[1.]	200.5	0.7	0.013

1. Surveyed 16 April 2019

Photo 2: Salisbury Road Culvert. View towards the south along Salisbury Road

Photo 3: View downstream towards Mangaotea Road Culvert

3.7 Bridges

The numerous bridges over the race have been excluded from the model as they generally appear to be located well above race invert levels. These bridges are presented within Table 10. The vertical distance from the race invert to the lowest point of the bridge is also presented within the table (as measured during the site inspection), along with the estimated underside of bridge levels. These levels have been estimated using the LiDAR information and bridge dimensions measured on-site. Some adjustments to the LiDAR information were required to remove bridges from the terrain.

Table 10: Bridges

Name	Height Above Race Invert (m)	Estimated Underside of Bridge Level (RL m)
Upper Silt Pond Bridge	2.7	211.4
Lower Silt Pond Bridge	4.4	213.2
Tariki Road Bridge	2.2	210.6
Salisbury Road Bridge	3.7	206.3
In Race Generator Bridge	4.7	205.2
Upper Mangaotea Bridge	Not inspected	201.1
Lower Mangaotea Bridge	Not inspected	199.368[1.]
Thompsons Bridge	Not inspected	201.3

1. Surveyed 16 April 2019

4.0 Mangaotea Stream Culvert

A culvert is located on the Mangaotea Stream to convey flows under Tariki Road South. The culvert entrance and exit were overgrown at the time of the inspections and therefore culvert dimensions are unknown. Assumed culvert information is presented within Table 11. The culvert has been modelled as an internal connection within the race 2D flow area.

Table 11: Mangaotea Stream Culvert

Culvert Type	Diameter (m)	Upstream Invert (m RL)	Downstream Invert (m RL)	Entrance Loss Coefficient	Manning's
Pipe	1.5	197.30	197.25	0.5	0.025

5.0 Ratapiko Culverts

We understand that there are two culverts (as discussed with Chris England) under the Ratapiko Road that connects the two portions of the lake. Only one Ratapiko Road culvert was located during the survey. This culvert was installed more recently than the other culvert (installed February 2003) and is a 2.4m diameter reinforced concrete culvert with wingwalls.

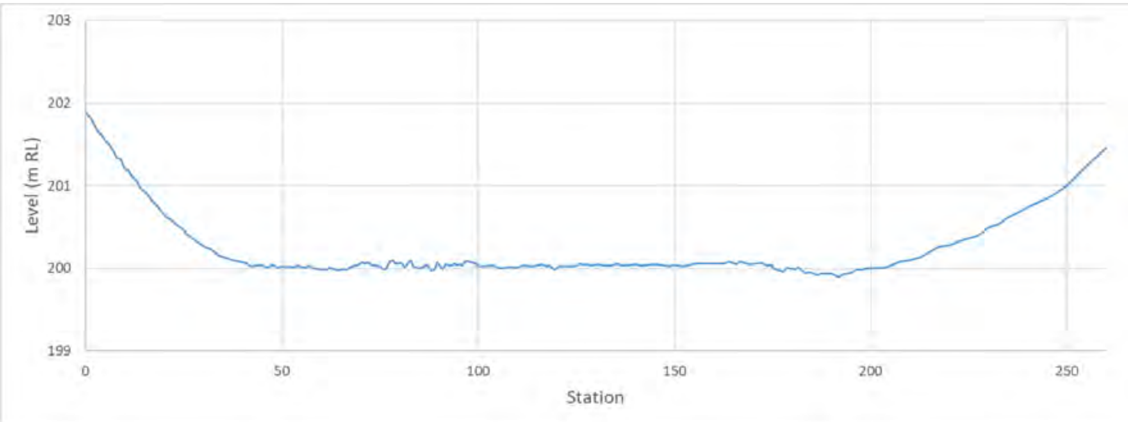
We have no information regarding the original culvert (although it has been suggested that it is also 2.4m diameter). We recommend that further attempts are made to locate and survey the culvert. Table 12 presents the culvert information used within the model. The original culvert was assumed to have the same diameter and invert levels as the surveyed culvert. Figure 3 presents weir/road profile as derived from the LiDAR survey and as used within the models.

Table 12: Ratapiko Road Culverts

Number of Culverts	Culvert Type	Diameter (m)	Upstream Invert (m RL)	Downstream Invert (m RL)	Entrance Loss Coefficient	Manning's
2	Circular	2.4	196.034[1.]	195.944[1.]	0.7	0.013

1 One culvert surveyed 16 April 2019.

Figure 3: Ratapiko Road Weir Profile

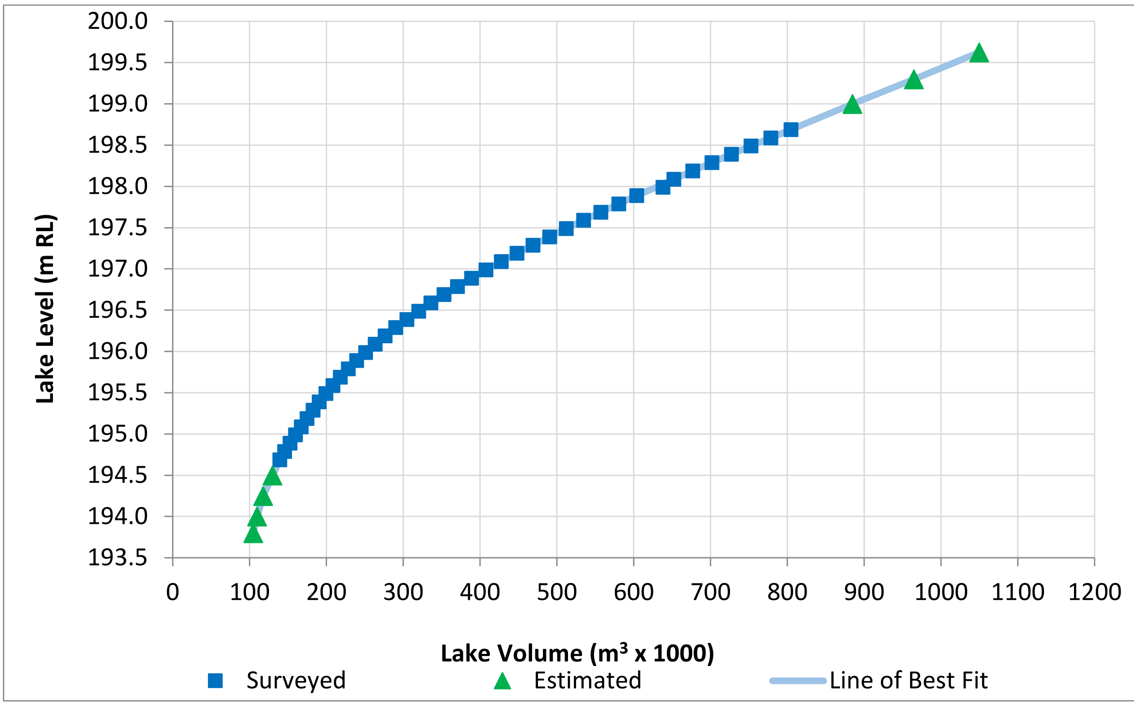


We note that the 2018 LiDAR indicates that there is head loss of approximately 100mm across the Ratapiko Road culverts during normal operating conditions. We also note that potential blockage of the culverts has not been considered.

6.0 Lake Ratapiko

Lake Ratapiko provides attenuation of flood inflows by temporarily storing flood waters within the lake basin. Ryder (2010) used a bathymetric survey undertaken by Hartley Contractors Ltd in May 2008 as the basis of the storage assessment as presented within Figure 4.

Figure 4: Ryder (2010) Lake Storage



We have separated the Lake Ratapiko into two separate storage areas, to reflect the upper and lower lake areas, separated by Ratapiko Road and the associated culverts. Within HEC-RAS storage areas differ from 2D flow areas, as storage areas use the level pool routing method (i.e., with a storage area the water level is level across the area at even given point in time). The advantage of using storage areas over 2D flow areas is the computational time and the ability to access and amend storage capacity easily. We have used two separate storage areas to allow for the different catchment areas draining to each portion of the lake and to model the attenuation effect that the Ratapiko Road culverts may have.

The 2018 LiDAR and 2019 photogrammetry surveys have provided further information, which has been used to develop the elevation storage relationship. The two survey datasets were combined to derive the elevation storage relationship as presented within Figures 5, 6 and 7. The revised relationship provides lower storage volumes at a given lake level than Ryder 2010.

Figure 5: Elevation Storage Relationship – Upper Lake

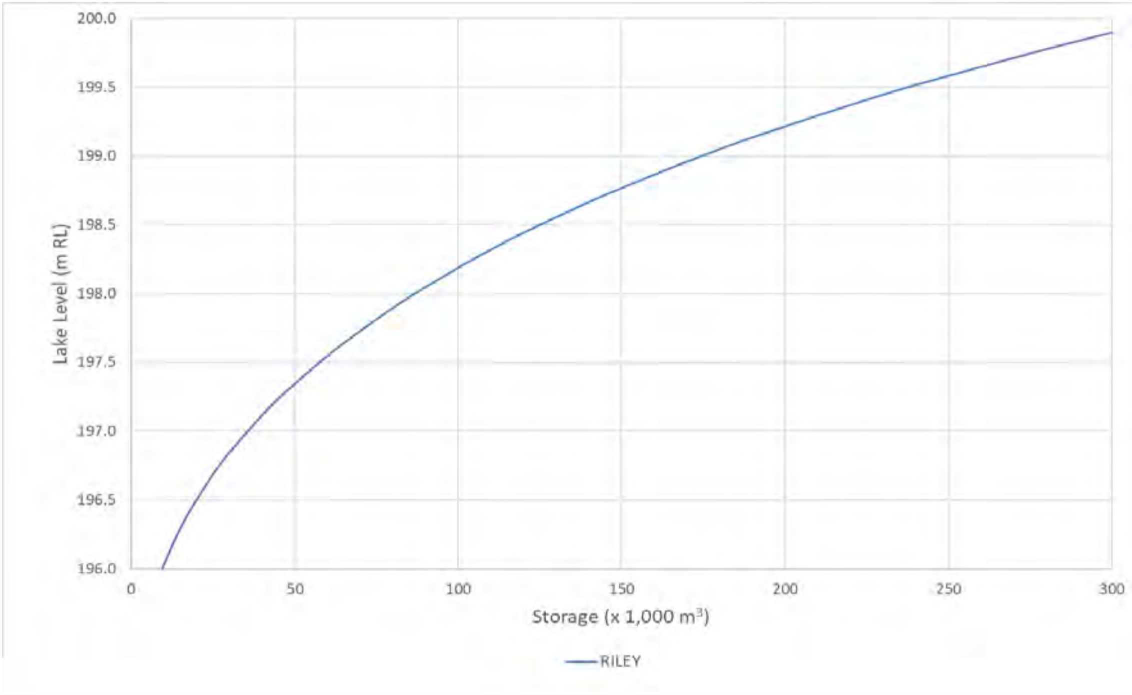


Figure 6: Elevation Storage Relationship – Lower Lake

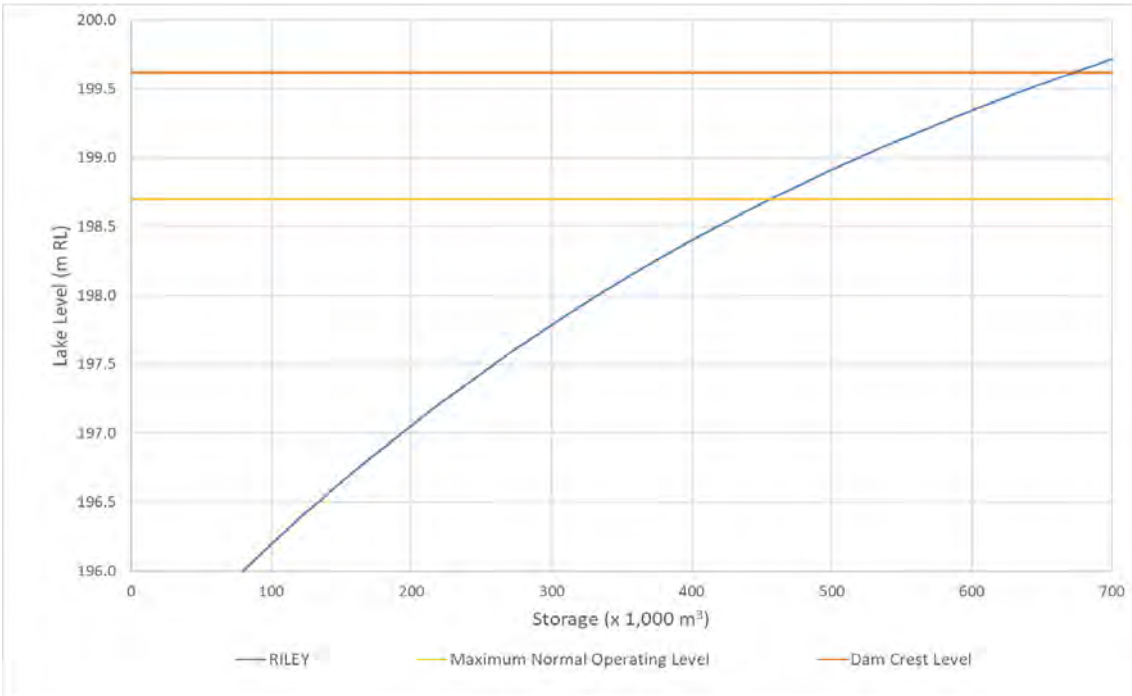
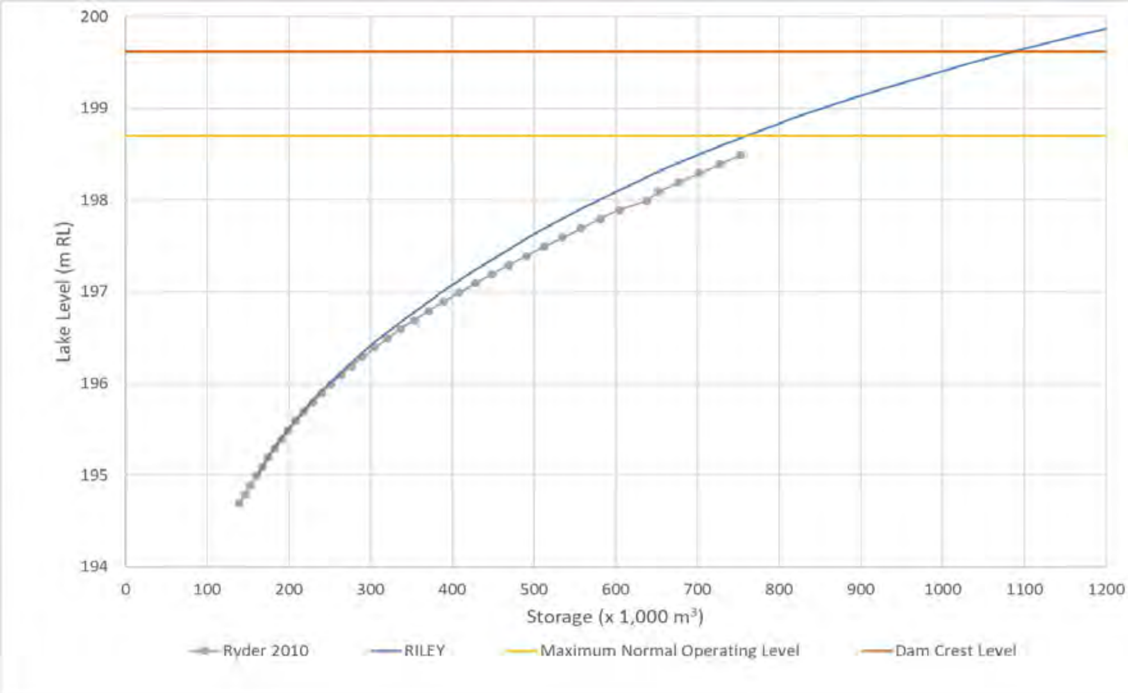


Figure 7: Elevation Storage Relationship – Comparison with Ryder 2010



7.0 Dam Spillways

The dam has both a service and flood spillway. The service spillway is located to the true right of the dam. The spillway consists of a concrete weir constructed within an excavated open channel. The weir is approximately 20m long and crosses the channel at an oblique angle. A topographical survey by Energy Surveys Ltd in 2013 measured the weir length at 19.59m.

Tonkin and Taylor Ltd (T&T), 1999 – Ratapiko Dam and Manganui Diversion Weir 1998 SEED Inspection Form states that the flashboards at the time were 300mm high and designed to fuse (i.e., break fully across the spillway width and no longer retain any static water) with 300mm of water over top of the flashboards. In 2003/2004 spring-loaded gates were installed to replace the previous flashboards.

More recently, five tipping flashboards, each 3.9m long and 300mm high, were installed on top of the weir in 2014. A completion report for the installation of the flashboards was prepared by Pattle Delamore Partners Ltd in 2014. The report made recommendations regarding the testing of the flashboards. It is not clear if these recommendations have been undertaken. We recommend that testing of the flashboards is undertaken and documented if it has not been done already. It is also not clear from the report what the effective weir crest elevation is when the flashboards have tipped (i.e., the flashboard will lie across the concrete crest). We have assumed that the effective weir height when tipped is RL 198.47m (i.e., 100mm above the concrete crest), unless otherwise stated.

Photo 4: Service Spillway Flashboards (Photograph taken in 2004)

Photo 5: Service Spillway Flashboards (Photograph taken in 2016)

The flood spillway is located immediately to the true right of the dam, between the dam and the service spillway. The spillway consists of an overflow weir arrangement, leading to an excavated open channel, which also discharges into a natural gully that leads to Mako Stream. The flood spillway was constructed in 1994, following the T&T (1994) – Ratapiko Dam Rehabilitation Design and Construction Report.

The flood spillway has a low height fuse plug located at the upstream end of the channel. As-built drawings indicate that the crest of the fuse plug is RL 199.22m. The fuse plug has been eroded (at least in part) by flood events in 2004 and 2015, and subsequently reconstructed. Once the fuse plug is breached, the concrete driveway downstream of the fuse plug is intended to be the control structure. It is not clear at what lake level the plug is designed to breach.

We note that as-built drawings of the flood spillway (T&T, 1994) indicate that the crest of the spillway downstream of the fuse plug is RL 198.85m. The LiDAR survey indicates that the crest is approximately RL 198.96m. We have conservatively used the LiDAR information as part of this assessment.

The LiDAR indicates that the that the sill width is approximately 15m. We note that the as-built drawings indicate that the downstream spillway channel invert is 14m wide. Figure 8 presents the assumed flood spillway cross section used within the model.

Figure 8: Flood Spillway Cross Section

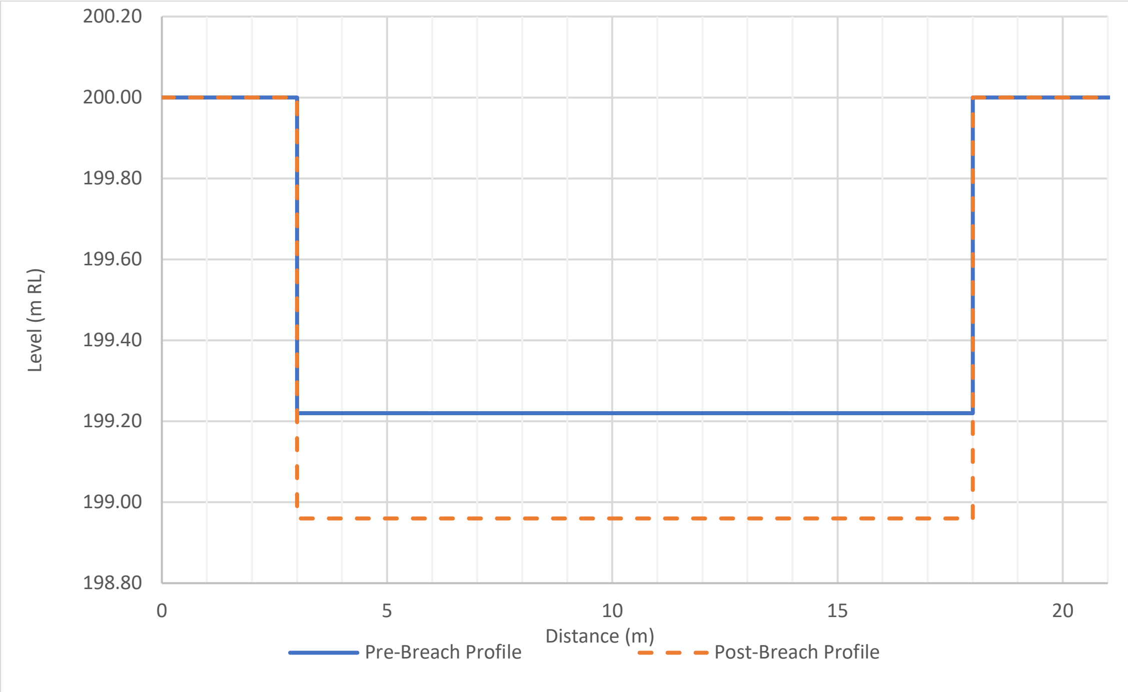


Table 13 presents a summary of the relevant elevations at the dam. We note that some different assumptions have been made for the calibration events, to reflect records and observations from the events. These differences are presented within the relevant calibration section (Appendix F).

Table 13: Elevations

Element	Elevation (m RL)
Consented Minimum Reservoir Level	194.00
Service Spillway Concrete Weir Crest	198.37
Flashboard Crest (Normal Maximum Reservoir Level)	198.67
Consented Maximum Reservoir Level	198.70
Flashboard Tipping Point Reservoir Level	198.70
Fuse Plug Crest	199.22
Flood Spillway Crest (driveway downstream of fuse plug)	198.96
Dam Crest	199.62

8.0 Tributary Storage Areas

Tributary 1 and 2 have been modelled as two separate storage areas. The storage relationship is based on the HEC-RAS terrain derived from the 2018 LiDAR as presented within Figure 9. The two storage areas are connected to the Lower Lake storage area via culvert/weir connections. The culverts have not been measured or surveyed as part of this assessment. The assumed culvert information used within the model is presented within Table 14. We recommend that the culverts are surveyed as part of future assessments.

Figure 9: Tributary Storage Curves

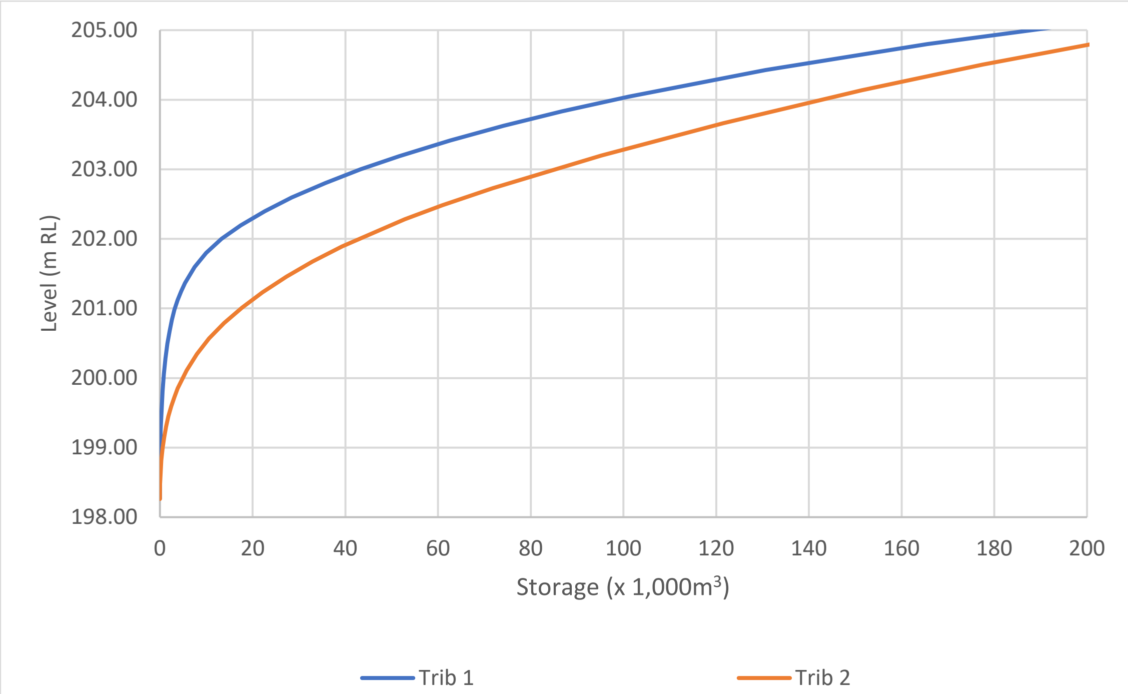


Table 14: Tributary Culvert Summary

Structure	Number of Culverts	Culvert Type	Diameter (m)	Upstream Invert (m RL)	Downstream Invert (m RL)
Tributary 1	1	Circular	1.25	198.4	198.4
Tributary 2	1	Circular	0.90	198.3	198.3

Figures 10 and 11 present the weir profiles as derived from the LiDAR survey and as used within the model.

Figure 10: Tributary 1 Weir Profile

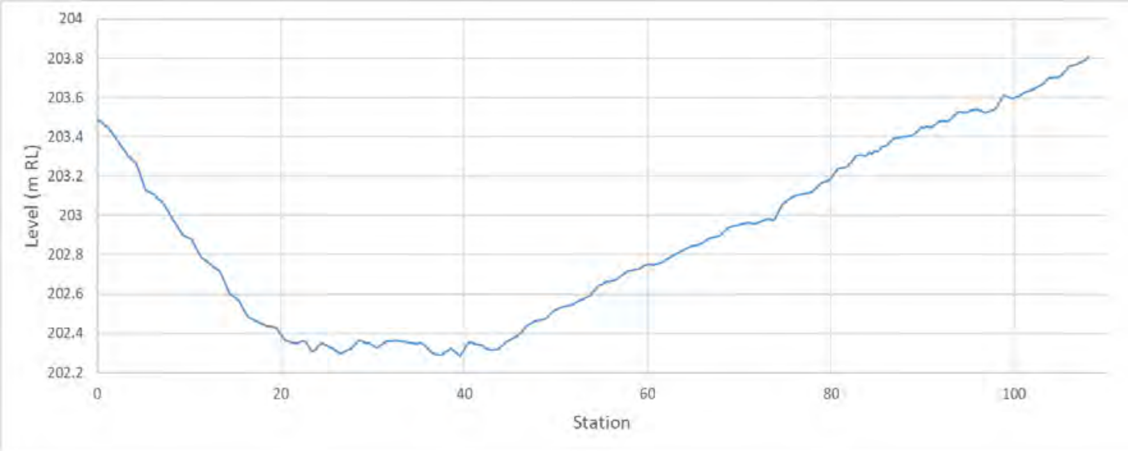
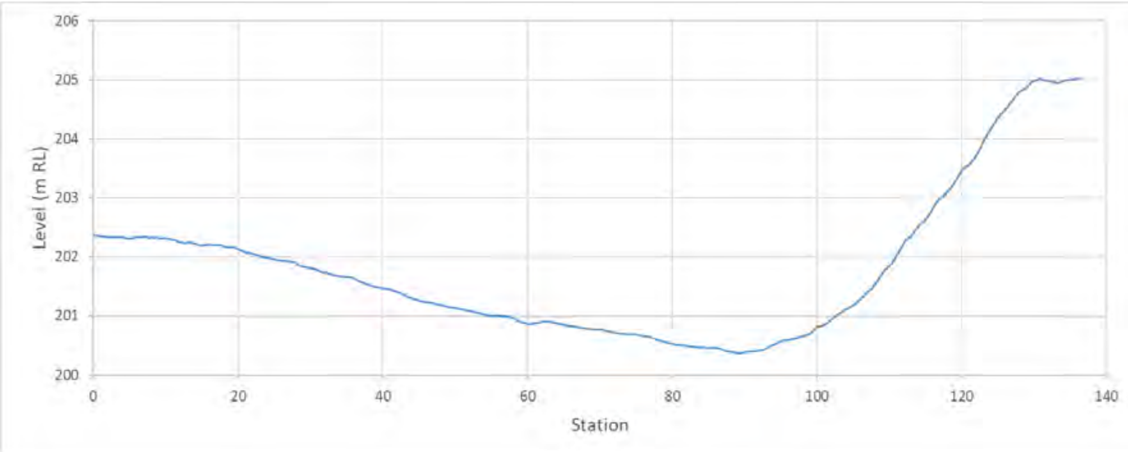


Figure 11: Tributary 2 Weir Profile



9.0 Spillway 2D Flow Area

The Spillway 2D flow area is connected to the Lower Lake storage area by the two spillway connections discussed previously. The flow area extends downstream past Ratapiko Dam.

10.0 Downstream Boundary Conditions

A downstream boundary condition has been used downstream of the dam and the Mangaotea Stream culvert. Both downstream flow boundary conditions use a downstream slope of 0.01 within a normal depth calculation.

11.0 Time Series Data

Flow hydrographs derived from the HEC-HMS model were used as inflow hydrographs at relevant locations within the model as presented with the HEC-RAS schematic. The Tariki Weir flow was also modelled as an inflow hydrograph at the weir location.

The penstock outflow was modelled as an outflow hydrograph (an inflow hydrograph with negative values) within the Lower Lake storage area.

All-time series data used 15 minute time intervals.

APPENDIX E

Calibration Background

Calibration Background

1.0 Objective and General Methodology

The objective of the calibration is to ensure that the combined HEC-HMS and HEC-RAS models best represent the hydrological and hydraulic behaviour of the catchment. We have placed an emphasis on matching the modelled accumulated runoff volume with derived accumulated runoff volume at the lower lake.

For each of the selected flood calibration events, catchment and hydraulic parameters have been varied until a good match with recorded data and derived information has been obtained.

The calibration model assumes that the same rainfall distribution occurs across the entire catchment area for a particular event. In reality, this is unlikely to occur and, therefore, a perfect match with recorded data and derived information is unlikely to occur.

Future assessments could attempt to refine the calibrations undertaken.

2.0 Inflow Derivation

The following methodology was used to derive an inflow series for the lower lake:

1. Stored lake volume time series derived (based on lake level record and the lower lake storage elevation relationship).
2. Net inflow for 15 minute period based on change in stored lake volume divided by change in time (15 minutes).

The gross and runoff inflows were subsequently derived using the following water balance equations:

Gross Inflow = Net Inflow + Penstock Outflow + Spillway Outflow

Runoff Inflow = Gross Inflow – Diversion Inflow/Tariki Weir Flow

Table 1 presents the annual maxima derived from the assessment.

Table 1: Annual Maxima - Peak Lower Lake Runoff Inflows

Rank	Date	Peak Lower Lake Runoff Inflow (m^3/s)
1	20 June 2015	42.4
2	27 September 2003	20.7
3	16 July 2012	19.0
4	28 February 2004	18.2
5	17 November 2006	16.5
6	3 October 2011	16.5
7	2 July 2007	16.4
8	12 July 2008	15.6
9	30 August 2009	15.1
10	30 September 2010	14.9
11	27 July 2017	14.6
12	24 July 2016	14.4
13	3 August 2014	13.6
14	6 January 2005	12.3
15	17 June 2013	10.3
16	21 May 2002	10.0

3.0 Event Selection

The approximate critical rainfall duration for the lake is 24-hours, noting that the critical duration event will vary with rainfall. Table 2 presents an annual maxima series derived from the recorded lake level data. Table 3 also presents a summary of the highest recorded lake and Lower Mangaotea levels.

Table 2: Annual Maxima Lake Levels

Rank	Date	Peak Water Level (RL m)
1	20 June 2015	199.26
2	28 September 2003	198.99
3	28 February 2004	198.99
4	16 June 2012	198.93
5	18 November 2006	198.85
6	6 September 2010	198.83
7	31 August 2009	198.8
8	12 July 2008	198.79
9	2 July 2007	198.78
10	4 October 2011	198.77
11	3 August 2014	198.75
12	24 July 2016	198.73
13	6 January 2005	198.73
14	17 July 2017	198.7
15	22 September 2013	198.65

Table 3: Highest Recorded Lake Levels

Peak Lake Level Rank	Event	Peak Lake Level (m RL)	Flood Spillway Operated	Peak Lower Mangaotea Rank
1	19 June 2015	199.26 (1)	Yes	1
2	27 February 2004	198.99 (2)	Yes	2
3	28 September 2003	198.99 (3)	Not Confirmed	3

Table 4 presents a summary of the largest rainfall events, recorded at the three rain gauges.

Table 4: 24-Hour Rainfall Depths and Rank

Event	Inglewood Gauge	Boyd Gauge	Jet Boat Club Gauge
19 June 2015	179 (1)	104 (5)	167 (1)
27 February 2004	176 (2)	129 (1)	123 (5)
28 September 2003	163 (4)	118 (2)	n/a (data gap)

We have selected the 2015 and 2004 events to calibrate, as both resulted in the highest and second highest recorded lake levels respectively, noting that the flood spillway operated in both these events. We note that the peak lake level recorded during the 2004 event is below the design crest level of the fuse plug. We therefore assume that the fuse plug had not been reinstated to the design crest level.

The flood spillway operated during the 2000 flood event (RILEY Ref: 00MTK/ENG-C), however, the powerhouse was out of service at the time and, therefore, the tunnel could not be used to pass a portion of the flood inflows. A flow depth of 50mm to 100mm was observed within the flood spillway channel.

We have also calibrated two smaller events, June 2013 and July 2017. These smaller events were calibrated to provide an understanding of the catchment response for events in the order of a mean annual event. Specifically, the June 2013 event was calibrated as no spill flows occurred during the event, meaning that some of the spillway assumptions that have been made did not affect the model results.

4.0 Rainfall Gauge Selection

Table 5 presents a comparison between the different gauges for the selected calibration events. The table shows that the Inglewood gauge records do not appear to typically represent the rainfall within the catchment.

We also note that the mean annual rainfall at Inglewood (north-west of site) is 2397mm and 1766mm at Tarata (north-east of site) (The Climate and Weather of Taranaki, NIWA, 2014). We would anticipate that the Motukawa catchment would have a similar mean annual rainfall to Tarata, due to its proximity. The mean annual rainfall ratio between the Tarata and Inglewood gauges is 0.74 (1766mm/2397mm). This ratio is similar to those presented within Table 5.

Table 5: Selected Calibration Events – 24-Hour Rainfall Depths

Event	Inglewood Depth (mm)	Boyd Depth (mm)	Boyd/Inglewood Ratio	Jet Boat Club Depth (mm)	Jet Boat Club/Inglewood Ratio
27 February 2004	176	129	0.73	123	0.70
19 June 2013	111	70	0.63	75	0.68
17 June 2015	178	104	0.58	167	0.94
27 July 2017	77	24	0.31	46	0.60

Table 6 provides some comments on the observed rainfall during the selected calibration events.

Table 6: Selected Calibration Events – Rainfall Gauge Selection

Event	Inglewood Comment	Boyd and Jet Boat Club Comment	Selected Calibration Gauge
27 February 2004	Higher rainfall total than both Boyd and Jet Boat Club.	Boyd and Jet Boat Club very similar total and rainfall pattern.	Boyd
19 June 2013	Higher rainfall total than both Boyd and Jet Boat Club.	Boyd and Jet Boat Club very similar total and rainfall pattern.	Boyd
17 June 2015	Higher rainfall total than both Boyd and Jet Boat Club.	Boyd total significantly lower than Jet Boat Club.	Jet Boat Club
27 July 2017	Higher rainfall total than both Boyd and Jet Boat Club.	Boyd total significantly lower than Jet Boat Club.	Jet Boat Club

Based on this, we consider the Inglewood gauge is typically receives higher rainfall than the Motukawa catchment for any given event (noting that we have identified some exceptions). Therefore, we consider that the use of the Inglewood gauge for calibration purposes is not appropriate, when other data within the catchment is available.

5.0 Available Race Water Level Data

There are some significant gaps and erroneous data within the race water level dataset. Table 7 provides a summary of the available data. A cross within the table indicates the data is not available or erroneous.

Table 7: Race Water Level Data

Event	Salisbury	Mangaotea Culvert	Aqueduct	Lower Mangaotea
27 February 2004	✓	✓	×	✓
19 June 2013	×	×	✓	×
17 June 2015	✓	×	✓	✓
27 July 2017	✓	✓	✓	✓

The Mangaotea Culvert, Aqueduct and Lower Mangaotea levels are typically affected by lower lake levels (except during lower lake levels) and therefore typical water levels during normal operating conditions range significantly. At the Salisbury gauge, the water level at a normal operating flow of $5.2m^3/s$ was RL 204.12m immediately prior to both the 2004 and 2015 events.

6.0 Peak Rate Factor

Gauged flow data is not available for each sub catchment, so we have used a global PRF across all sub catchments. We have used a PRF of 450 for the assessment, as we found it generally gave a good fit to the calibration events.

7.0 Baseflow Coefficients

A baseflow coefficient of 10 hours has been used for the smaller catchments, with higher values for the larger catchments.

8.0 Estimated and Modelled Inflow Hydrographs

A smoothed lake level record was prepared for each of the calibration events to improve the estimated inflow hydrograph.

The modelled lower lake runoff inflow hydrograph was extracted from the model results and from recorded data as follows:

Modelled Runoff Inflow = Ratapiko Road Modelled Inflow + Tributary 1 Inflow Modelled + Tributary 2 Modelled Inflow + Lower Lake Local Modelled Inflow – Tariki Weir Recorded Flow

APPENDIX F

Calibration

Calibration

1.0 February 2004

1.1 Recorded Rainfall

The 27 February 2004 rainfall event caused significant flooding across the North Island (associated with ex-tropical Cyclone Ivy). Other significant rainfall events also occurred earlier in February 2004, with February 2004 the wettest month on record at the Inglewood gauge. Figure 1 presents the recorded lake level and cumulative rainfall at the Boyd gauge during February 2004 (calendar month). The figure demonstrates that the lake level response (and therefore inflows) during the 28 February 2004 event were significant compared to other similar rainfall events earlier in the month. This indicates that initial losses in summer months are typically high.

Figure 1: Recorded Accumulated Rainfall and Lake Level

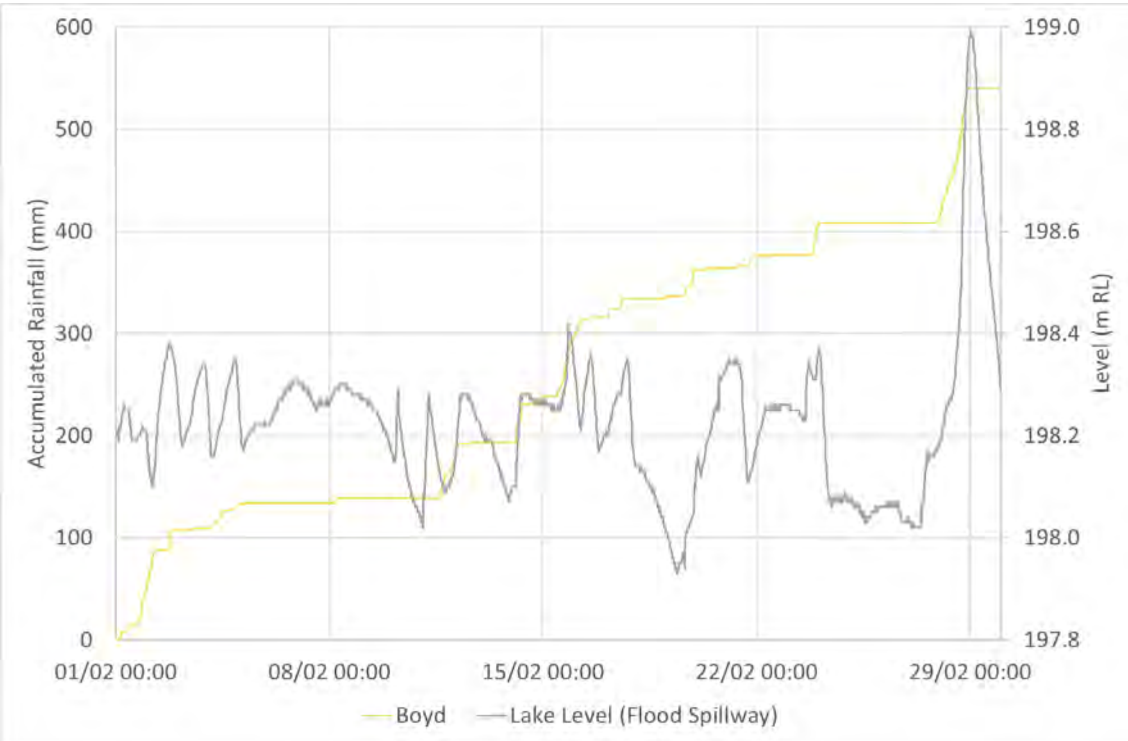
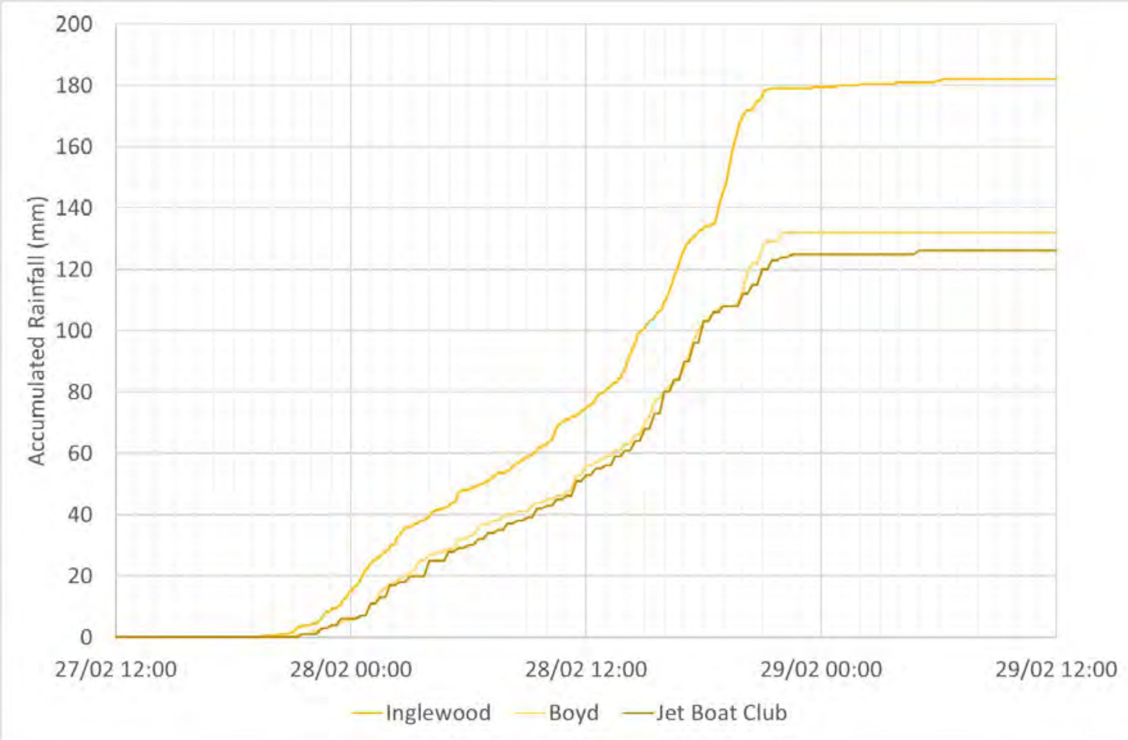


Figure 2 presents the recorded cumulative rainfall during the event. The Jet Boat Club gauge and the Boyd gauge recorded similar rainfall totals during the 28 February 2004 event. The total accumulated rainfall at the Trustpower gauges was approximately 70% of that recorded at Inglewood. We consider the both the Jet Boat Club gauge and Boyd gauge are appropriate to use within the calibration. For the purposes of this assessment, we have used the Boyd Gauge data.

Figure 2: February 2004 - Recorded Accumulated Rainfall

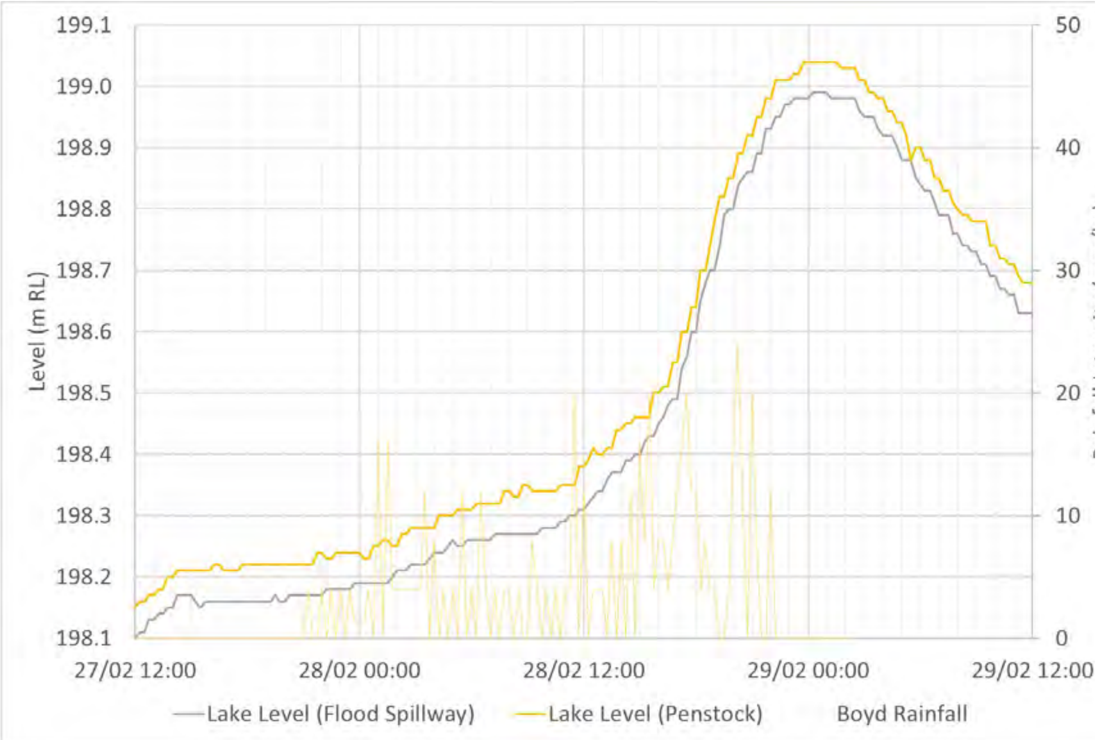


Steady rainfall occurred over an approximate 24-hour period, from 9pm on 27 February through to 10pm on 28 February.

1.2 Recorded Lake Levels

Figure 3 presents data from the two lake level gauges that were in operation in 2004. As noted previously, the penstock water level gauge typically records levels approximately 50mm higher than the flood spillway gauge. The lake level peaked at around 12.30am on 29 February 2004. We have elected to use the flood spillway records for calibration purposes as it is located at the dam (peak water level 198.99m).

Figure 3: February 2004 - Recorded Lake Levels and Rainfall



1.3 Recorded Canal Levels

Figure 4 presents the recorded canal levels in the lower canal downstream of the in-race generator. There is a period of erroneous data for the Lower Mangaotea gauge, which we have removed from the figure for clarity. The two canal water level gauges are influenced by the downstream lake level. The Mangaotea Road gauge data appears to be significantly affected by the downstream Mangaotea Road culvert (obvert RL 199.21m, surcharged by approximately 600mm) as shown by the period of relatively steady water levels at the peak. The LiDAR indicates that the canal would have been spilling over the true left canal bank by up to 300mm (i.e., recorded peak water level RL 199.87m versus lowest canal top of bank of RL 199.58m)

The LiDAR survey also indicates that spill flows commence from the canal at water levels above RL 199.49m at a point between the aqueduct and Tunnel 2 (Lower Mangaotea gauge), and therefore it appears that spill flow occurred at this location as well.

The recorded canal levels were well in excess of the Lower Lake levels at the peak (by 600-800mm). We therefore consider that there is evidence that the Mangaotea Stream overtopped into the canal. We also note that the canal peak levels occurred prior to the Lower Lake peak (at approximately 12:45am) by approximately two hours.

Figure 4: February 2004 – Recorded Lower Canal Levels

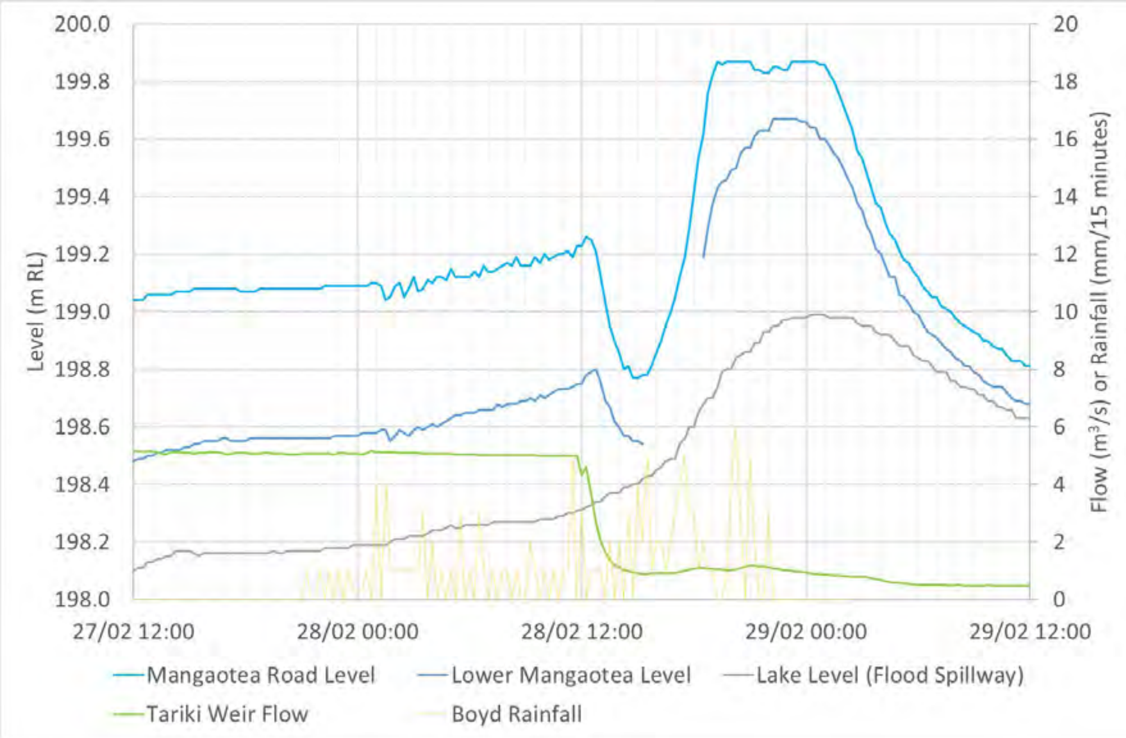


Figure 5 presents the Salisbury water level gauge data. We note that a peak water level (and therefore peak flow) occurred at the Salisbury gauge at 10.15pm, indicating a lag time of approximately two hours to the gauge. The peak recorded water level of RL 204.14m, is similar to that recorded during normal operating conditions (i.e., with no local catchment inflow and a flow take of $5m^3/s$), indicating that the peak catchment runoff inflow was also approximately $5m^3/s$. It would also appear that the canal did not overtop its banks upstream of Tunnel 1.

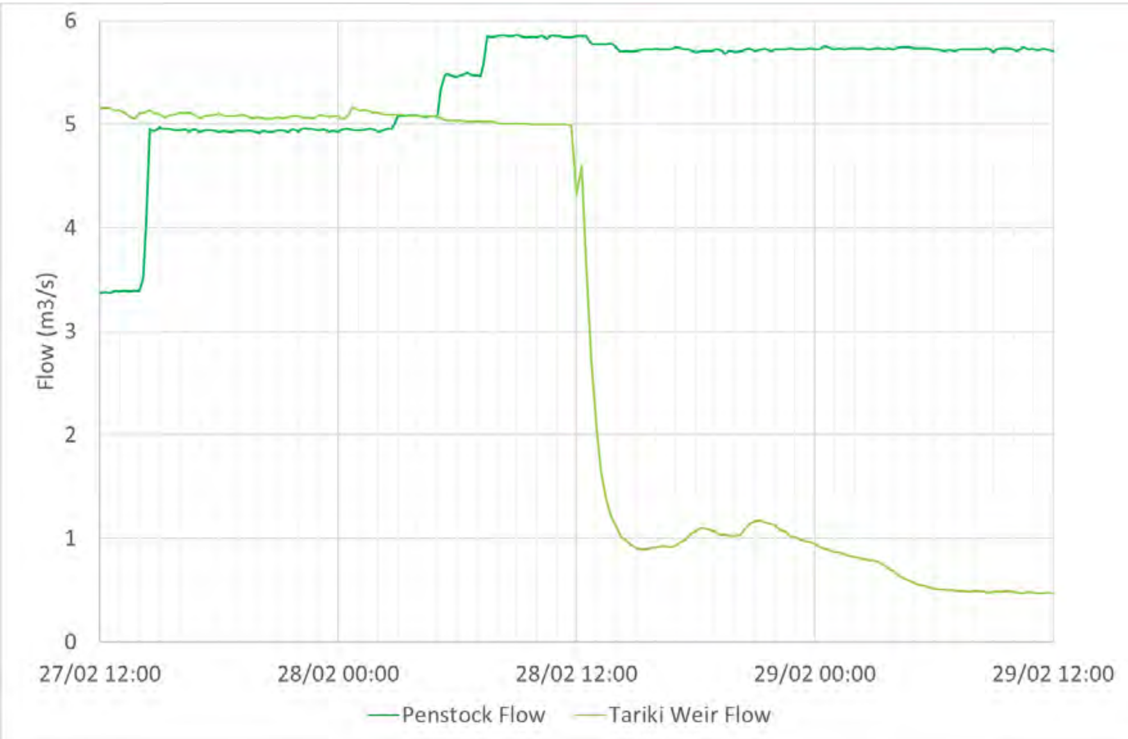
Two separate peaks are visible within both the Tariki Weir flow data and the Salisbury data.

Figure 5: February 2004 – Recorded Upper Canal Levels

1.4 Recorded Flows

The recorded flow hydrographs are presented within Figure 6.

Figure 6: February 2004 - Recorded Flow



1.5 Spillway Operation and Derived Inflow

We understand the spring loaded flashboards were in place at the time of the event. It is not clear if the flashboards were activated (i.e., tipped) during the event. For the purposes of this assessment, we have compared two scenarios:

1. No activation of the tip boards
2. Full activation of the tip boards with lake levels above RL 198.60m, with an effective sill level of RL 198.47m

We have selected an activation level of RL 198.60m as on the falling limb there appears to be a change in lake level gradient at this level. We note that this assumption does not effect the peak derived inflow.

We understand that the flood spillway did operate during the flood event, even though the lake level did not reach the design fuse plug crest level. Andecaotally, the fuse plug was removed in part by Trustpower during the event to allow some flow discharge at the flood spilllway. The flow through the flood spillway is likely to have been relatively small compared to the service spillway. For the puposes of this asesssment, we have assumed that no flow through the flood spillway ocurred.

Figure 7 presents the two derived runoff inflow scenarios. We consider the full activation inflow hydrograph appears to provide a better hydrograph shape than the no activation scenario. The rising limb on the no activation scenario appears too flat compared to the falling limb and the peak flow of 11.2m^3/s appears low for a significant rainfall event compared to derived inflows from other events. We note that higher derived spill flows will require lower catchment losses within the calibration process, which is conservative. We make the following comments based on a review of the selected full activation inflow hydrograph:

- The hydrograph consists of a single peak.
- The baseflow commences immediately after the rainfall commences, with a slow steady increase in runoff.
- The flow up to approximately 12noon on 28 February appears to consist of baseflow only, without any direct runoff.
- The baseflow is approximately 2m^3/s at 12noon on 28 February.
- The peak runoff inflow of approximately 18m^3/s is delayed significantly (i.e., lags) relative to the recorded rainfall, with the peak occurring at approximately 11pm on 28 February, approximately 2.75 hours after the peak rainfall intensity occurred at 8.15pm.
- The baseflow at 12noon on 29 February is likely to be approximately 5m^3/s (i.e., direct flow will be nil at the point of inflection on the hydrograph).

If baseflow only occurred up until 12noon 28 February, this indicates that the initial loss is high. The Boyd rainfall record indicates that 56mm of rainfall occurred up until this time.

Figure 7: February 2004 – Derived Runoff Inflow

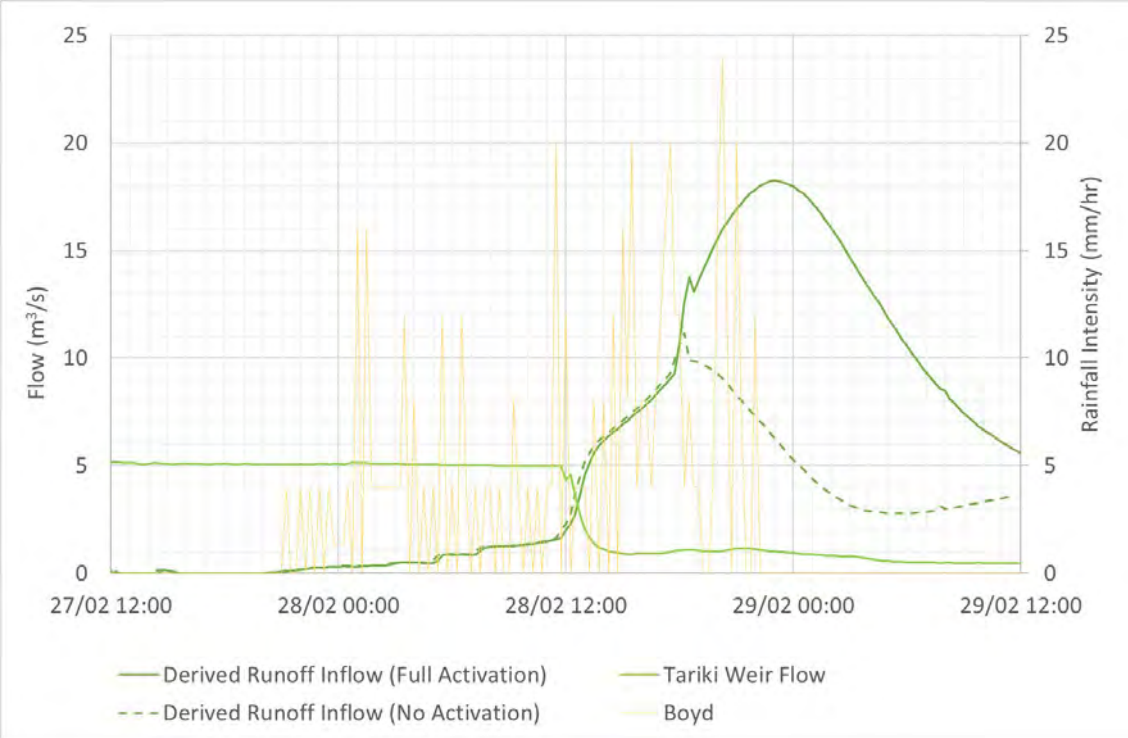


Figure 8 presents the derived spillway hydrograph.

Figure 8: February 2004 – Derived Spillway Flow

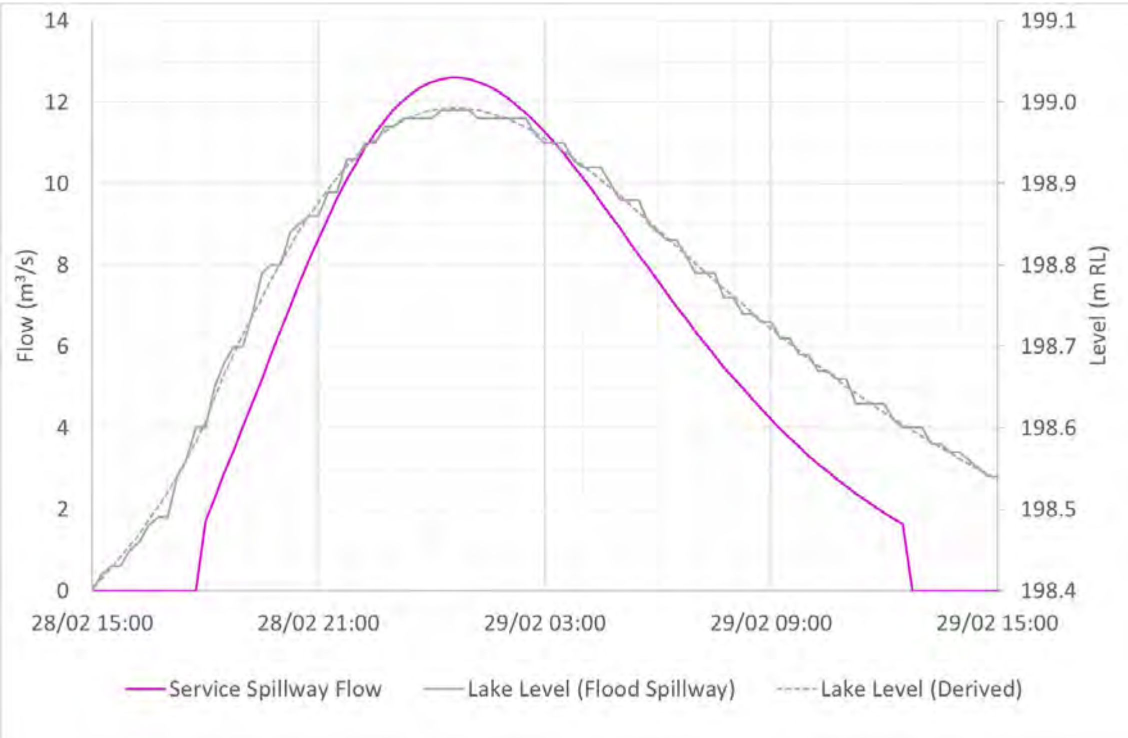
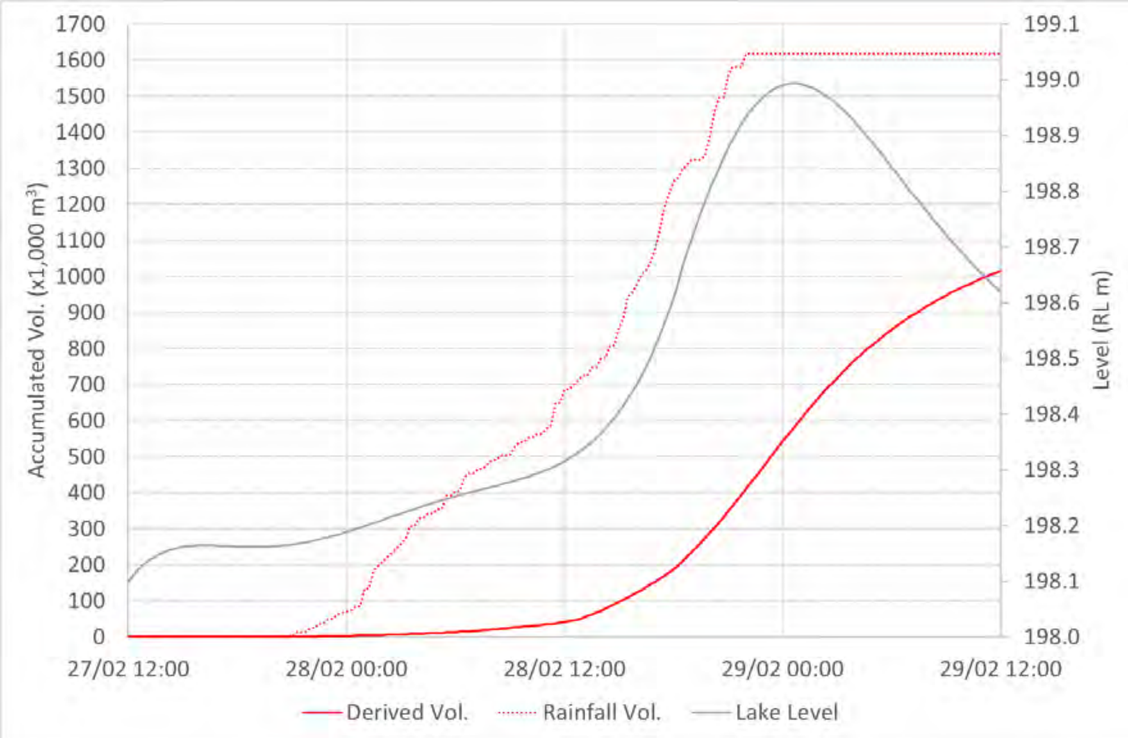


Figure 9 presents the derived runoff inflow accumulated volume.

Figure 9: February 2004 – Derived Volume



1.6 Calibration Parameters

We have used the variable input parameters as summarised within Table 1.

Table 1: February 2004 - Calibration Parameters

Catchment	Initial (mm)	Constant (mm/hr)	PRF	Lag Time (minutes)	Initial Discharge (m^3/s)	Baseflow Coefficient (hr)	Baseflow Fraction	Baseflow Steps
Salisbury	75	3	450	75	0.01	10	1.0	2
Mangaotea Road	75	3	450	75	0.01	10	1.0	2
Lower Mangaotea	75	3	450	75	0.01	40	1.0	2
Tunnel 3	75	3	450	75	0.01	10	1.0	2
Upper Lake	75	3	450	75	0.01	10	1.0	2
Lower Lake	75	3	450	75	0.01	10	1.0	2
Tributary 1	75	3	450	75	0.01	40	1.0	2
Tributary 2	75	3	450	75	0.01	20	1.0	2
Mangaotea	75	3	450	120	0.01	60	1.0	2

1.7 Results and Discussion

Figures 10, 11, 12, 13 and 14 present the results of the calibration. We consider the calibration provides a conservative match.

Figure 10: February 2004 – Modelled Levels

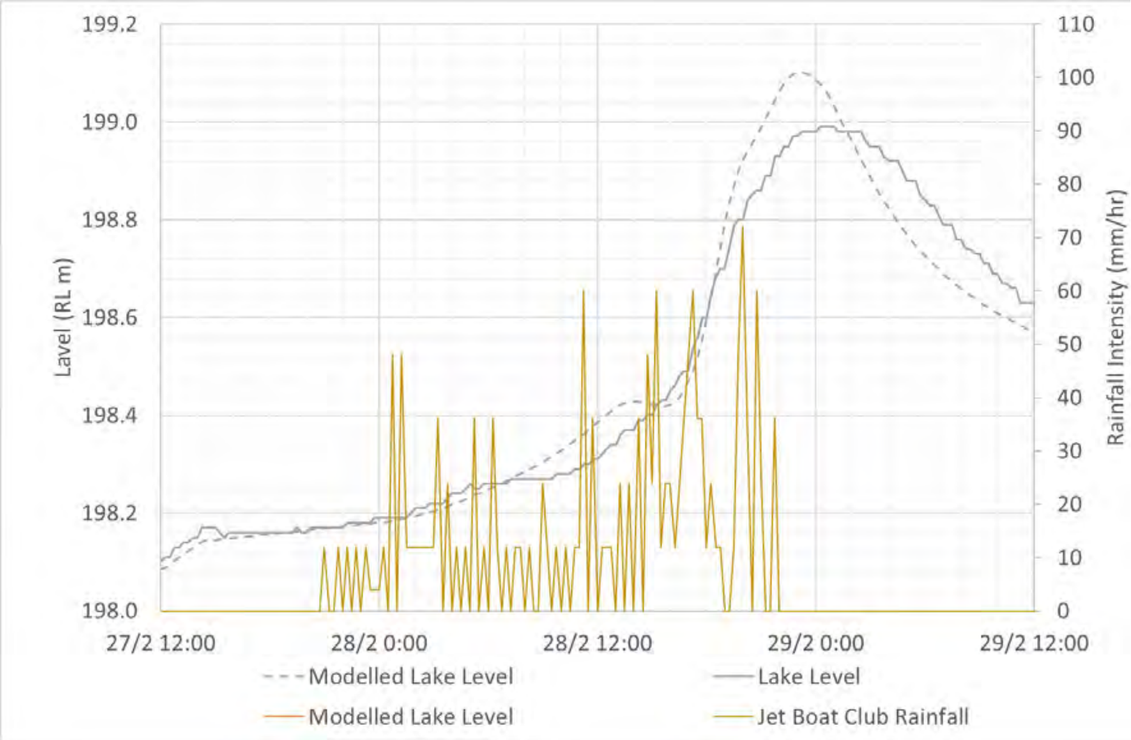


Figure 11: February 2004 – Modelled Runoff Inflow

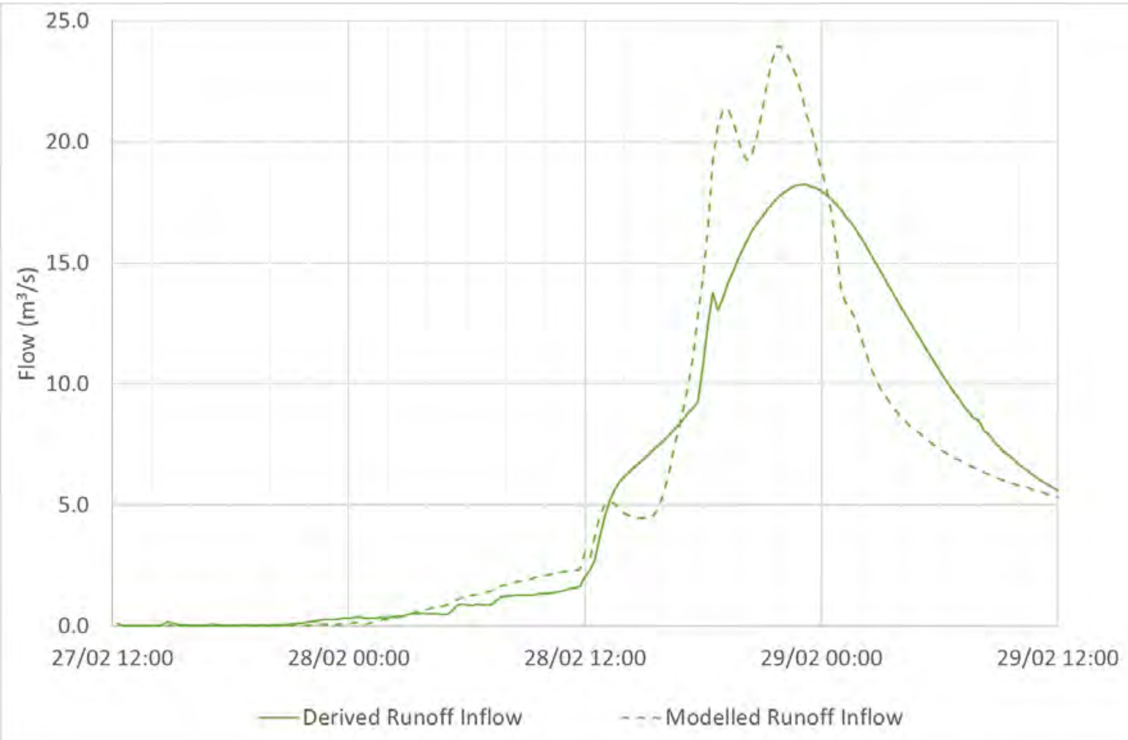


Figure 12: February 2004 – Modelled Lake Volume

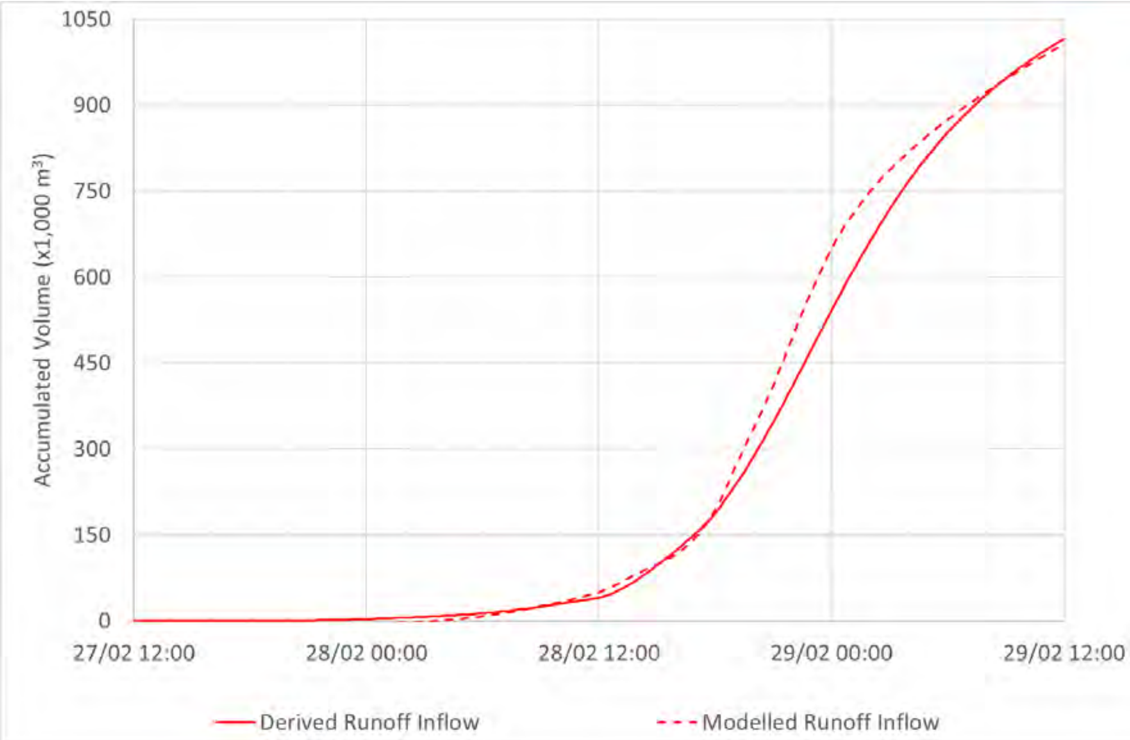


Figure 13: February 2004 – Modelled Lower Canal Levels

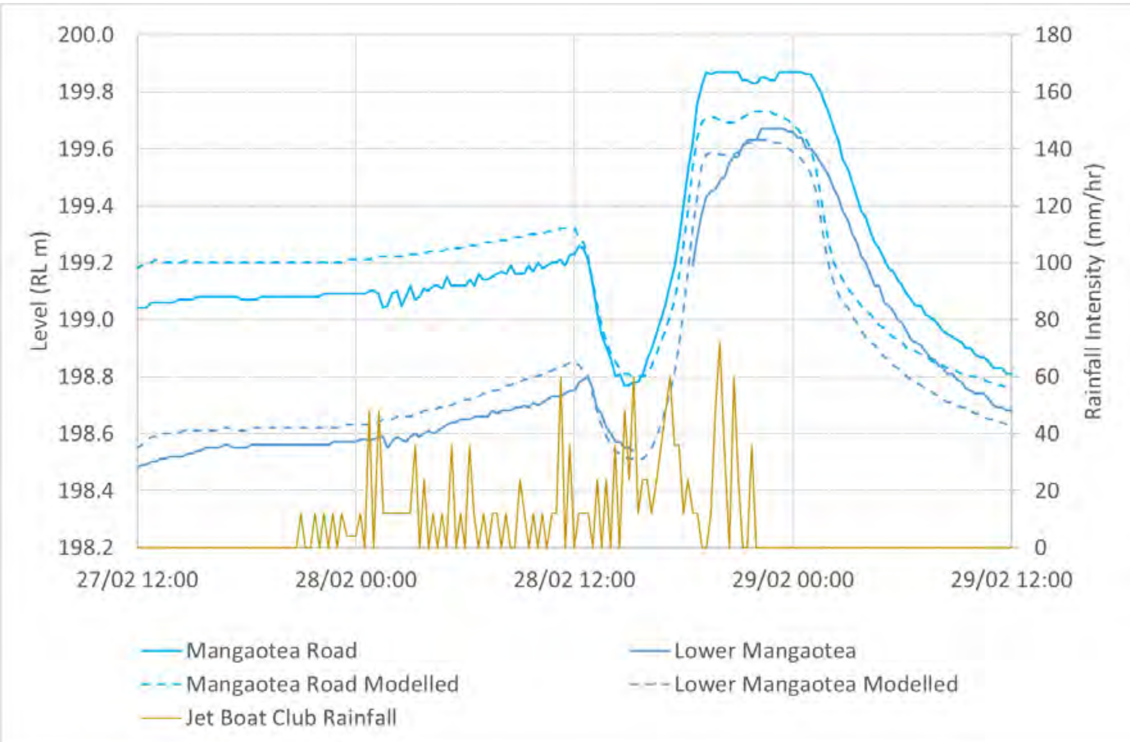
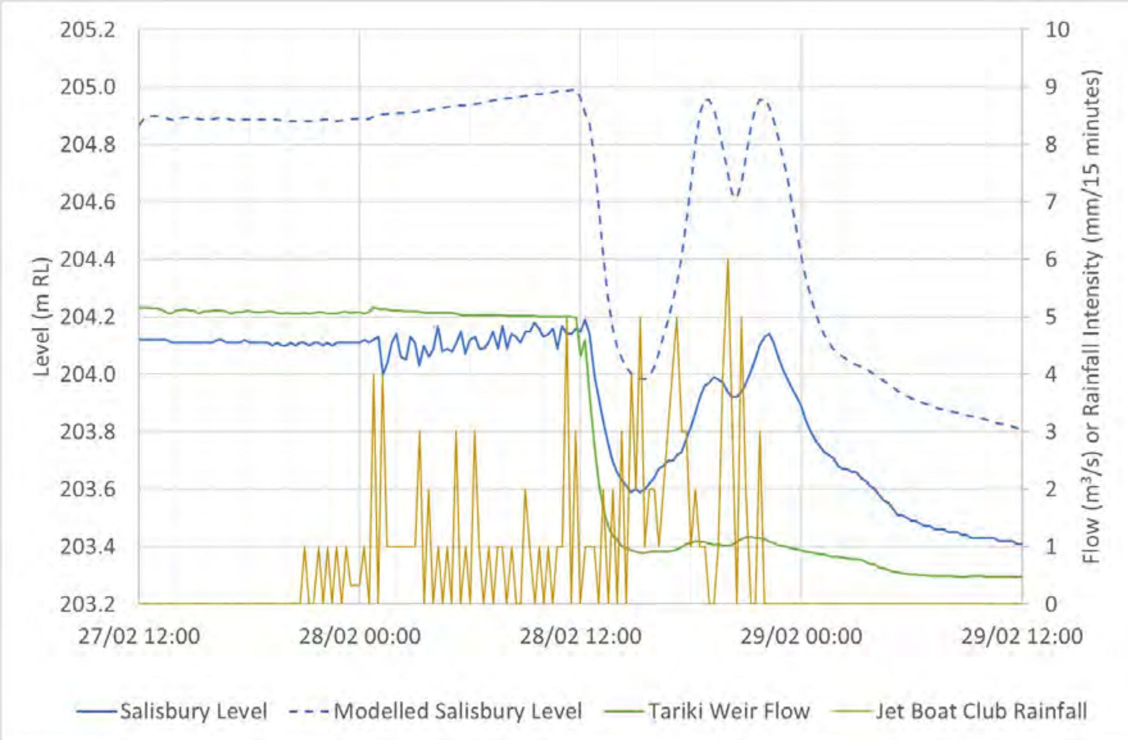


Figure 14: February 2004 – Modelled Upper Canal Levels



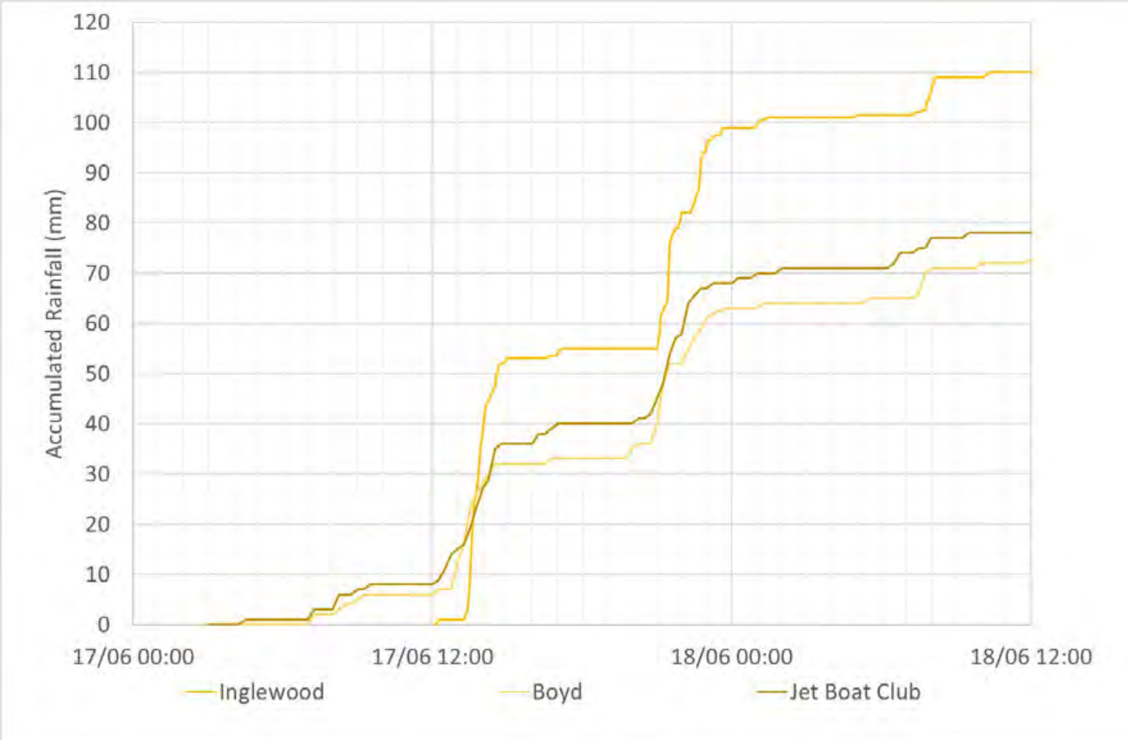
We note that the In-Race generator had yet to be constructed at the time of the February 2004 event. This may have some effect on the model results at the Salisbury gauge, as the construction of the In-Race generator would have increased the water level at the location of the Salisbury gauge.

2.0 June 2013

2.1 Recorded Rainfall

Figure 15 presents the recorded rainfall at the three gauges available at the time. The Jet Boat Club and Boyd gauges recorded similar rainfall patterns, and therefore we have some confidence in the accuracy of the recorded rainfall at these two gauges. The Inglewood gauge however recorded significantly more rainfall. We have discounted the Inglewood gauge for calibration as the in favour of one of the gauges located within the catchment. We consider that both the Boyd and Jet Boat Club data are both suitable for calibration. For the purposes of this assessment, we have selected to use the Boyd gauge.

Figure 15: June 2013 - Recorded Accumulated Rainfall

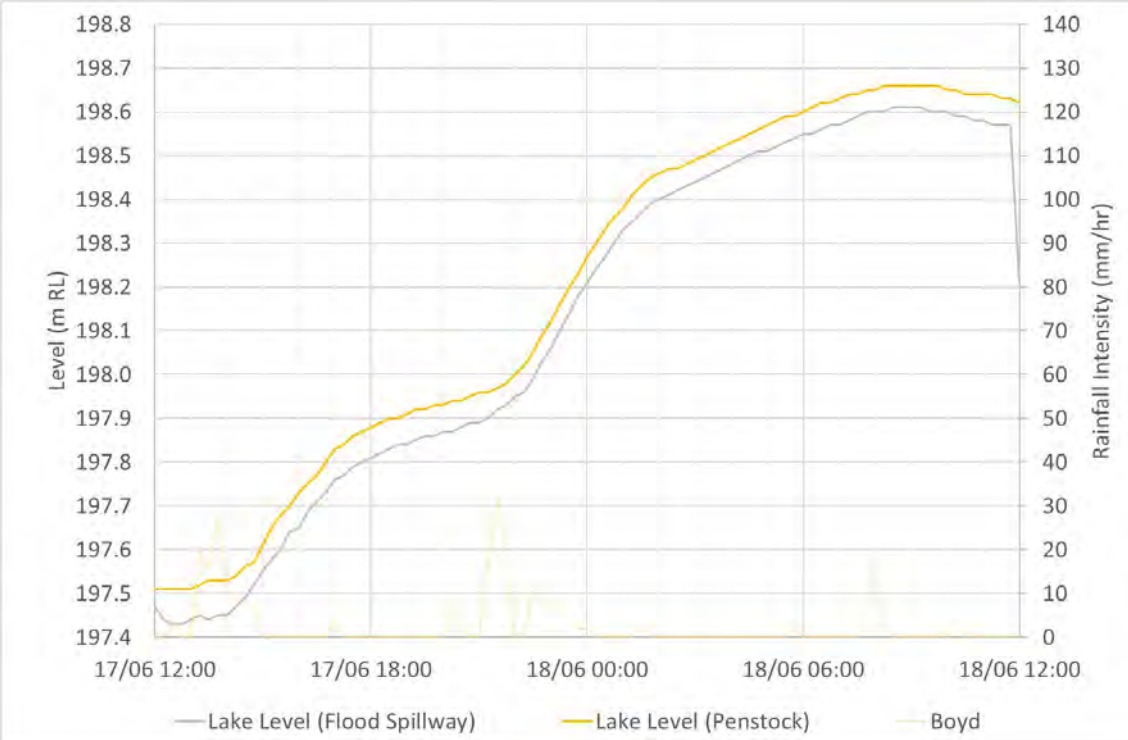


The majority of rainfall occurred across two distinct time periods, 12noon to 3pm (26mm), and 8pm to midnight (30mm) on 17 June.

2.2 Recorded Lake Levels

A peak lake level of RL 198.61m (flood spillway) was recorded at approximately 9am on 18 June. With a service spillway crest of RL 198.67m, we note the spillway did not operate. We note that the lake level continued to rise for approximately nine hours after the most significant rainfall ceased at approximately 11pm on 17 June. This is indicative of a slow release type catchment. There is also a significant change in the lake level gradient at approximately 2am on 18 June. This represents the commencement of the baseflow component of the flood event shown in Figure 16.

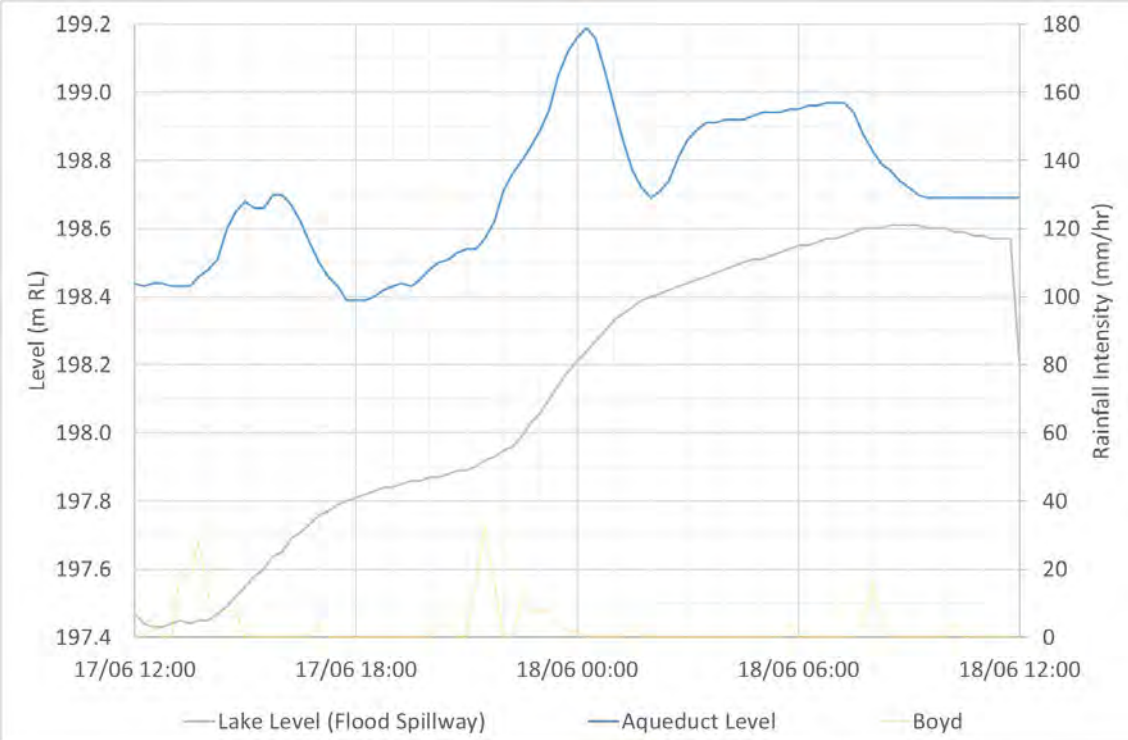
Figure 16: June 2013 - Recorded Lake Levels and Rainfall



2.3 Recorded Canal Levels

Figure 17 presents the recorded water levels within the lower canal. The Lower Mangaotea gauge appears to provide erroneous data during the event from approximately 2pm on 17 June and therefore has not been used within the calibration. The Aqueduct canal water level peaked at RL 199.19m (i.e., it appears the canal did not overtop with the aqueduct spillway gate crest at RL 199.72m).

Figure 17: June 2013 - Recorded Lower Canal Levels



2.4 Recorded Flows

The recorded flow hydrographs are presented within Figure 18.

Figure 18: June 2013 - Recorded Flow

2.5 Spillway Operation and Derived Inflows

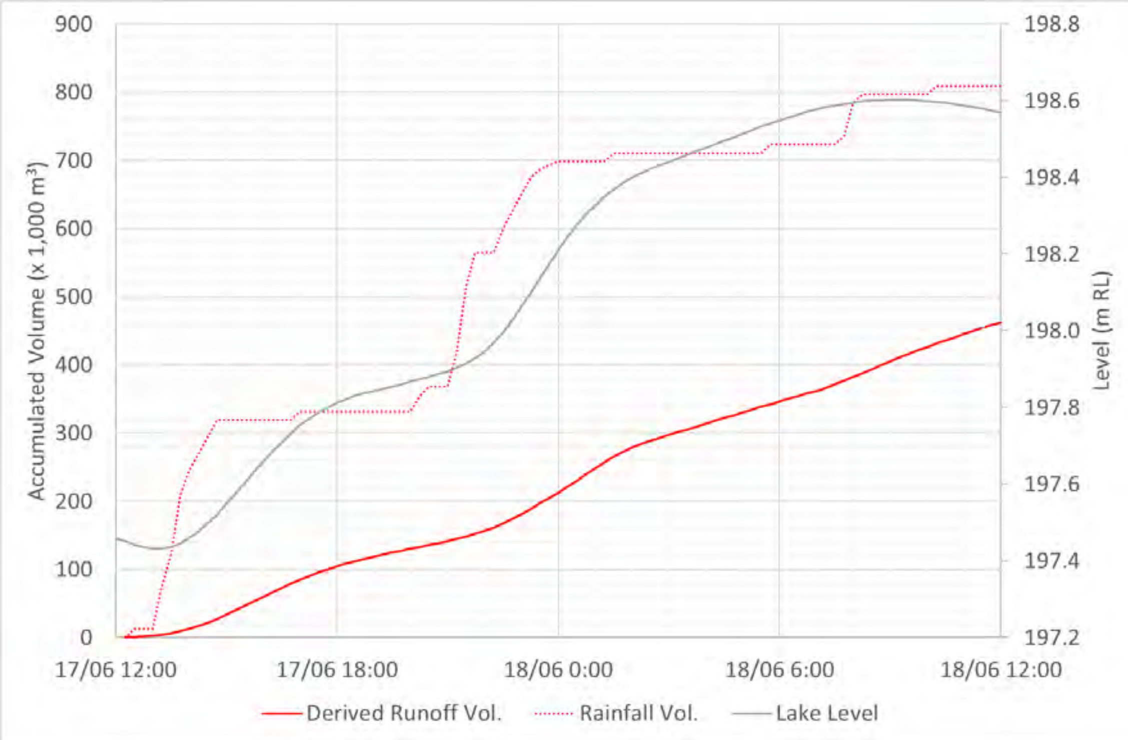
The spillways did not operate during the event.

Figure 19 presents the derived runoff inflow. The peak derived runoff inflow is approximately 10m^3/s. There are two distinct peaks to the flood event, with significant baseflow between the peaks, and after the second peak. The baseflows remain elevated after 3am on 18 June at around 5m^3/s. The baseflow prior to the event appears to be negligible.

Figure 19: June 2013 - Derived Runoff Inflow

Figure 20 presents the derived volumes.

Figure 20: June 2013 - Derived Runoff Volume



2.6 Calibration Parameters

Table 2 presents the selected calibration parameters.

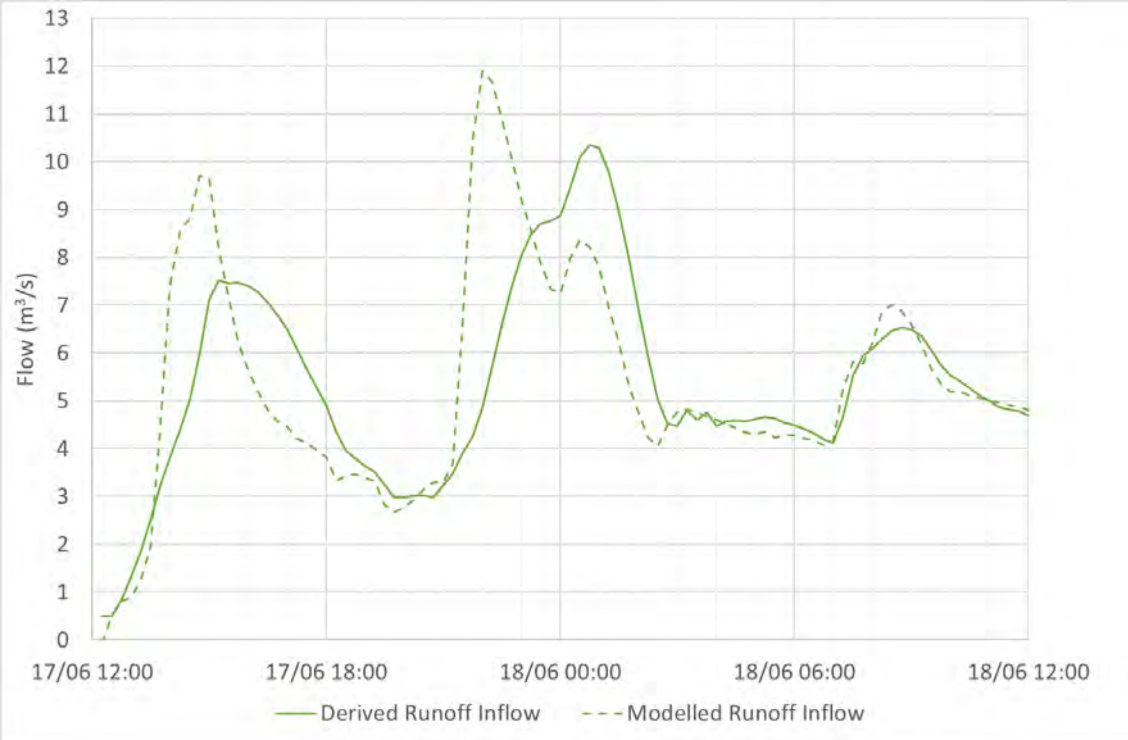
Table 2: Calibration Parameters

Catchment	Initial (mm)	Constant (mm/hr)	PRF	Lag Time (minutes)	Initial Discharge ($m^3/s/km^2$)	Baseflow Coefficient (hr)	Baseflow Fraction	Baseflow Steps
Salisbury	7	13	450	25	0.04	10	1.0	1
Mangaotea Road	7	13	450	15	0.04	10	1.0	1
Lower Mangaotea	7	13	450	70	0.04	40	1.0	1
Tunnel 3	7	13	450	25	0.04	10	1.0	1
Upper Lake	7	13	450	25	0.04	10	1.0	1
Lower Lake	7	13	450	25	0.04	10	1.0	1
Tributary 1	7	13	450	75	0.04	40	1.0	1
Tributary 2	7	13	450	35	0.04	20	1.0	1
Mangaotea	7	13	450	120	0.04	60	1.0	1

2.7 Results and Discussion

Figures 21 and 22 present the results of the flood event calibration, using the variable input parameters summarised within Table 2.

Figure 21: June 2013 – Modelled Runoff Inflow



The model results at the aqueduct and the lower lake are presented within Figure 22. A good match with recorded water level was obtained, particularly at higher lake levels. We consider that further survey of the canal is required to obtain an improved match at lower lake levels (noting that the model provides conservative results). We note that the timing of the peaks could be improved with further calibration, however we do not consider such works is critical to the assessment.

Figure 22: June 2013 – Modelled Levels

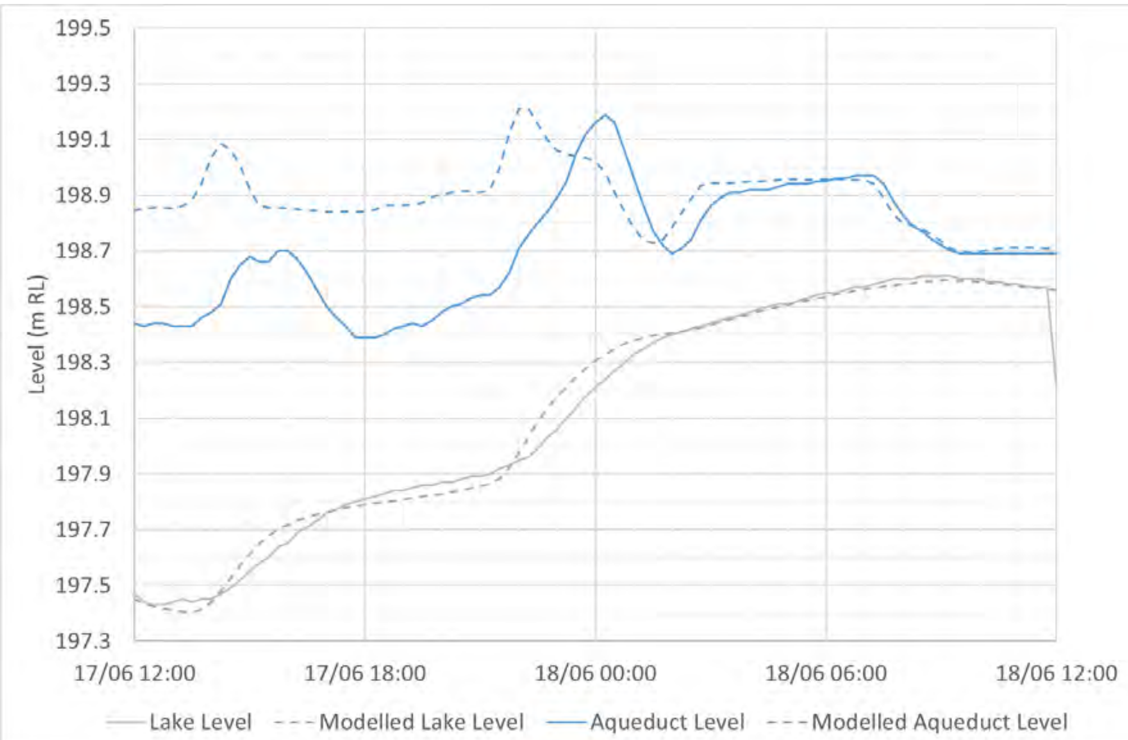
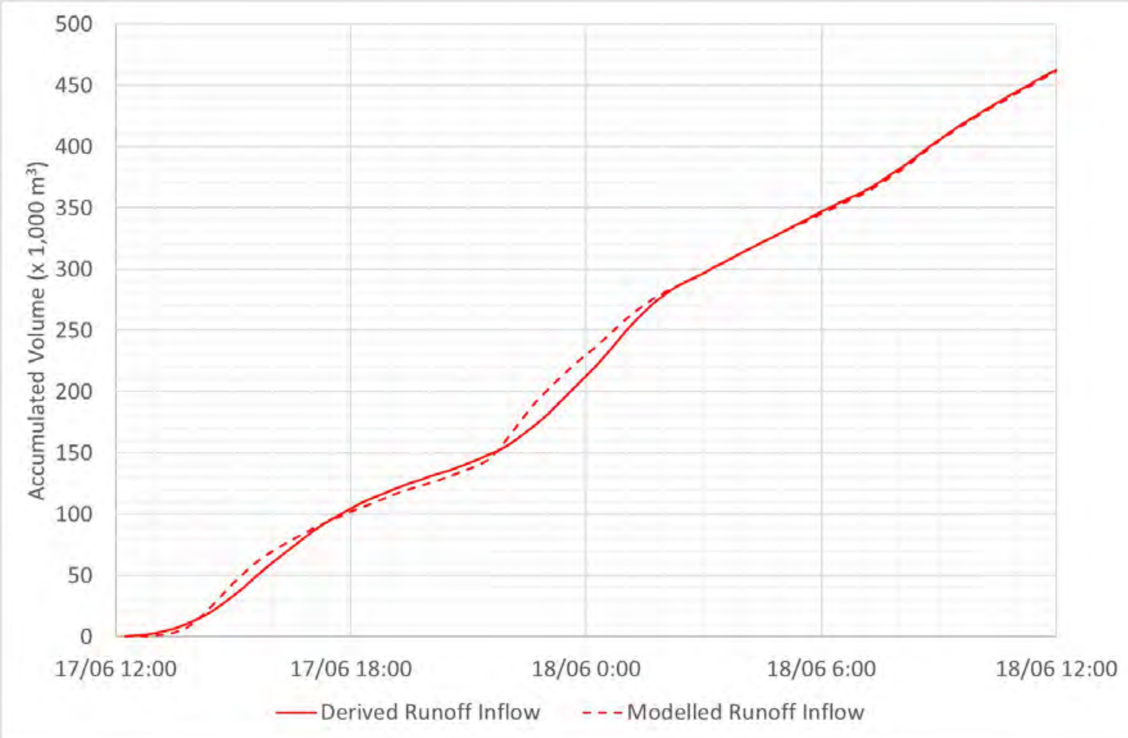


Figure 23 presents the modelled lake volume. We consider a good match has been obtained.

Figure 23: June 2013 – Modelled Lake Volume



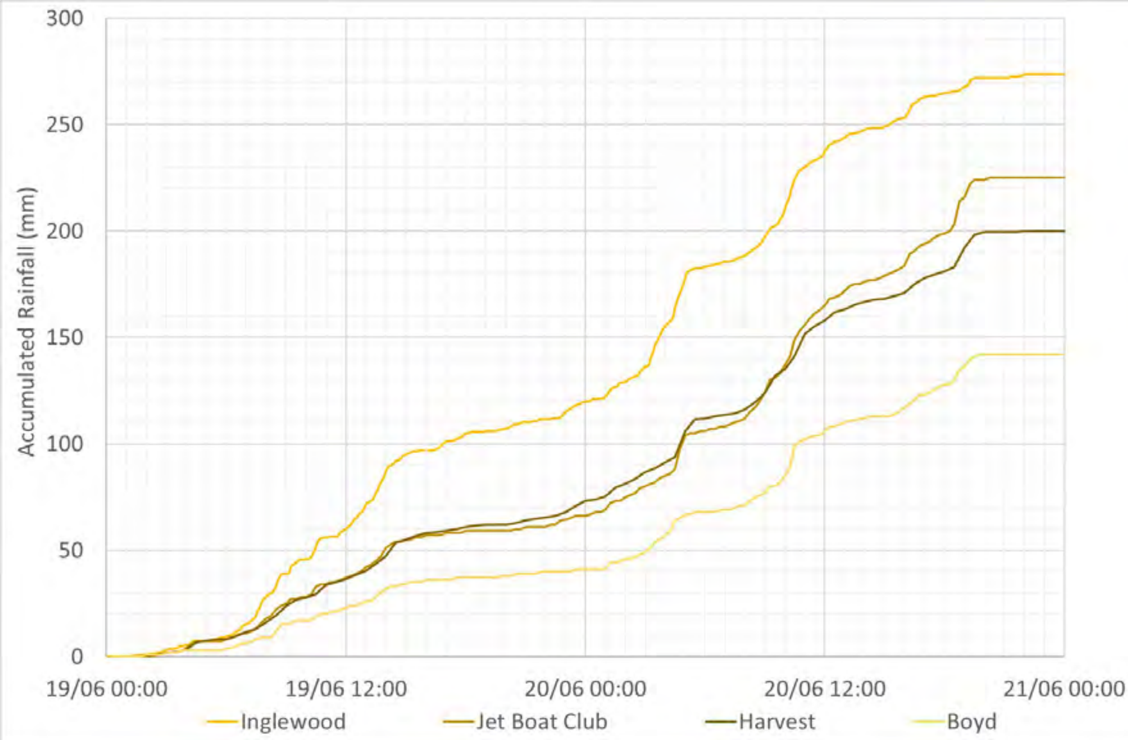
3.0 June 2015

3.1 Recorded Rainfall

The 2015 event was a long duration event, with rainfall commencing soon after midnight on 19 June and continuing through into the evening of 20 June (i.e., essentially a 48-hour event). Figure 24 presents the observed rainfall information during the event, from the relevant and available rainfall gauges.

As discussed within Appendix E, we have opted to use the Jet Boat Club gauge for calibration purposes as we consider it is most likely to be representative of the rainfall within the catchment during the event. We note that the Boyd gauge appears to have under recorded rainfall as it is significantly lower than both the Jet Boat Club and Harvest gauges.

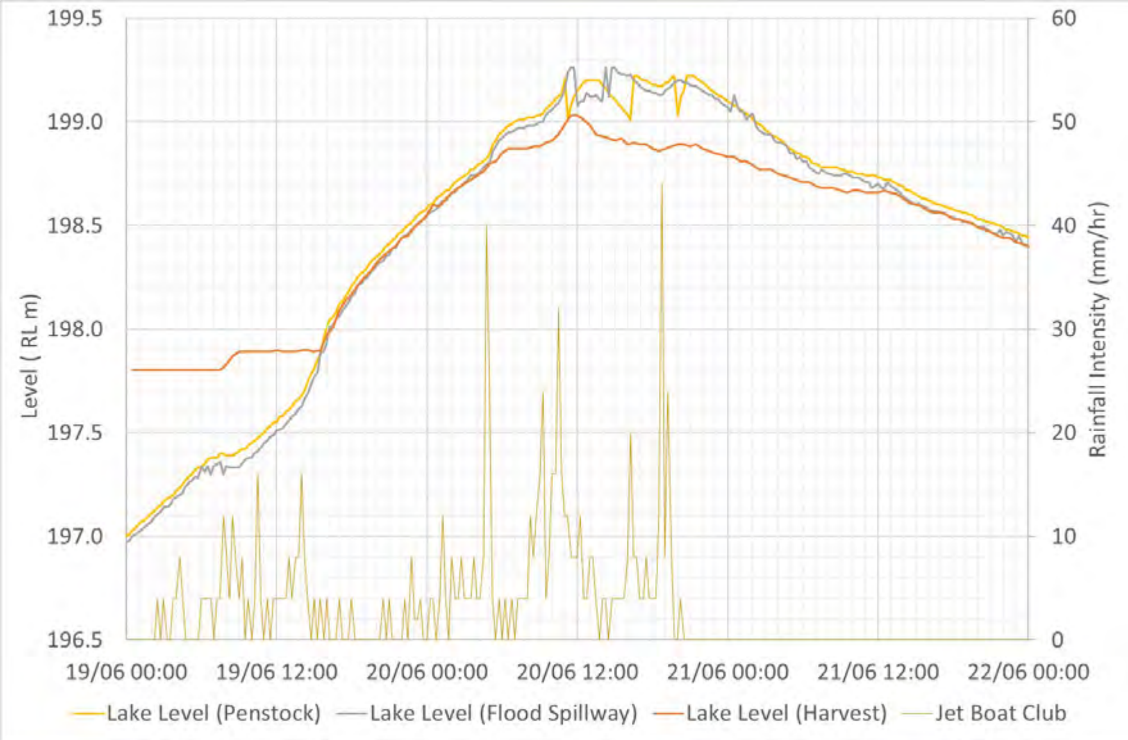
Figure 24: June 2015 - Recorded Accumulated Rainfall



3.2 Recorded Lake Levels

Recorded lake level information is presented within Figure 25. Some anomalies are evident near the flood peak. The flood spillway instrument MTK_H13LT_007 appears to have recorded erroneous data in the period from midday through to approximately 3pm. All three lake level instruments are potentiality affected by drawdown as they are located in close proximity to spillways or offtakes. We note that the three records are in reasonable agreement prior to any spilling (prior to 6am 20 June), with the penstock water levels typically reading 40mm higher than the flood spillway water level.

Figure 25: June 2015 – Recorded Lake Levels and Rainfall



3.3 Recorded Canal Levels

Table 3 and Figure 26 summarise the canal water level information recorded during the event.

Table 3: June 2015 - Recorded Canal Water Levels

Location	Peak Recorded WL (m RL)	Comment
Sediment Pond	N/A	Not installed at this date.
Tariki Weir	209.75	Data not relevant as gates closed during flood peak.
Salisbury Road Bridge	204.11	Data quality appears adequate (some erroneous)
Mangaotea Road Culvert	N/A	No data 2013 to 2016.
Mangaotea Aqueduct	199.88	Data quality appears adequate.
Lower Mangaotea	199.84	Data quality appears adequate.

Figure 26 demonstrates that the lower canal water levels peaked at around 12 noon on 20 June 2015. The recorded peak water level at the aqueduct was RL 199.88m, some 160mm higher than the aqueduct spillway gate crest of RL 199.72m (i.e., the canal overtopped). Photo 1 provides further evidence of such overtopping.

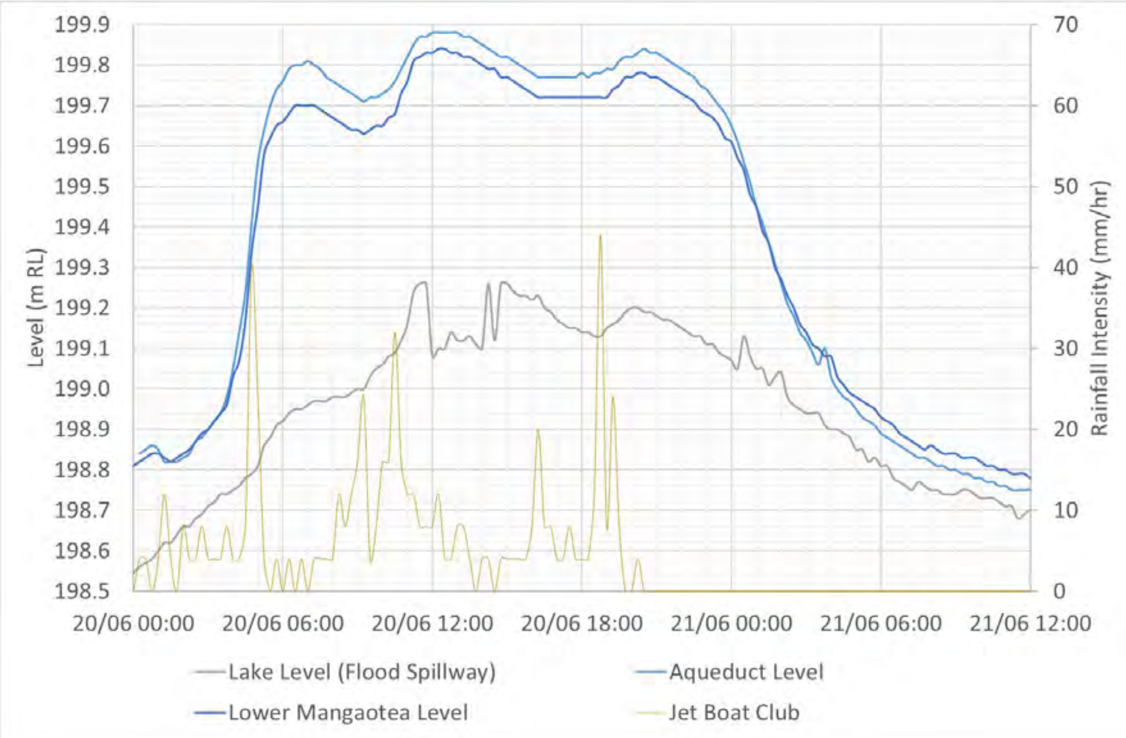
The elevated canal water levels (i.e., elevated well above Lower Lake levels) for the approximate 16-hour period from 4am 20 June through to 12am 21 June, indicate that the Mangaotea Stream overtopped into the canal during this period. Once overtopping commenced the canal water levels rose rapidly (up to 400mm per hour). Likewise, once overtopping ceased at around midnight, the canal water levels also decreased reasonably rapidly (approximately 150mm per hour). By 4am 21 June, the canal water levels were back to approximately 100-200mm above Lower Lake levels, as under typical normal operating conditions.

The canal water level data indicates that the canal overtops its true left bank when water levels reach approximately RL 199.6m. This can be seen in Figure 26 where the water level gradient on the rising limb starts to flatten, and on the falling limb, where the steep (and steady) gradient commences. The canal appears to have overtopped (spilt) from 5am 20 June (i.e., one hour after overtopping into the canal commenced) through to 12am 21 June.

The falling limb from 12am through to 4am 21 June is relatively steep. This indicates that once overtopping into the canal ceased, the stored volume within the canal and surrounding areas draining to Tunnel 2 was relatively small. This suggests that the Mangaotea Stream culvert (Tariki Road South Road embankment) did not provide stored volume which drained to Tunnel 2.

The two sets of canal water levels gauge data match well over the calibration period. The Lower Mangaotea data is generally lower than the Aqueduct data, which we would expect if the canal were conveying water further downstream. We note that the Lower Mangaotea data is higher than the Aqueduct data after the peak of the event has occurred. We have assumed that this is a data accuracy issue.

Figure 26: June 2015 - Recorded Lower Canal Levels



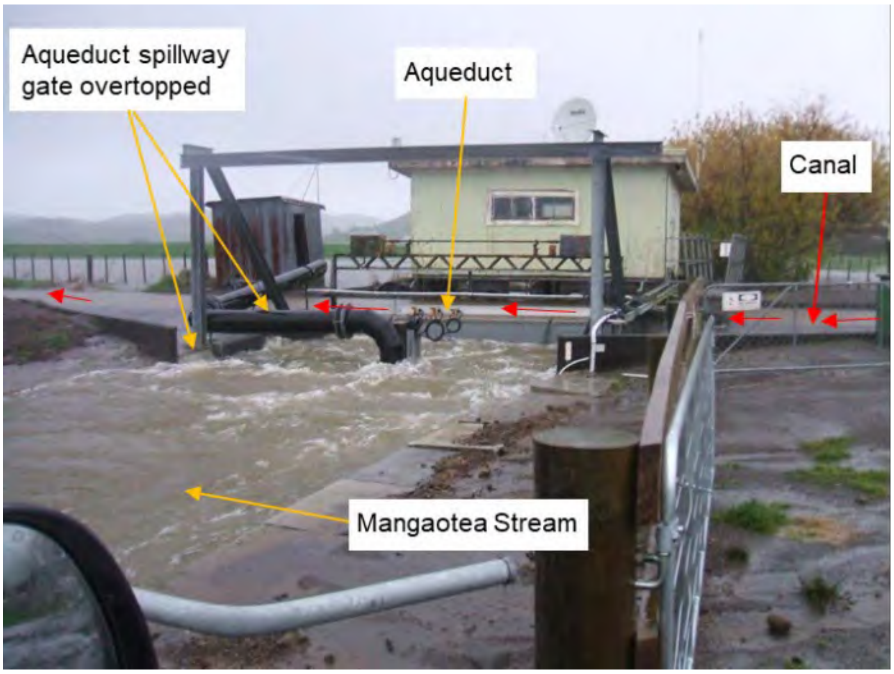
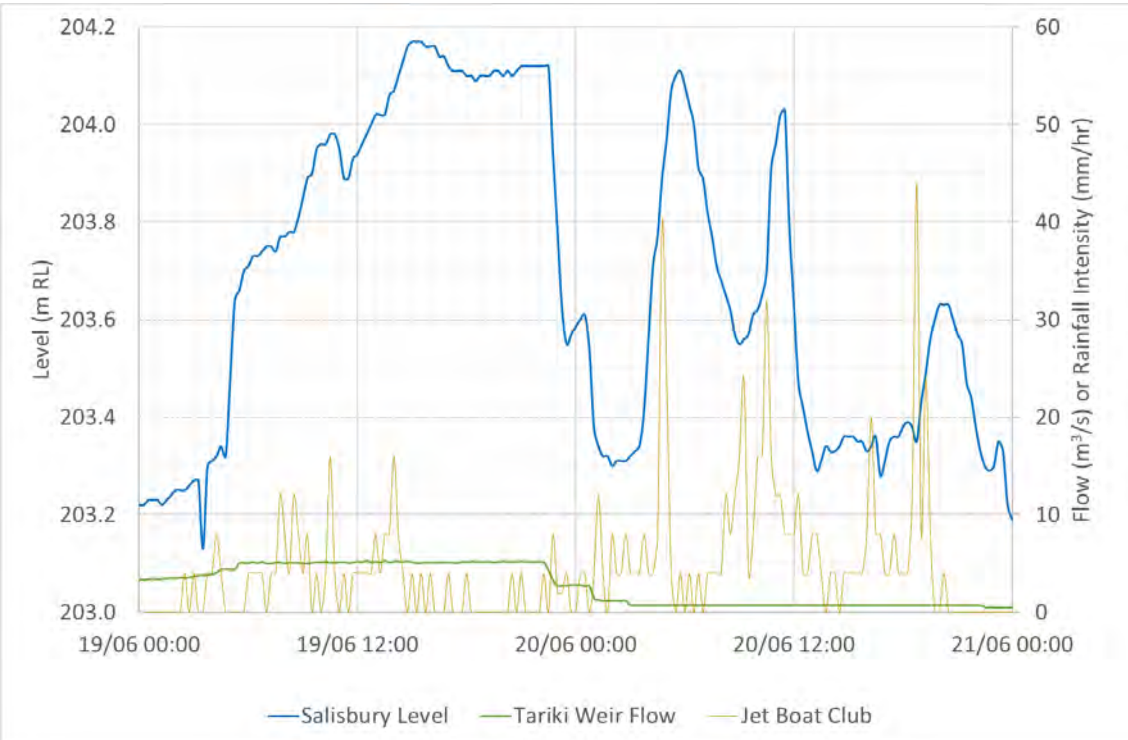


Photo 1: Flow spilling out of canal into Mangaotea Stream (June 2015 flood event)

Figure 27 presents the recorded level information from the Salisbury gauge. The figure demonstrates that the peak water levels at the Salisbury gauge occurred while flow take from the Manganui River was still occurring (i.e., while the Tariki Weir flow was approximately $5m^3/s$).

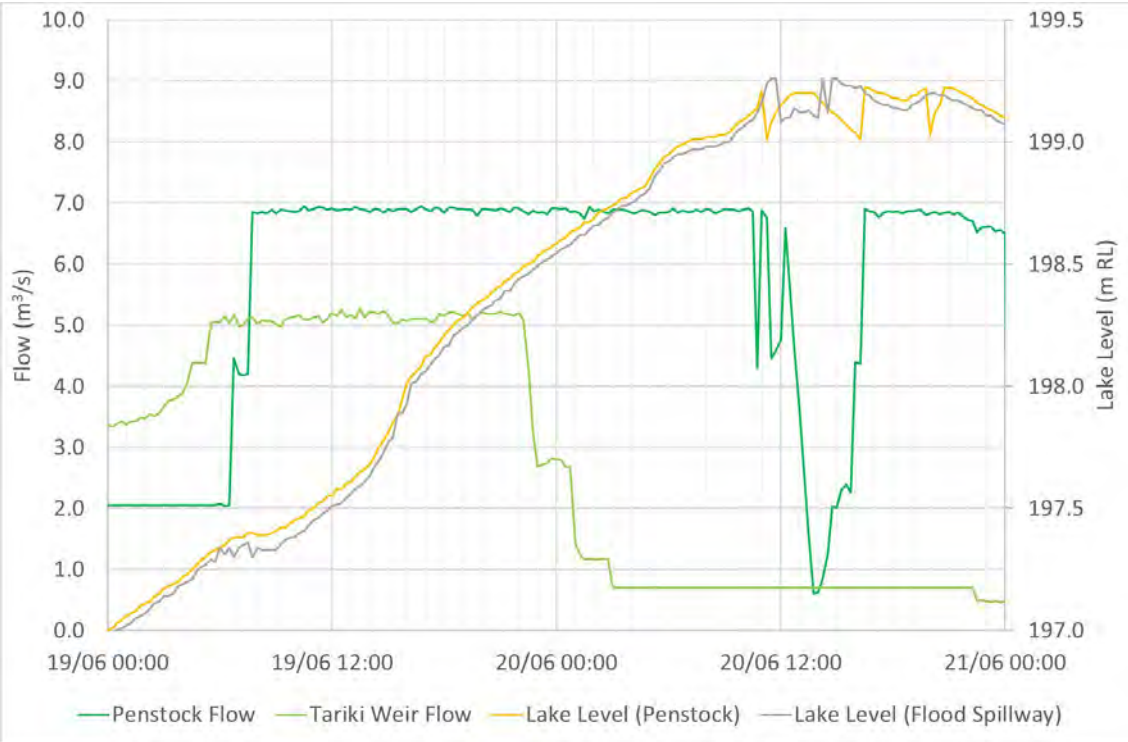
Figure 27: June 2015 - Recorded Upper Canal Levels



3.4 Recorded Flows

The recorded flow hydrographs are presented within Figure 28. We note that the recorded lake level by the penstock instrument MTK_H13LT_007 is likely to be erroneous between 11am and 4.20pm due to the penstock intake gate operation. The change in lake level gradient at approximately 6am on 19 June is due to the increase in the penstock flow.

Figure 28: June 2015 - Recorded Flow

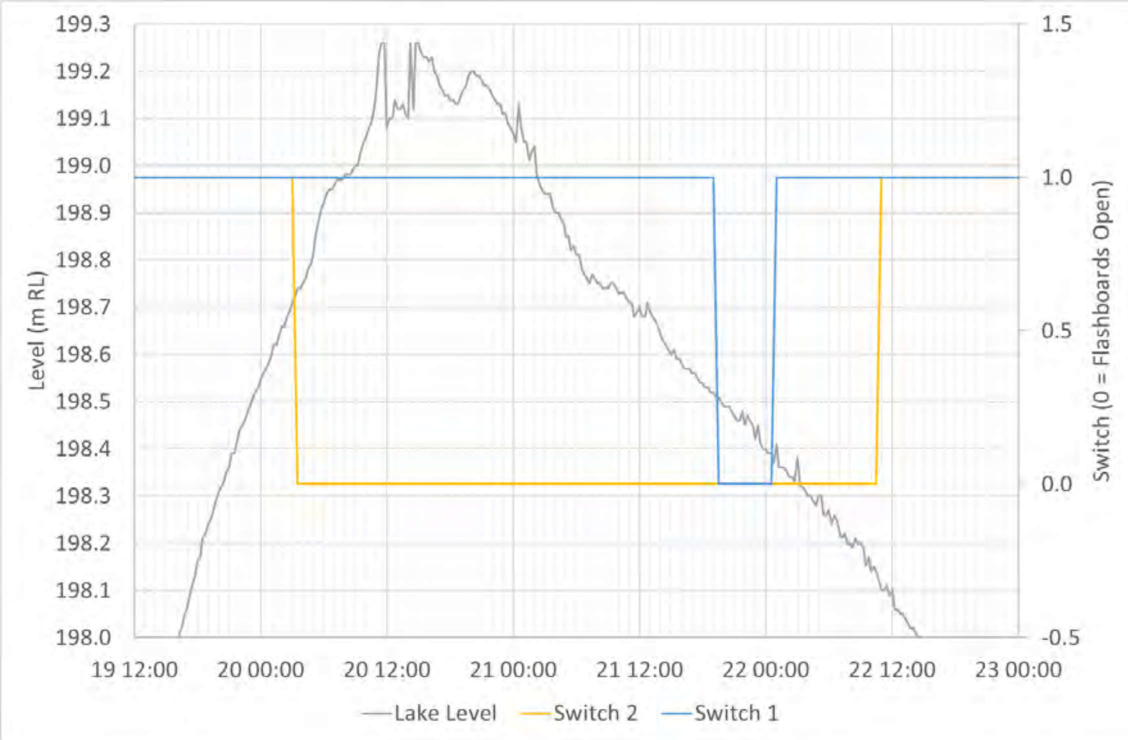


3.5 Spillway Operation and Derived Inflow

Service Spillway

The harvest site recorded the tipping of the flashboards, as presented within Figure 29. The harvest data indicates that not all boards were tipped during the peak of the event. We also understand that although the switches may indicate that the flashboard has tipped, the flashboard may physically not have fully tipped.

Figure 29: June 2015 - Recorded Flashboard Operation



It appears that the harvest unit lake level was only influenced by a drawdown affect once the lake level reached RL 198.80m.

We consider there is uncertainty with the recorded lake levels at the time of the peak (i.e., as the recorded lake level fluctuations appear erroneous) and with the service spillway performance. For the purposes of this assessment, we have assumed:

- Flashboard crest at RL 198.67m. Five flashboards fully activated when the lake level reached RL 198.70m

Flood Spillway

A maximum lake level of RL 199.26m was recorded by the flood spillway instrument (MTK_H13LT_007) (i.e., 40mm over the design fuse plug crest level). The lake level information indicates that the flood spillway was initially overtopped between 11am and 11.15am on 20 June. We consider it is likely that the fuse plug fused/breached between 11.45am and midday, prior to the recorded instantaneous 150mm fall in lake level.

Mr Grant Hurlstone (Trustpower Limited Contractor) has confirmed that at around midday he walked through flow within the flood spillway channel with relative ease. Approximately half an hour later when he returned to pass through the channel, the flow was much more difficult to walk through with a flow depth of approximately 350mm. It is, therefore, likely that the fuse plug fused/breached during Mr Hurlstone's visit to the site.

Mr Hurlstone returned to site at approximately 5pm, and at that time he observed that the flood spillway was not operating. The lake level at this time was approximately RL 199.15m. We note that the design flood spillway crest (downstream of the fuse plug) elevation is RL 198.85m.

Vaughan Martin (RILEY) inspected the site on 7 July 2015. The fuse plug was observed to have only fused on the true left, although it was evident that the fuse plug had been overtopped along its entire length (rilling erosion was noted on the downstream side of the plug).

Photo 2: Photograph taken on 7 July 2015 (after June 2015 flood event) Note: Only the true left-hand-side of the fuse plug has fused, although scour is evident on the downstream face of the unfused plug. Lake level instrument MTK_H13LT_007 is located on the face of the gabion mattress.

We also noted that the gabion mattress on the upstream face became the controlling structure after the fuse plug had fused, across the part width of the spillway. The level of the gabion mattress explains why the flood spillway was observed to have stopped spilling, at approximately RL 199.15m.

Overall, the assumptions for the flood spillway operation are not critical to the calibration as the flood spillway flows are small compared to the service spillway flows. We have assumed the fuse plug breached to an elevation of RL 199.15m over a 5m width (as measured on-site).

Derived Data

The derived lake level and spillway hydrographs are presented within Figure 30. Figure 31 presents the derived runoff inflow.

Figure 30: June 2015 – Derived Spillway Flow

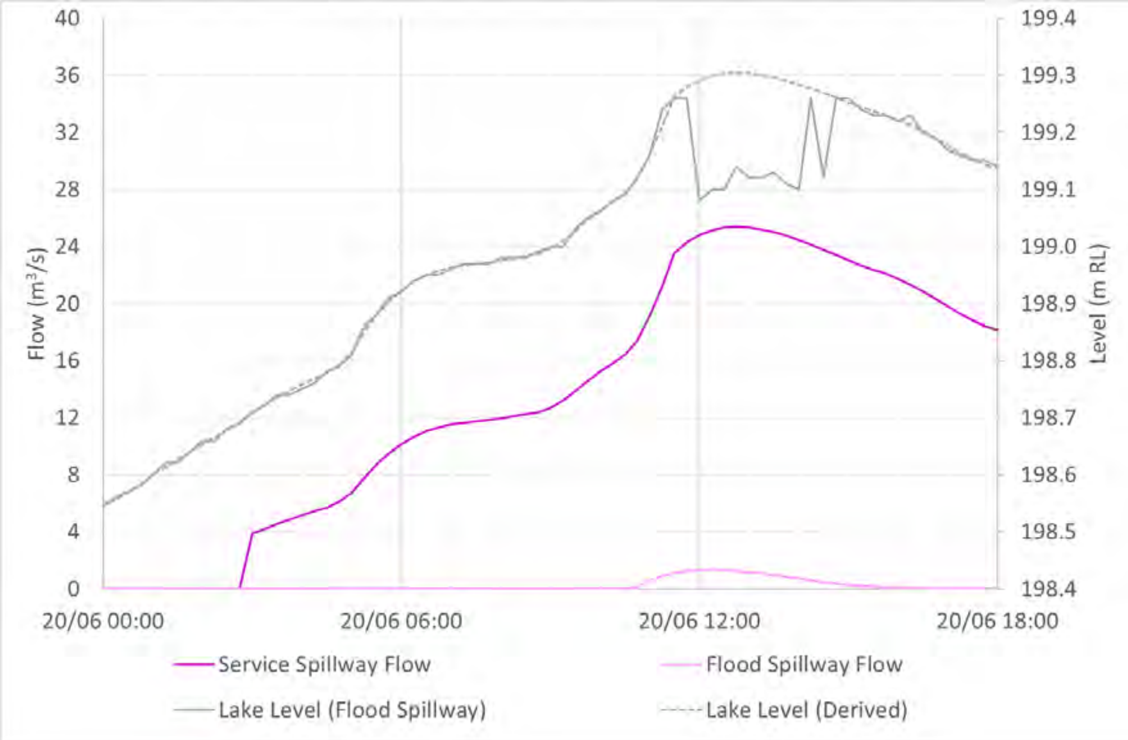


Figure 31: June 2015 - Derived Runoff Inflow

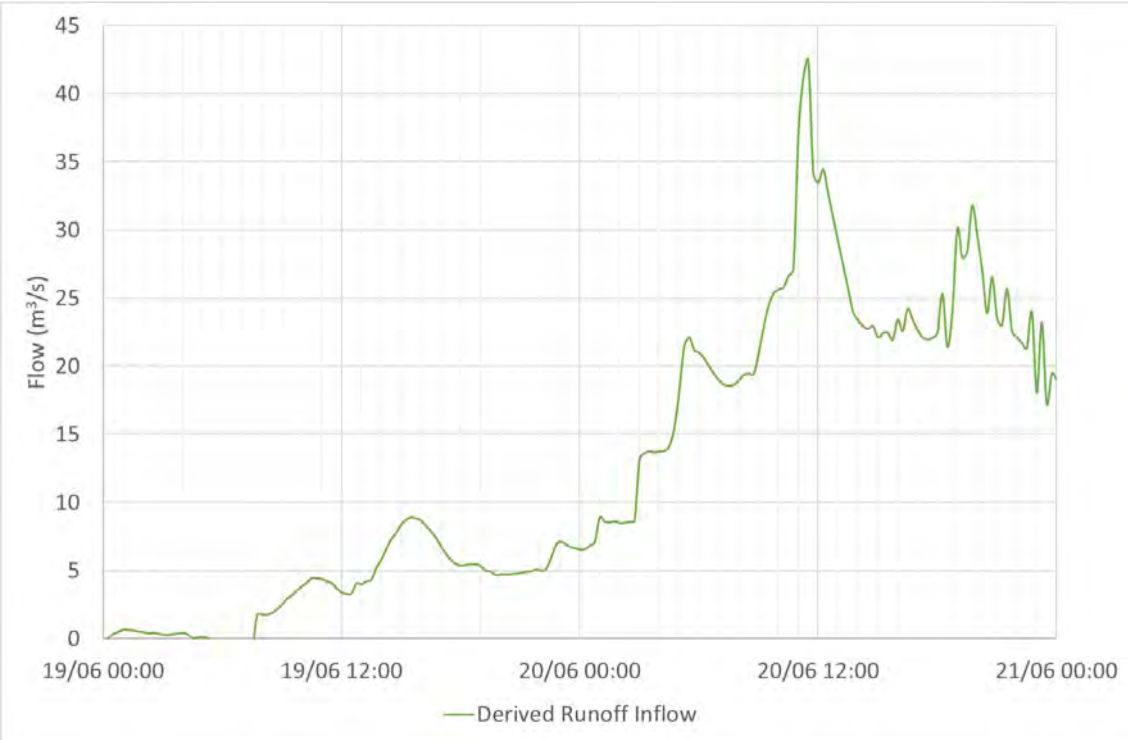
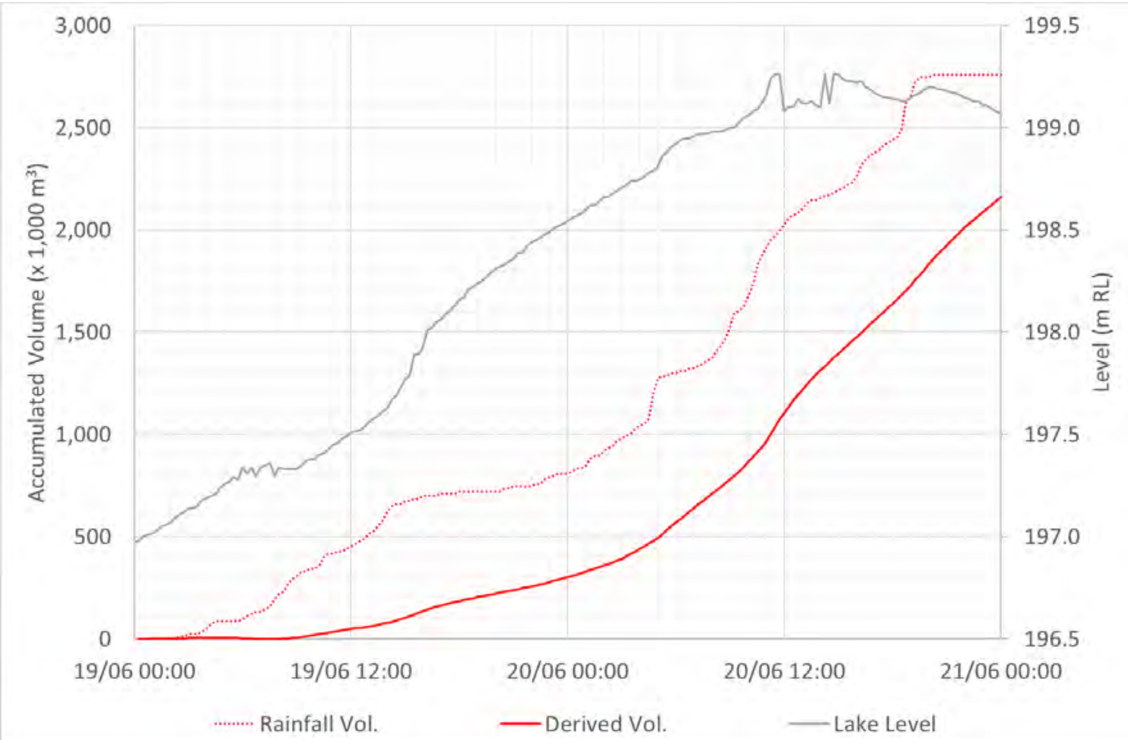


Figure 32 presents the derived runoff volume for the lower lake (i.e., derived from recorded lake levels, recorded flows, and the storage elevation relationship). For comparison purposes, the figure also presents the rainfall volume based on a catchment area of 12.26km^2 (i.e., excluding the Mangaotea catchment). The total rainfall volume from the event was approximately 2,800,000m^3 (assuming the recorded rainfall at the Jet Boat Club gauge occurred uniformly over the catchment area). In comparison, a total runoff volume of approximately 2,200,000m^3 reached the lower lake over the 48-hour period, with the majority of the volume reaching the lake in the latter half of the event. We consider this comparison provides further confidence that the Jet Boat Club rain gauge provides suitable data for calibration purposes.

Figure 32: June 2015 - Derived Volumes



3.6 Calibration Parameters

We have used the variable input parameters as summarised within Table 4.

Table 4: June 2015 Calibration Parameters

Catchment	Initial (mm)	Constant (mm/hr)	PRF	Lag Time (minutes)	Baseflow Initial (m^3/s)	Baseflow Coefficient (hr)	Baseflow Fraction	Baseflow Steps
Salisbury	60	3	450	25	0	10	1.0	1
Mangaotea Road	60	3	450	15	0	10	1.0	1
Lower Mangaotea	60	3	450	70	0	40	1.0	1
Tunnel 3	60	3	450	25	0	10	1.0	1
Upper Lake	60	3	450	25	0	10	1.0	1
Lower Lake	60	3	450	25	0	10	1.0	1
Tributary 1	60	3	450	75	0	40	1.0	1
Tributary 2	60	3	450	35	0	20	1.0	1
Mangaotea	60	3	450	120	0	60	1.0	1

3.7 Results and Discussion

Figures 33 and 34 present the results of the calibration at the dam. The modelled volume compares favourably to the derived volume.

Figure 33: June 2015 - Modelled Volumes and Lower Lake Levels

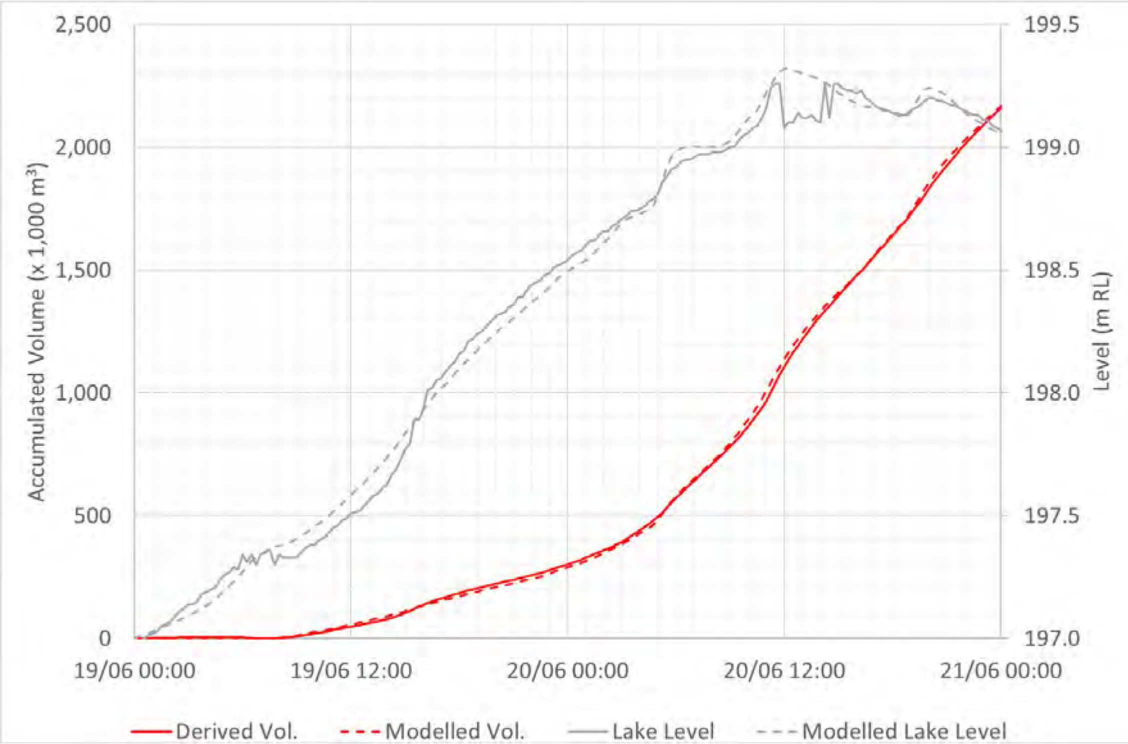
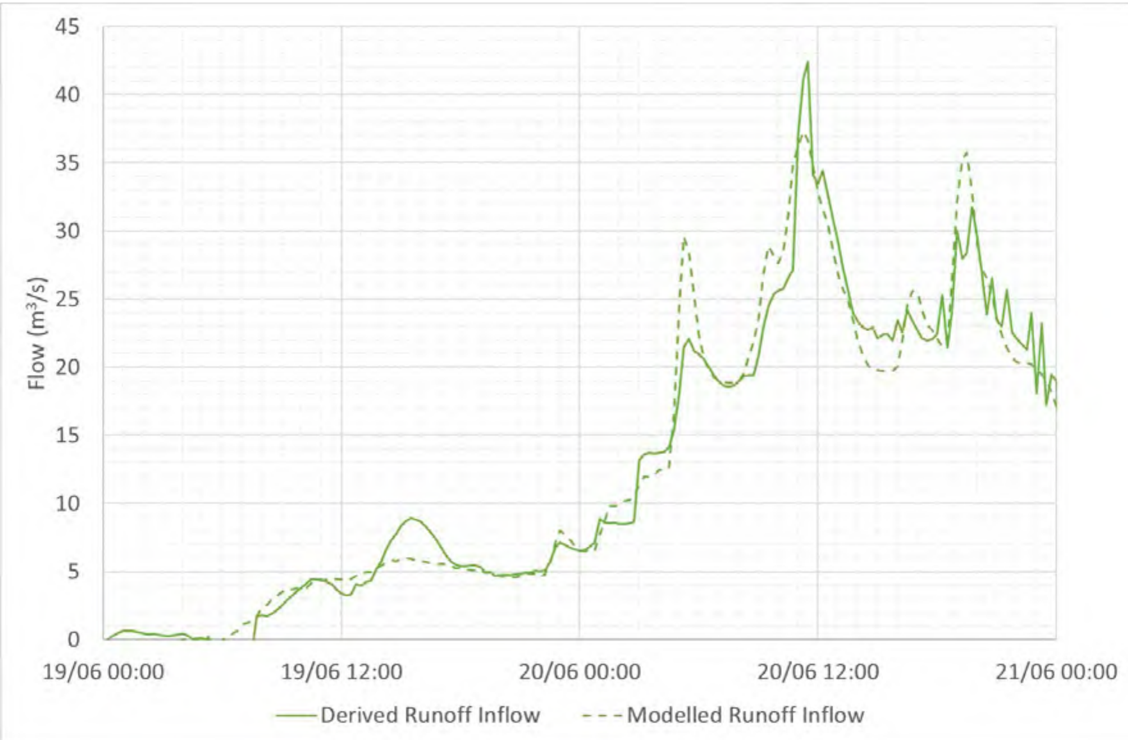


Figure 34: June 2015 - Modelled Runoff Inflow



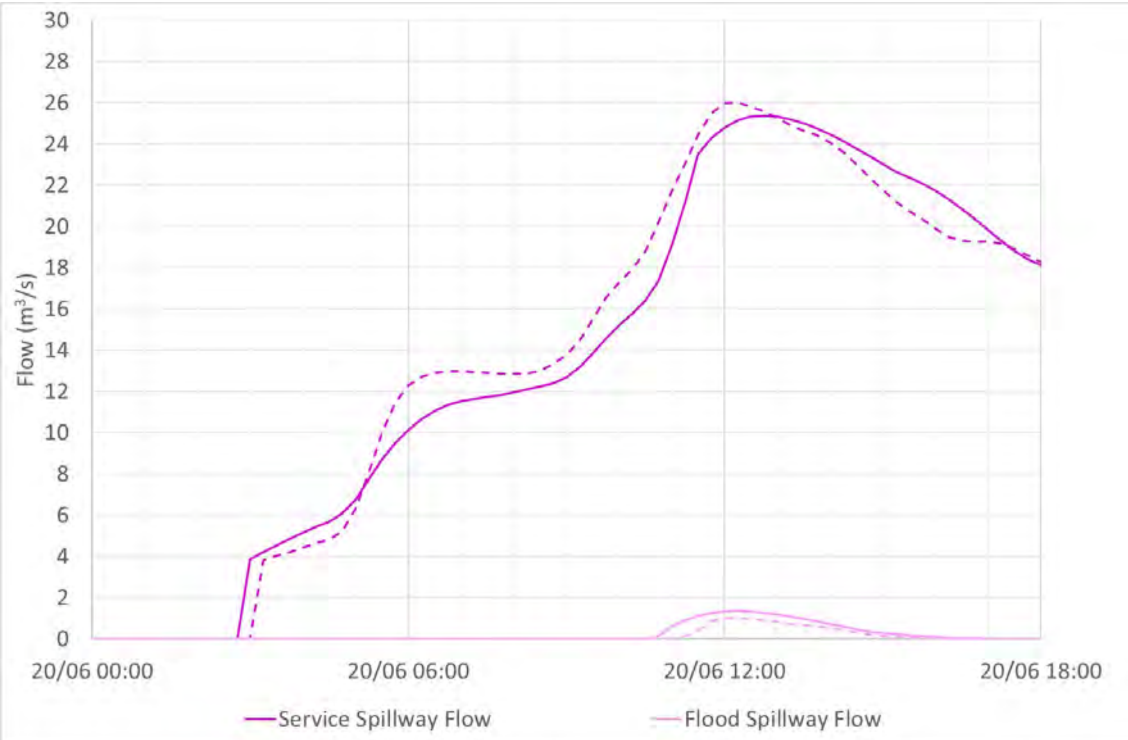
We consider that the 2015 peak derived runoff inflow may be overestimated (between 11am and 11.45am), with derived runoff inflows of over 40m^3/s. We consider there are two possible reasons for this apparent discrepancy.

Firstly, there is uncertainty with the recorded lake levels, spillway operation, and penstock operation at the peak of the event. We note that the peak derived runoff inflow is from the period 11am to 11.15am on 20 June (with a recorded lake level increase of 80mm over this period). Given that the fuse plug was overtopped at about this time, and that the penstock flow was also fluctuating at this time (possibly causing surges in lake levels), we consider that there is uncertainty with the recorded lake levels and therefore the peak derived inflow.

Secondly, blockage of the two tributary culverts by catchment debris cannot be discounted. Blockage of these culverts during the event would affect the peak inflows. We understand that the Tributary 1 culvert and embankment was damaged during the event and subsequently repaired by the landowner. We also understand through discussions with the landowner that the Tributary 2 culvert embankment was overtopped during the event. The culvert inlet was covered by vegetation growth at the time of a recent inspection.

Figure 35 presents the modelled spillway flows compared to the derived flows. A good match has been obtained.

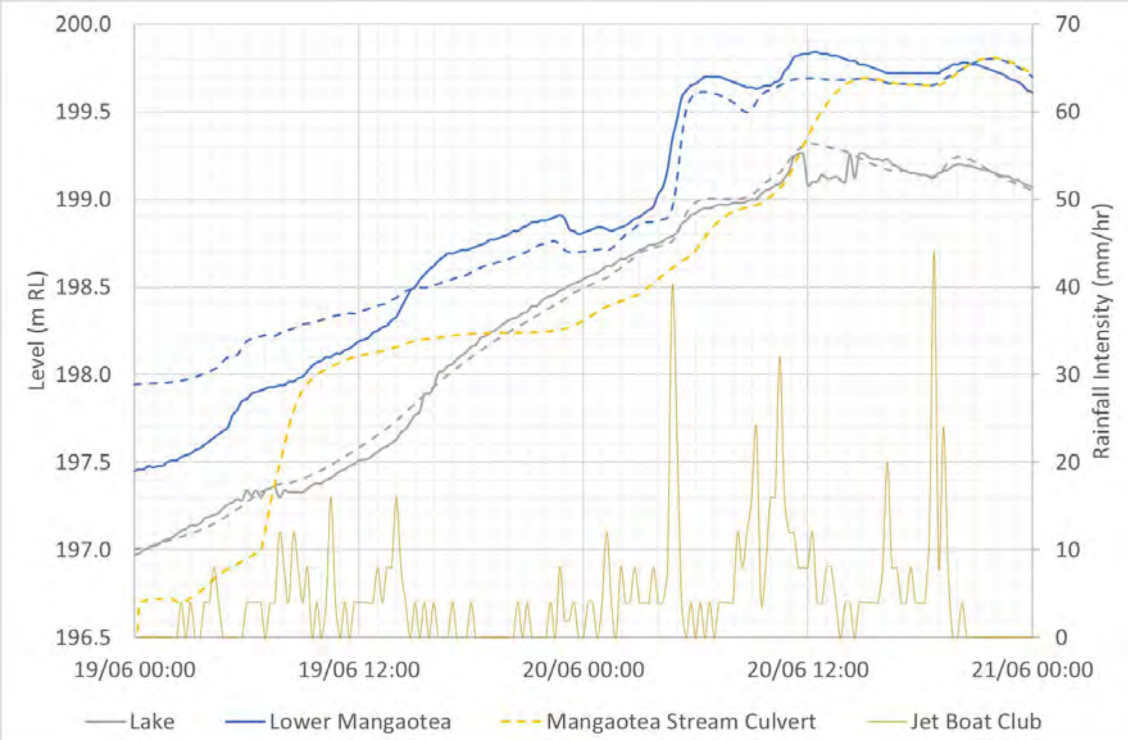
Figure 35: June 2015 - Modelled Spillway Flow



Note: Modelled results are dashed.

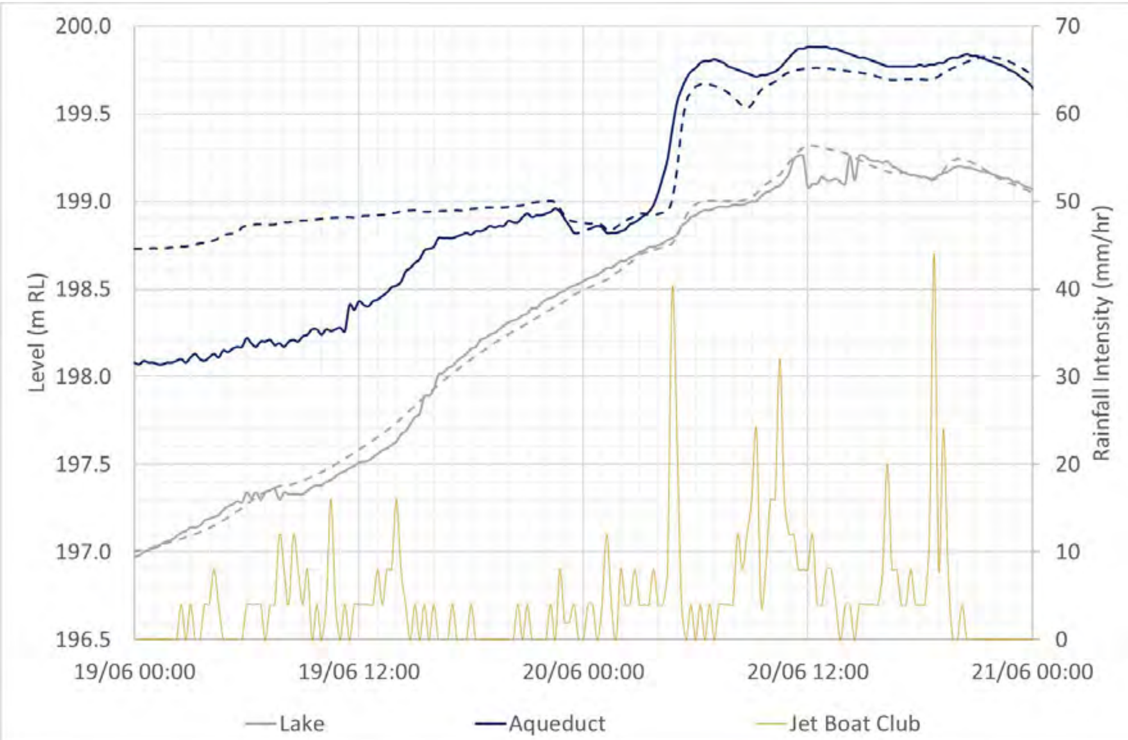
Figures 36 and 37 present the modelled results within the lower canal. There is a reasonable match with peak levels (within 200mm). Near the end of the event, the model results indicate that the Mangaotea Stream culvert becomes the water level control at Lower Mangaotea (i.e., the two water level results merge at around 2pm on 20 June). We note that the culvert parameters have been assumed. We recommend that the flood maps are reviewed by Chris England (Trustpower) who observed the 2015 event to ensure the mapped flood event matches observed flooding across Mangaotea Road)

Figure 36: June 2015 – Lower Mangaotea Canal Levels



Note: Modelled results are dashed.

Figure 37: June 2015 – Aqueduct Canal Levels



Note: Modelled results are dashed.

Figure 38 demonstrates that a peak flow of approximately $10m^3/s$ occurred through Tunnel 2 (Lower Mangaotea), with significant overflows/spill out of the canal (peak spill flow of $26m^3/s$). The majority of the spill is due to the Mangaotea Stream overtopping into the canal, although some spill flow is also from the local canal catchment.

Figure 38: June 2015 – Modelled Lower Canal Flows

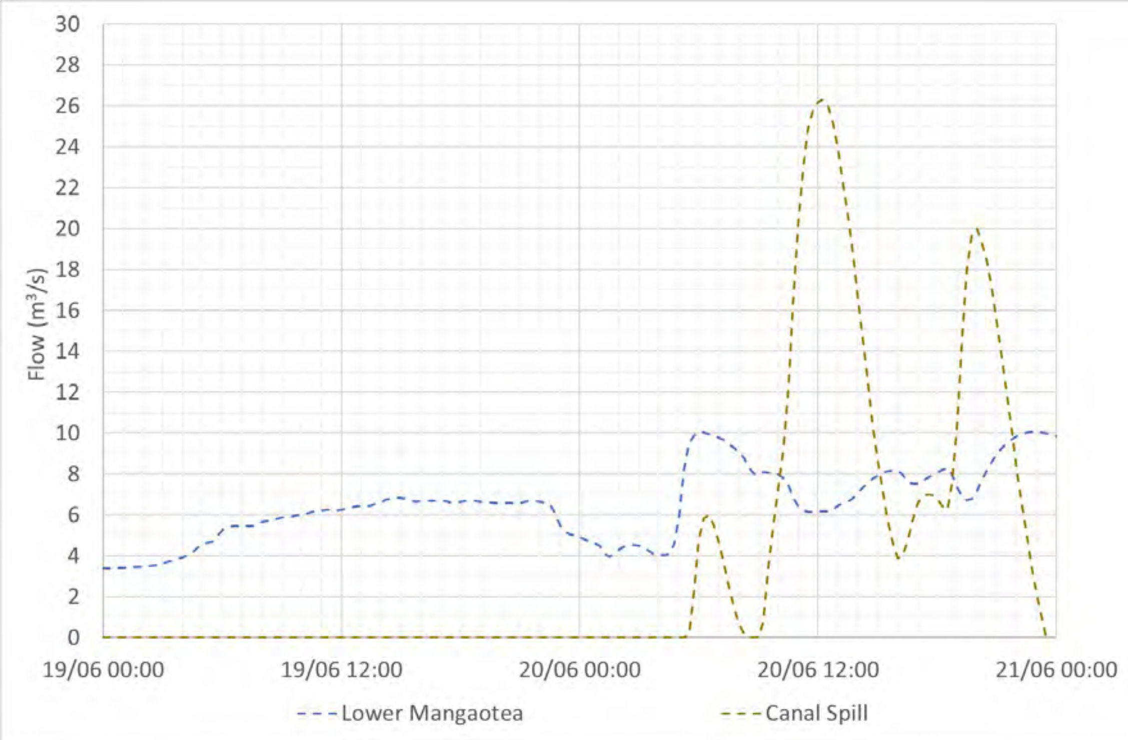


Figure 39 presents recorded and modelled water levels at the Salisbury gauge. The recorded peak Salisbury gauge water level (with the intake gates closed) at approximately 6am 20 June, is similar to normal operating levels immediately prior to the event (i.e., the peak runoff flow at the Salisbury gauge was similar to normal operating flow of 5.2m^3/s). The model results also show a similar trend (i.e., the peak levels are similar to the levels prior to the runoff peaks). We also consider there is a reasonable match with the time of the peaks. The model water level results however are typically 500-800mm higher than the recorded levels. We consider the model results at this location are sensitive to the status of the In-Race Generator at the time of the event, as we note that under normal operating flows, the water levels at the Salisbury gauge are in the order of RL 204.70m at times. Further information regarding the status of the In-Race generator at the time of the event would be required to improve the calibration.

Figure 39: June 2015 – Modelled Upper Canal Levels

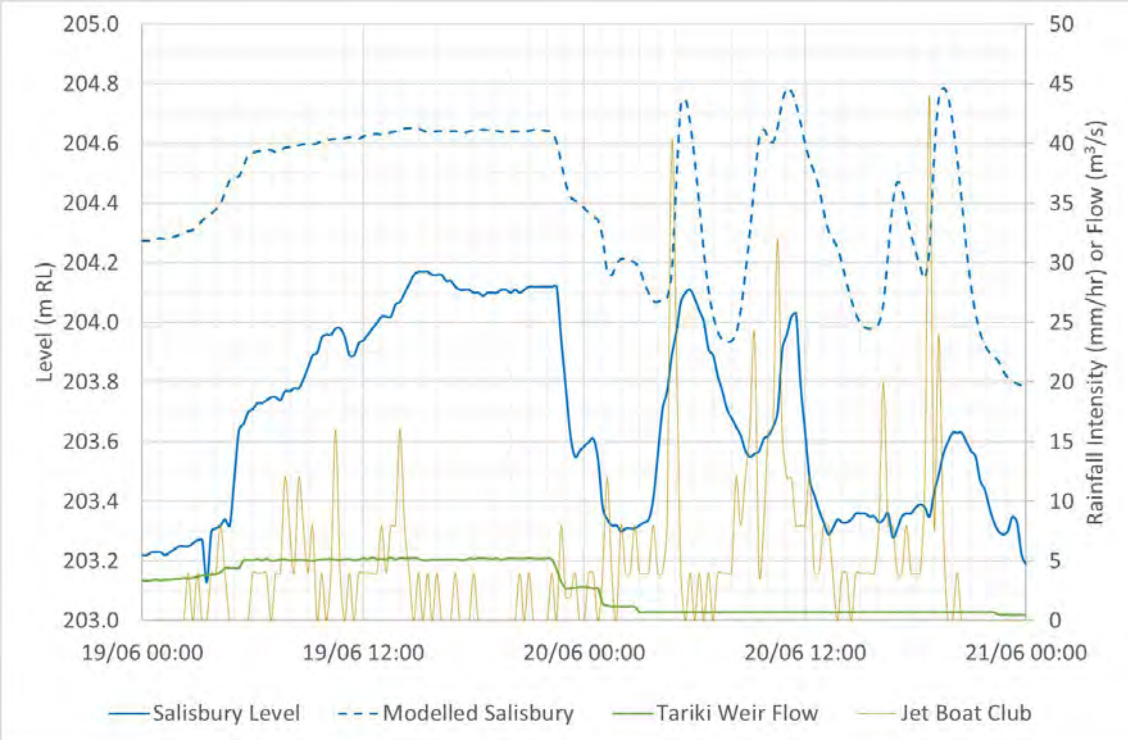
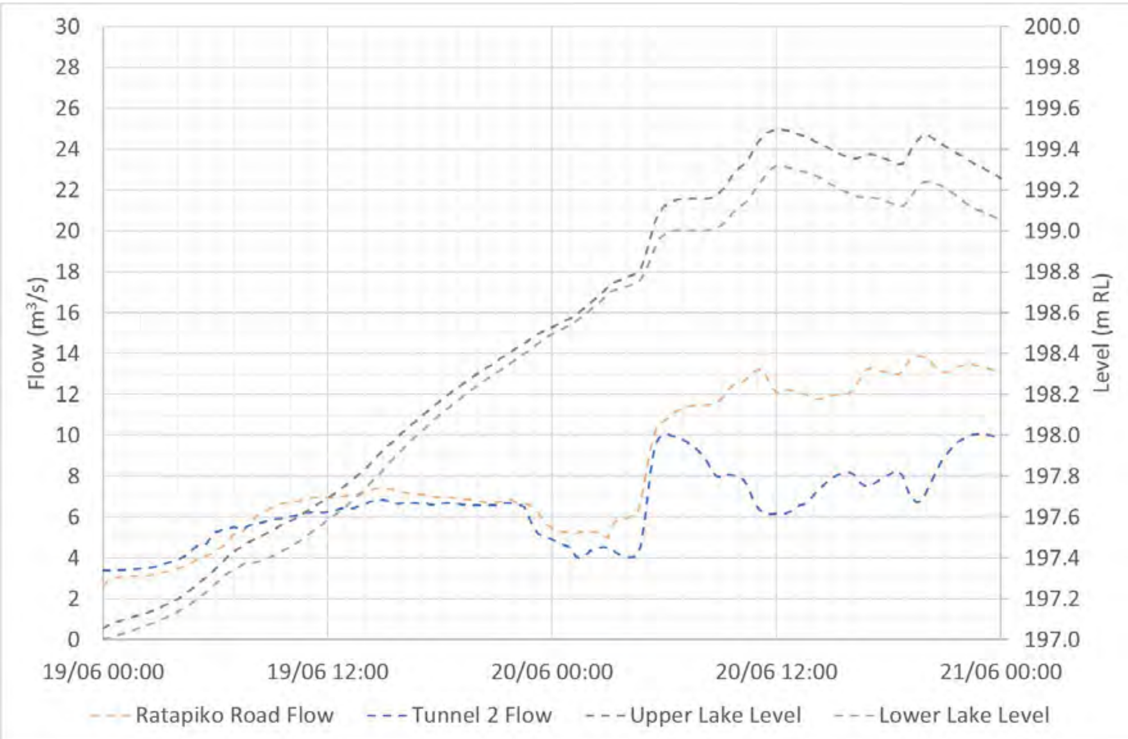


Figure 40 presents the model results at Ratapiko Road. The results indicate that a peak flow of approximately 14m^3/s passed through the culverts and into the lower lake. The head loss through the culverts appears to have been approximately 200mm at the peak of the event. We understand that Ratapiko Road did not overtop, and the model results reflect this observation.

Figure 40: June 2015 – Modelled Ratapiko Road Flow and Levels

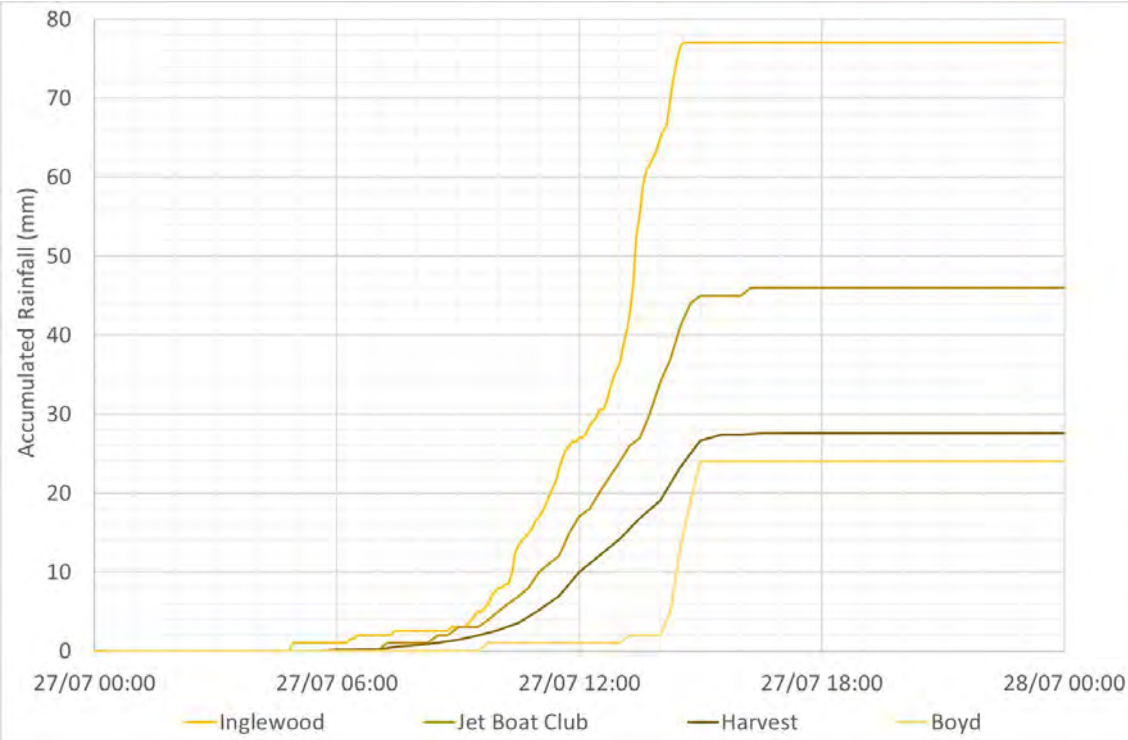


4.0 July 2017

4.1 Recorded Rainfall

Figure 41 presents the recorded rainfall at the four rainfall gauges. The Boyd rainfall gauge appears to have provided erroneous data during the event as the rainfall pattern is significantly different to the other gauges. Again, the Harvest gauge total depth is significantly lower than the Jet Boat Club although the rainfall pattern is very similar. Similarly, the Inglewood gauge demonstrates a similar pattern, but with a significantly higher rainfall depth. We have discounted the Inglewood gauge for calibration as the in favour of the Jet Boat club gauge located within the catchment.

Figure 41: July 2017 - Recorded Accumulated Rainfall



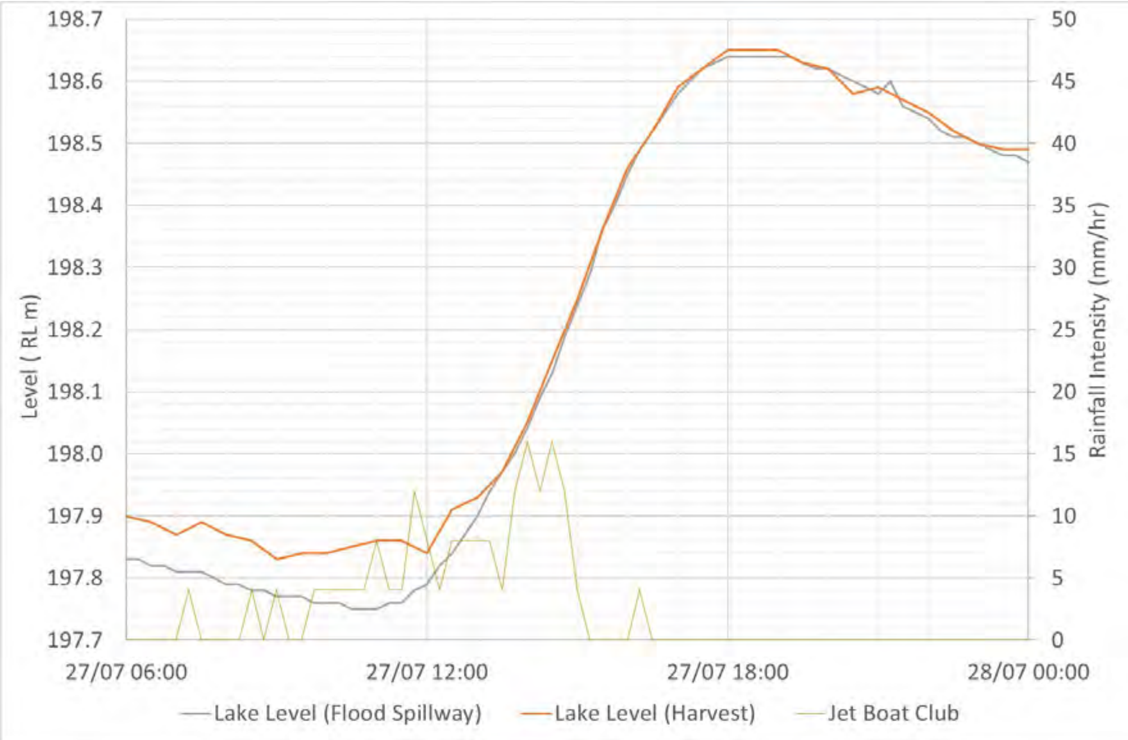
The majority of rainfall occurred across a six hour period from approximately 9am to 3pm on 27 July 2017. The rainfall was relatively steady with an average intensity of 7mm/hr and peak 15 minute intensity of 16mm/hr. The rainfall ceased relatively abruptly at approximately 3pm.

We have placed the selected Jet Boat Club gauge in a regional context within RILEY Dwg. 18MTK/ENH-50, which shows the recorded totals throughout the event from a number of other gauges in the region. This shows that the Jet Boat Club recorded similar rainfall depth to other gauges further to the south.

4.2 Recorded Lake Levels

A peak lake level of RL 198.64m (flood spillway) was recorded an extended period from approximately 6pm – 7pm. With a service spillway crest of RL 198.67m, we note the spillway does not appear to have operated. The Harvest recorded lake level and flashboard operation data also appear to confirm this.

Figure 42: July 2017 - Recorded Lake Levels and Rainfall

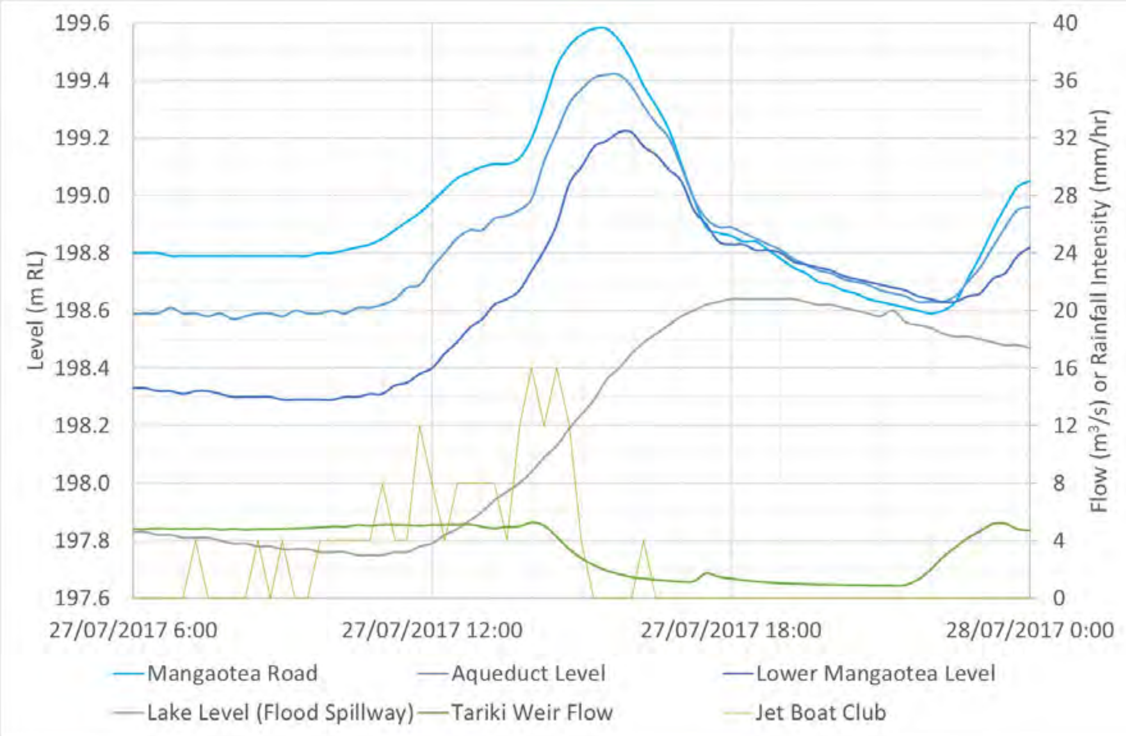


4.3 Recorded Canal Levels

Figure 43 presents the recorded water levels within the lower canal. The Aqueduct canal water level peaked at RL 199.42m (i.e., it appears the canal did not overtop as the canal spillway gate crest is at RL199.72m). The recorded canal water levels indicate that the Mangaotea Stream flow did not overtop into the canal.

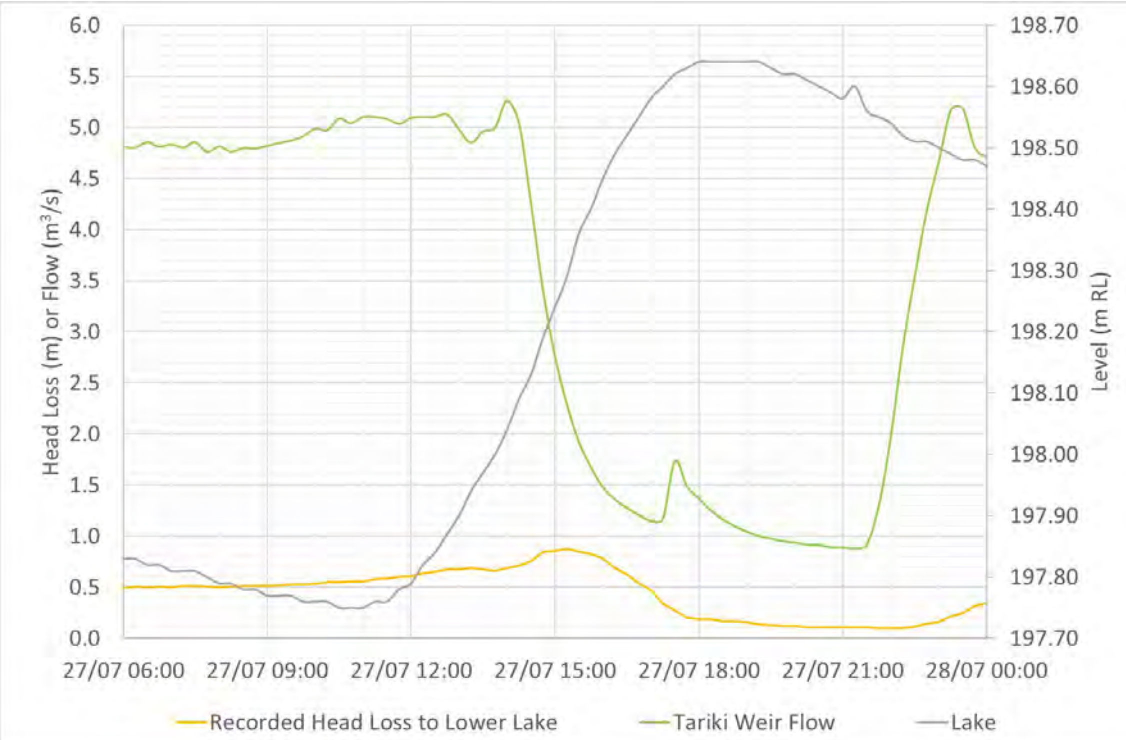
The head losses between the three canal instruments appear to remain relatively constant throughout the event.

Figure 43: July 2017 - Recorded Lower Canal Levels



At Lower Mangaotea the peak head loss to the Lower Lake occurred at 3:15pm (indicating the time of peak canal flow), approximately 0.75 hours after the peak rainfall occurred. Canal head loss to the Lower Lake peaked at approximately 0.9m during the event as presented with Figure 44.

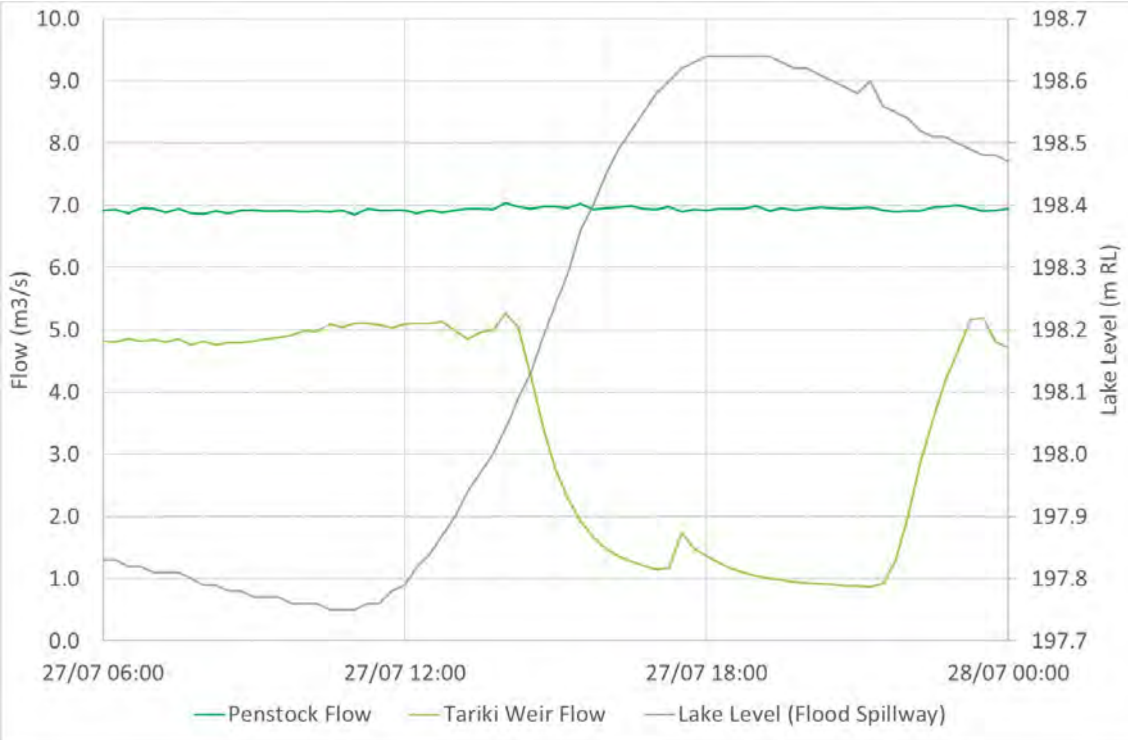
Figure 44: July 2017 - Recorded Head Loss (Lower Mangaotea to Lower Lake)



4.4 Recorded Flows

The recorded flow hydrographs are presented within Figure 45.

Figure 45: July 2017 - Recorded Flow



4.5 Spillway Operation and Derived Inflows

The spillways did not operate during the event.

Figure 46 presents the derived runoff inflow. The inflow hydrograph has a single peak, with a peak derived runoff inflow of approximately 15m^3/s. There appears to be approximately 1 hour delay (i.e., lag) from the peak rainfall to the peak inflow to the lower lake.

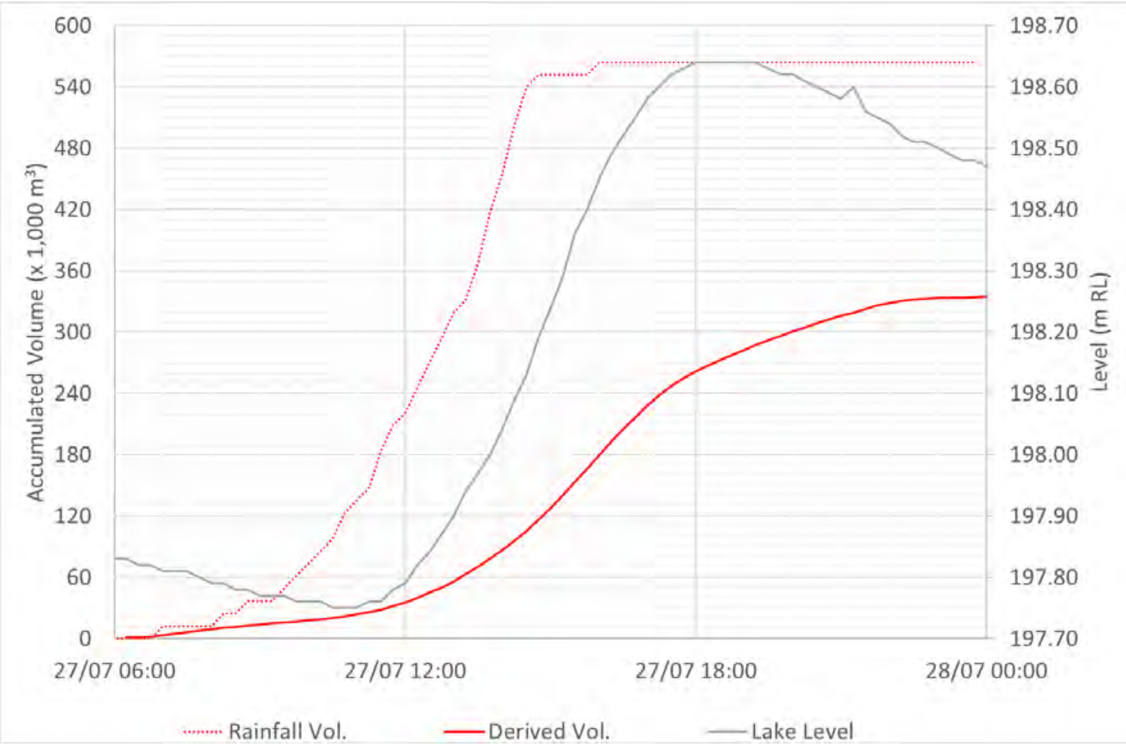
The falling limb is relatively steep as the rainfall ceased relatively abruptly. Direct flow appears to end at approximately 6.15pm which happens to coincide with peak lake level.

The baseflow prior to the event appears to be approximately 1.0m^3/s, with direct flow commencing at approximately 11am. Prior to 11am, the total rainfall depth was 8mm, indicating that the initial loss was approximately 8mm.

Figure 46: July 2017 - Derived Runoff Inflow

Figure 47 presents the derived volumes. By midnight 28 July, approximately 60% of the rainfall volume appears to have entered the lower lake. The direct flow volume appears to have been approximately 280,000m^3 (from 11am to 8pm). This equates to an excess depth of approximately 23mm over the catchment area.

Figure 47: July 2017 - Derived Runoff Volume



4.6 Calibration Parameters

Table 5 presents the selected calibration parameters. A constant loss of 3.5mm and an initial loss of 8mm results in an excess depth of 21.8mm (from the total 46mm), with a direct runoff volume of approximately 270,000m^3.

Table 5: Calibration Parameters

Catchment	Initial (mm)	Constant (mm/hr)	PRF	Lag Time (minutes)	Initial Discharge (m^3/s/km^2)	Baseflow Coefficient (hr)	Baseflow Fraction	Baseflow Steps
Salisbury	8	3.5	450	75	0.1	10	1.0	2
Mangaotea Road	8	3.5	450	75	0.1	10	1.0	2
Lower Mangaotea	8	3.5	450	75	0.1	40	1.0	2
Tunnel 3	8	3.5	450	75	0.1	10	1.0	2
Upper Lake	8	3.5	450	75	0.1	10	1.0	2
Lower Lake	8	3.5	450	75	0.1	10	1.0	2
Tributary 1	8	3.5	450	75	0.1	40	1.0	2
Tributary 2	8	3.5	450	75	0.1	20	1.0	2
Mangaotea	8	3.5	450	120	0.1	60	1.0	2

Two baseflow steps were required to distribute the baseflow over a long duration.

Figures 48 and 49 present the calibrated excess rainfall, with a total excess depth of approximately 22mm.

Figure 48: Calibrated Excess Rainfall

Figure 49: Calibrated Accumulated Excess Rainfall

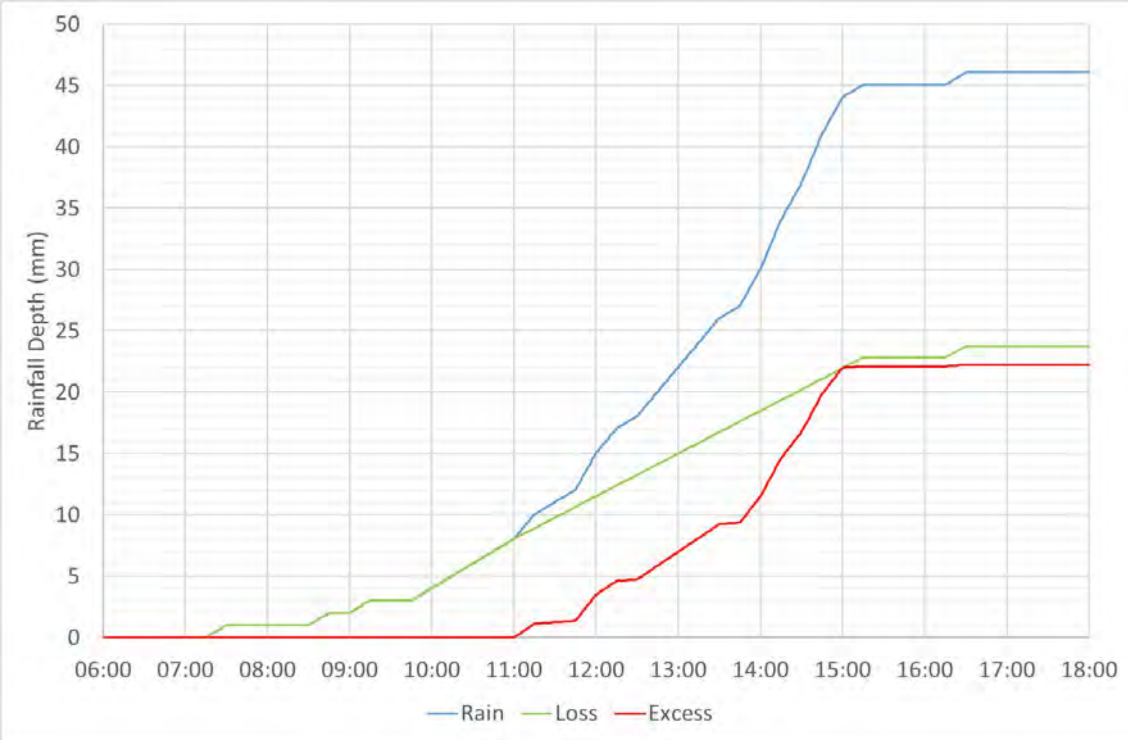


Table 6 summarises the trialled initial and constant losses. We consider the selected initial and constant losses best represent the estimated direct inflow volume of approximately 280,000m^3.

Table 6: Trialled Initial and Constant Losses

Initial (mm)	Constant (mm/hr)	Excess Rainfall (mm)	Direct Volume (x 1,000m^3)
3	3.0	25.8	319
8	3.0	24.5	303
8	3.5	22.2	275
8	4.0	20.0	248

4.7 Results and Discussion

Figure 50 and 51 present the modelled lake volume and flows. We consider a good match has been obtained.

Figure 50: July 2017 – Modelled Lake Volume

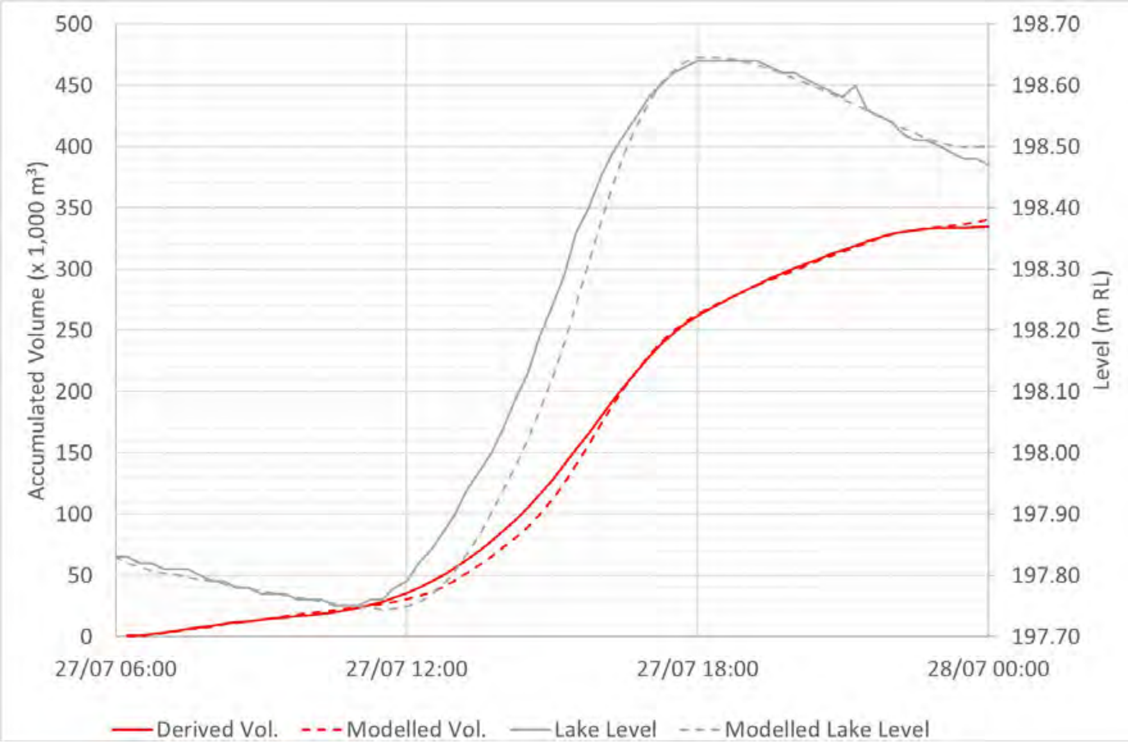
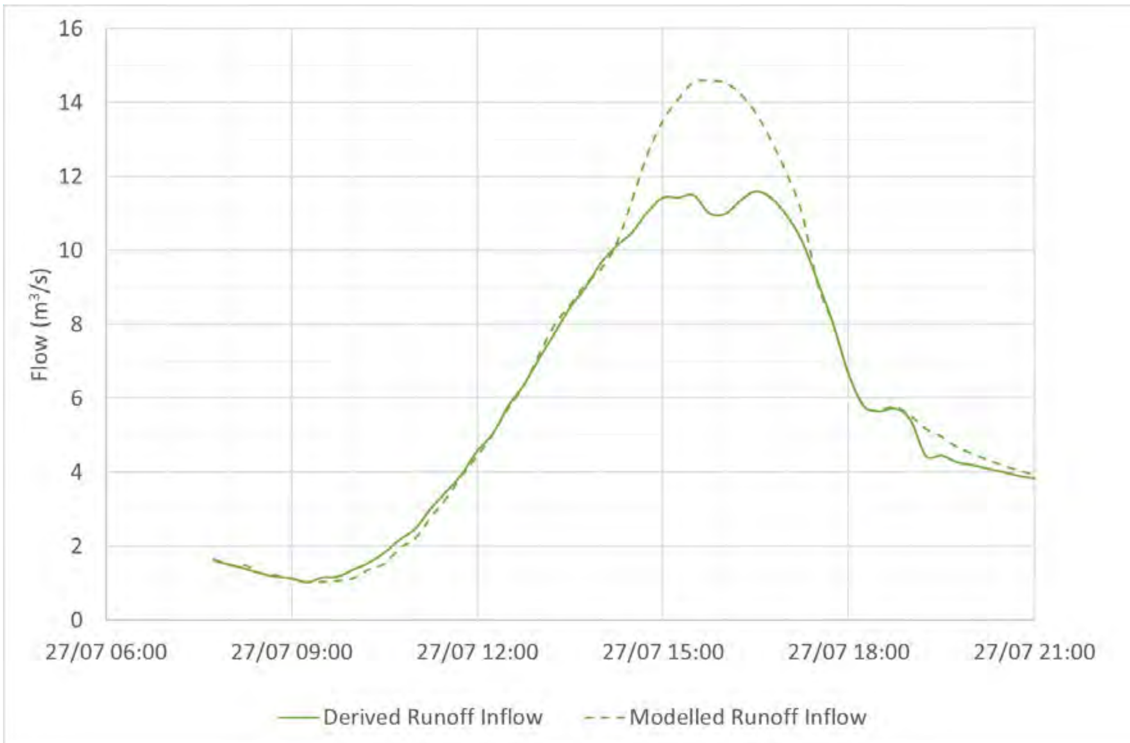


Figure 51: July 2017 – Modelled Runoff Inflow



We consider that the selected lag times provide a good match with the time of peak flow and a conservative match with the peak flow at the lower lake. We note lag times can vary seasonally and with storm direction and rainfall temporal distribution.

The model results at Lower Mangaotea and the Aqueduct are presented within Figures 52 and 53. A good match with recorded water level was obtained, particularly at higher lake levels. The peak flow at Lower Mangaotea appears to have been approximately 9m^3/s. The selected lag time provides a good match with the time of peak water level.

Figure 52: July 2017 – Modelled Lower Mangaotea Results

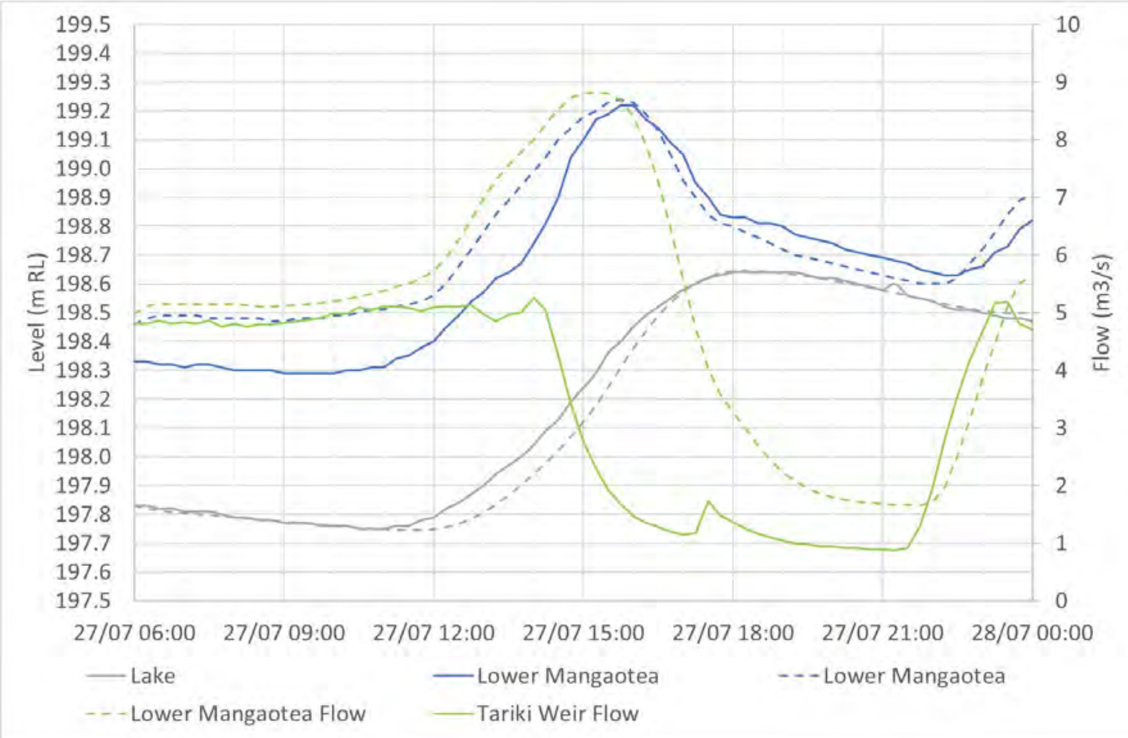


Figure 53: July 2017 – Modelled Aqueduct Results

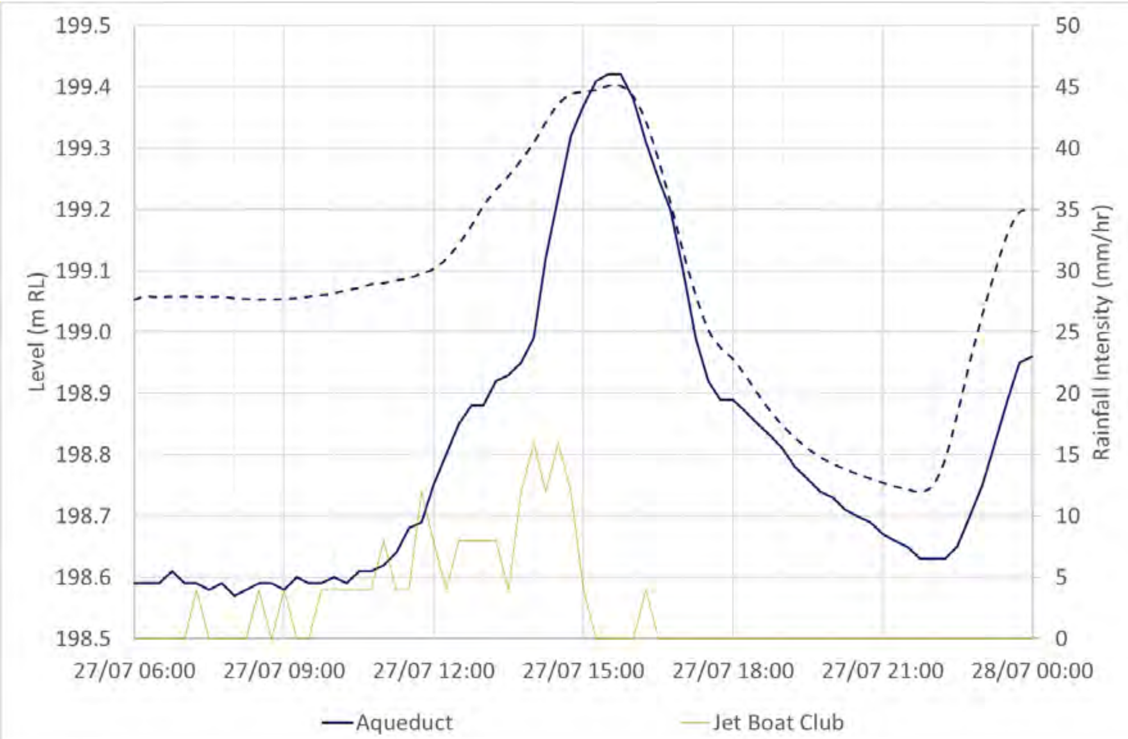
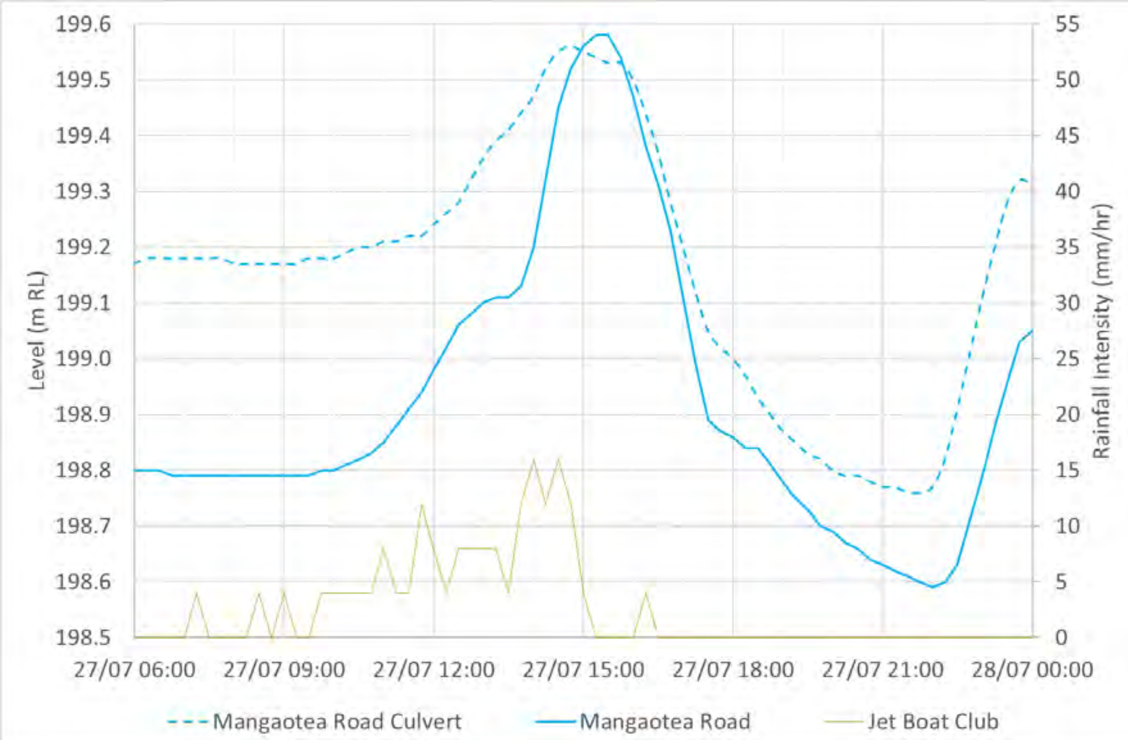
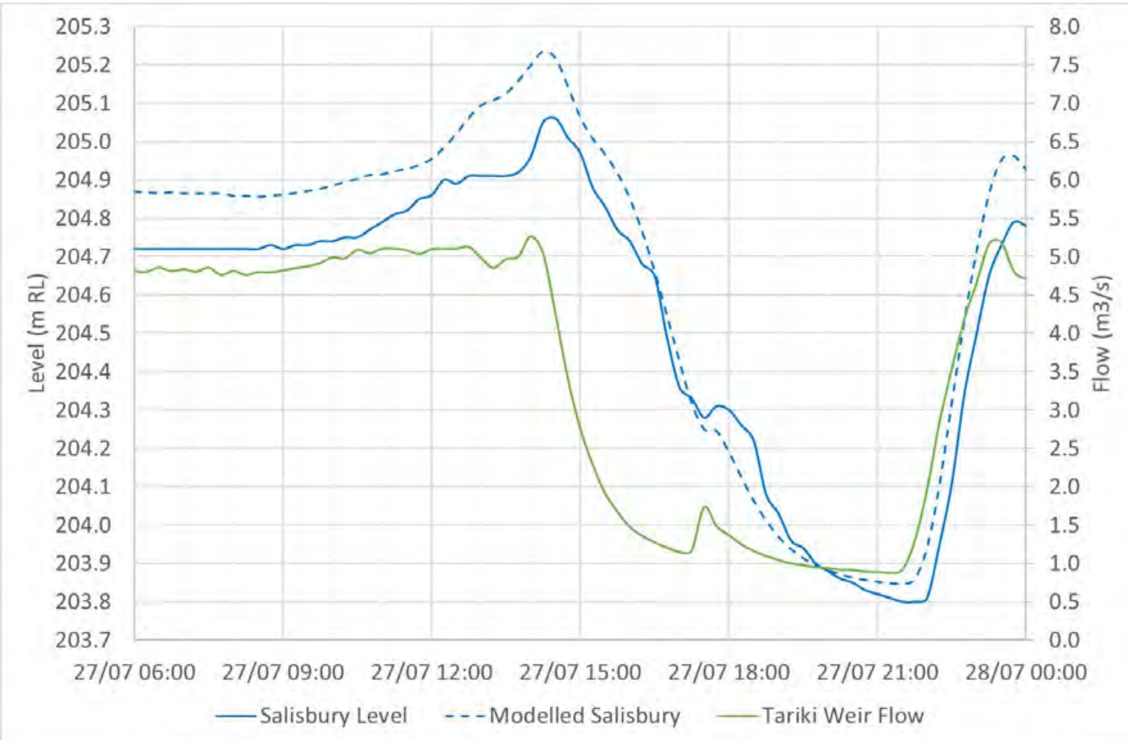


Figure 54: July 2017 – Modelled Mangaotea Road Results



The model results at the Salisbury gauge are presented within Figure 55. The model results appear conservative. We note that the model assumptions made for the In-Race generator appear to provide a better match with the status of the In-Race generator at the time of this event.

Figure 55: July 2017 – Modelled Salisbury Results



5.0 Summary

Table 7 presents a summary of the calibration model results.

Table 7: Model Results[1.]

	Peak Runoff Inflow (m^3/s)			Flood Volume[2.] ($x1,000m^3$)			Peak Lake Level (m RL)		
Event	**Derived**	**Modelled**	**Diff.**	**Derived**	**Modelled**	**Diff.**	**Recorded**	**Modelled**	**Diff.**
2017	15	12	+3	335	335	0	198.64	198.65	0.01
2013	10	12	+2	462	462	0	198.61	198.59	-0.02
2004	18	24	+6	1,016	1002	-14	198.99	199.10	0.11
2015	42	36	-6	2,160	2,089	-71	199.30[3.]	199.32	0.02

1. Presented in increasing flood volume.
2. Over modelled period.
3. Estimated.

Table 8 provides a summary of the calibration parameters which vary between the calibration events. Other catchment parameters that have not been varied have not been presented.

Table 8: Calibration Parameters

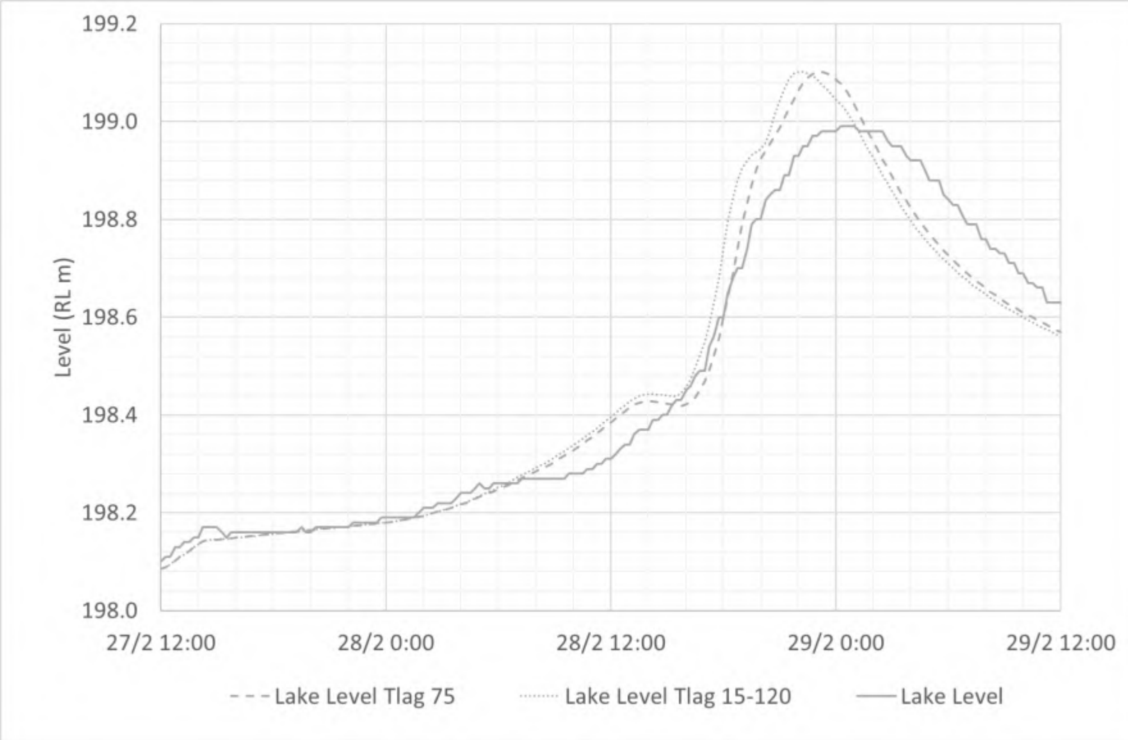
Event	Initial (mm)	Constant (mm/hr)	Lag Time Range (minutes)	Baseflow Steps
February 2004	75	3.0	75	2
June 2013	7	13	15 - 120	1
June 2015	60	3.0	15 - 120	1
July 2017	8	3.5	75	2

The initial loss values appear to be variable across the calibration events. However, we consider there is sufficient evidence to demonstrate that initial losses within the catchment are relatively high with drier antecedent conditions (as shown in Figure 1 for the February 2004 event). The distribution of rainfall within the catchment, rainfall intensity, and additional storage effects within the catchment are likely to have some effect on the effective loss values.

The constant loss values have close agreement across the three larger events.

Two different lag time ranges were used when modelling the calibration events. We have compared the two ranges in the context of the 2004 event, to assess their impact on the outputs of the assessment. Modelled lake level results are presented in Figure 56. They show that the two lag time ranges, both present comparable results (to each other) for the 2004 event, with the 75-minute lag time range being the most appropriate.

Figure 56: Lag Time Comparison – 2004 Event



We consider that the July 2017 event provides the most suitable parameters to be used within the design event models as the initial loss is low (i.e., wet antecedent conditions) and the constant loss value of 3.5mm/hr is representative of low losses that also occurred during the two largest events on record (i.e., 2004 and 2015).

Table 9 provides a summary of the calibrated water levels within the canal.

Table 9: Canal Summary – Modelled Peak Water Level Versus Recorded Peak Water Level (mm)

Event	Salisbury	Mangaotea Road	Aqueduct	Lower Mangaotea	Comment
February 2004	+800	-100	n/a	-100	Prior to construction of in-race generator leads to uncertainty with Salisbury Road.
June 2013	n/a	n/a	+100	n/a	Adequate calibration
June 2015	+700	n/a	-200	-200	Canal overtopped. Uncertainties with surveyed overtopping level.
July 2017	+200	0	0	0	Adequate calibration

1. Depths rounded to nearest 100mm.

Overall, we consider that the developed model provides a good basis for predicting the flood effects during the design events. Further calibration with other events and more detailed calibration could be considered, particularly for the canal.

We recommend that the canal and lake level recorder sites are reviewed, to ensure that the peak water level during future events is adequately captured e.g., lake level gauge situated at a location some distance away from spillways and the penstock intake. Water level information immediately upstream of the Tributary 1 culvert, within the upper lake and upstream of Tunnel 3 would assist further assessments.

Future assessments could consider improving the understanding of the tunnel hydraulics and controls/flow types at a range of flows and downstream lake levels.

APPENDIX G

Design Rainfall

Design Rainfall

1.0 General

Ratapiko Dam has most recently been assessed as having a low Potential Impact Classification (PIC) (RILEY, 2017). The New Zealand Society on Large Dams Safety Guidelines, 2015 (NZSOLD Guidelines) recommend that a low PIC dam has an inflow design flood of between the 100-year and 1,000-year flood event. Trustpower Limited has advised RILEY that they wish to use the 1,000-year flood event as the inflow design flood. In order to determine the design flood event (and any others which may be of relevance), extreme rainfall estimates are required.

We note that the NZSOLD Guidelines do not recommend inclusion of the effects of climate change in the estimation of extreme flood events.

The latest Comprehensive Dam Safety Review (AECOM, 2017) recommended that measures are investigated and implemented to ensure the dam meets the minimum requirements of the NZSOLD Guidelines.

As part of this assessment, we have assessed the following extreme rainfall events:

- 2.33-Year (mean annual)
- 10-Year
- 100-Year
- 1,000-Year

It is generally accepted within the engineering industry that when estimating extreme rainfall depths (say in excess of the 100-year event) there is a large range of uncertainty. We have therefore used a number of assessment methods to assess the design rainfall depths. The methods used were as follows:

- Review of historical events
- HIRDS V4
- “At site” frequency analysis
- Griffiths (2015) methodology

The rainfall monitoring locations included in this analysis and the length of record are included in Appendix A, Table 4 and have been mapped in RILEY Dwg. 18MTK/ENH-10. For clarity these are as follows:

- Inglewood @ Oxidation Ponds
- Boyd (Mangaotea Aqueduct)
- Service Spillway
- Jet Boat Club (Lake Ratapiko)

2.0 Historical Events

Table 1 provides a summary of the most significant rainfall events at the historical Inglewood gauge (from 1909 to 1993) (sourced from the National Climate Database) and the current Inglewood gauge. The February 1971 and March 1990 events are commonly accepted as the most significant flood events that have occurred in the inland Taranaki region in recent history. We note that the historical data is daily data and given that daily readings would be from manual rain gauges, logged at potentially inconsistent times of day (refer to definition of daily manual gauges in NEMS, 2013), it is possible 24-hour depths may be under or over recorded.

Table 1: Inglewood Rainfall Depths

Date (of reading)	Period	Depth (mm)
22 February 1935	Daily	278
20 April 1939	Daily	193
25 February 1971	Daily	338
10 March 1990	Daily	275
19 June 2015	24-hour	178

The 1979 Hydrology and Flood Study for the Patea Hydro Electric Power Scheme discusses the 1971 event. The report presents an isohyetal map of the recorded rainfalls (two day duration) during the event. A copy of this map (Map 1) is presented at the rear of this Appendix. We have also created a new map (Map 2) presenting the one day duration (9am to 9am) depths, using rainfall depth data sourced from the NIWA National Climate Database (CliFlo). The Lake Ratapiko catchment boundary is also included in Map 2.

The maps show that rainfall depths were greatest to the north of the Lake Ratapiko catchment (at the Inglewood and Tarata gauges) and on the higher slopes of Mount Taranaki.

Map 3 shows the mean annual total rainfall depths across the region. This map also indicates that the Lake Ratapiko catchment receives slightly lower rainfall to the Inglewood gauge and that there is a relatively small rainfall gradient across the catchment.

3.0 HIRDS V4

Total rainfall depths were obtained from HIRDS V4. HIRDS V4 includes the current Inglewood gauge within the analysis used to produce depth-duration frequency and intensity-duration frequency data (High Intensity Rainfall Design System Version 4, NIWA, August 2018). The selected HIRDs V4 locations are shown on RILEY Dwg. 18MTK/ENH-50. We note that the HIRDs V4 Inglewood location is the same as the Inglewood rainfall gauge. The HIRDs V4 -Lake location is also shown on RILEY Dwg. 18MTK/ENH-10. We note that HIRDS V4 uses a Generalised Extreme Value (GEV) distribution and depths are provided for return periods up to 250 years.

The results presented within Table 2 for the100-year event indicate that rainfall is lower at the lake than at Inglewood (difference increases with increasing rainfall duration). We consider that it is most appropriate to use the HIRDS data for the lake site in preference to the Inglewood site as the lake site is located within the catchment. Between the Inglewood and the Lake site, HIRDs v4 identifies a rainfall gradient of increasing prevalence as the rainfall duration increases. We note that this gradient is less than 10% for durations less than 24 hours, and we therefore do not consider that incorporation of a rainfall gradient is necessary for this assessment. Table 3 and Figure 1 present the HIRDS V4 rainfall depths for the lake site. 1,000-year return period rainfall depths were derived using the output model parameters from HIRDS (parameters c – i).

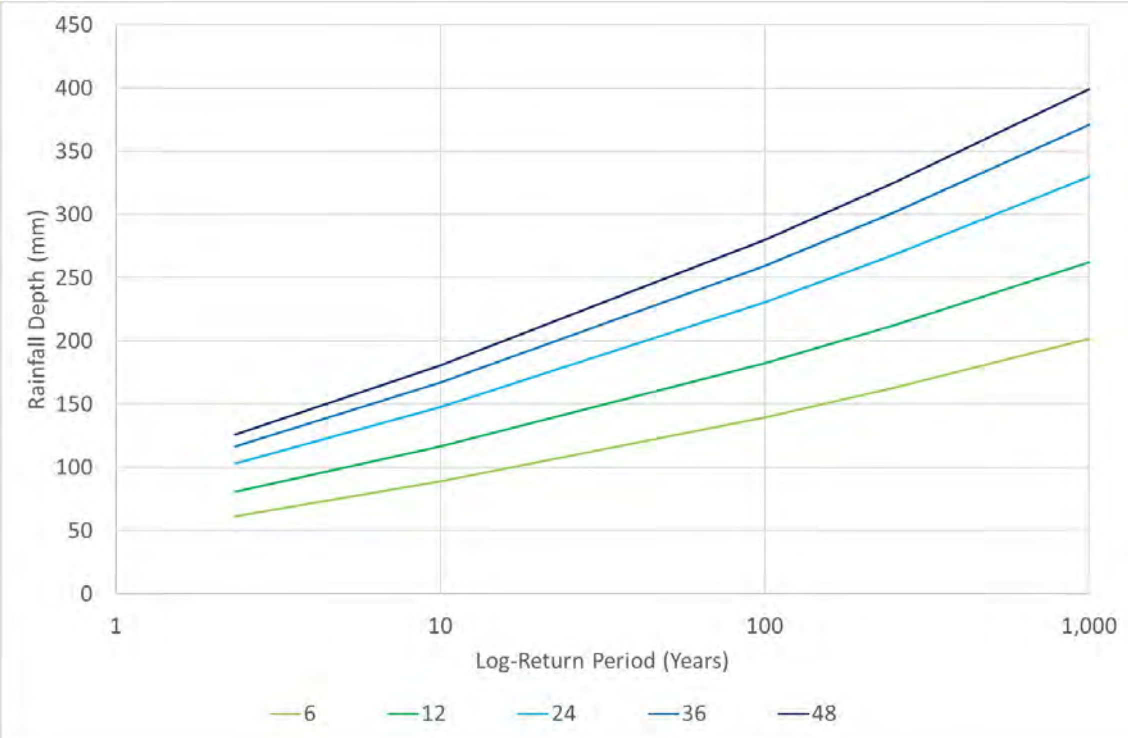
Table 2: HIRDS V4 100-Year Rainfall Depths (mm)

Duration (Hours)	HIRDS V4 - Inglewood	HIRDS V4 - Lake	Lake/Inglewood Ratio
6	143	140	0.97
12	192	182	0.95
24	254	230	0.91
36	328	259	0.85
48	377	280	0.82

Table 3: HIRDS V4 Rainfall Depths - Lake Site (mm)

Return Period (Years)	1-Hour	3-Hour	6-Hour	12-Hour	24-Hour	36-Hour	48-Hour
2.33	28	46	61	81	103	116	126
10	40	66	89	116	148	167	180
100	64	104	140	182	230	259	280
1,000	93	152	202	262	330	371	399

Figure 1: HIRDS V4 Rainfall Depths - Lake Site (mm)



4.0 Frequency Distribution Methods

We have reviewed the data for the Inglewood, Boyd and Jet Boat Club gauges. The data for the Jet Boat Club and Boyd gauges appears to have significant gaps. Therefore, we consider the data is not appropriate to be used for frequency analysis. Figures 2 and 3 demonstrate that both the 24 hour duration annual maxima for the Jet Boat Club and Boyd gauges display a different trend to that for the Inglewood gauge. We note that we have used the period of record defined in Table 4 of Appendix A, Available Time Series Data. We have selected the full period available, to maximise available record length.

We do not consider the Harvest unit is suitable for use in a frequency analysis due to the shorter record available and the apparent under recording of rainfall depths.

Figure 2: Annual Maxima Rankings

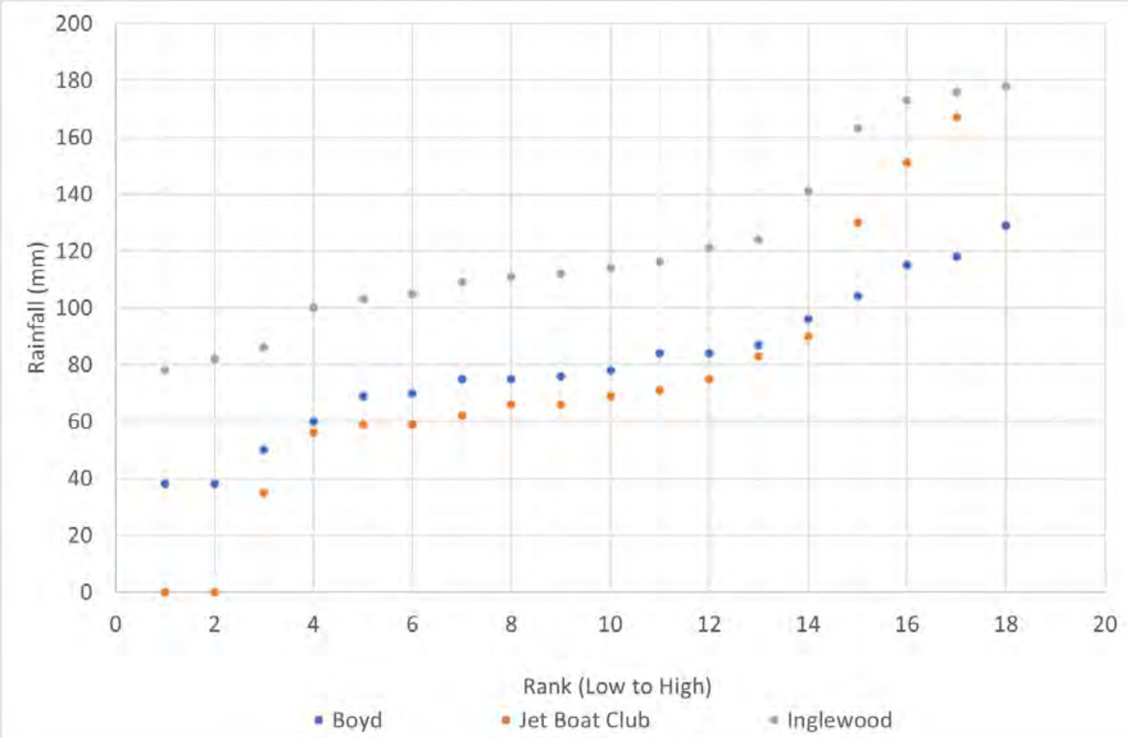
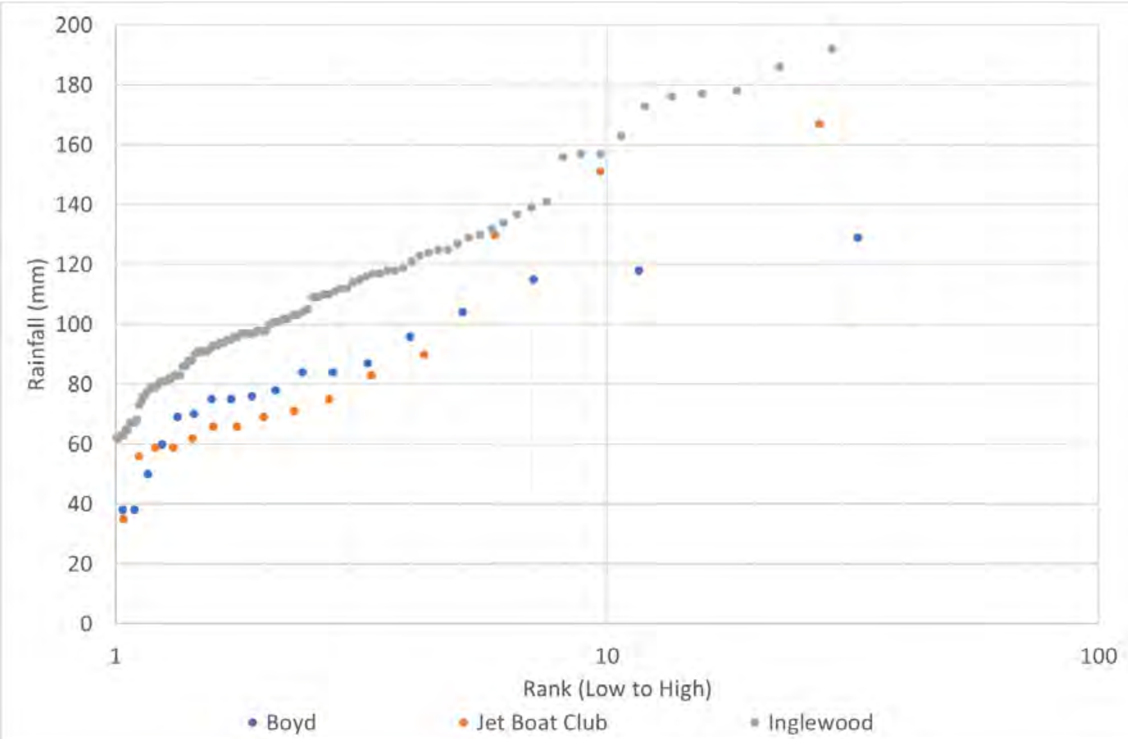


Figure 3: Annual Maxima Gringorten Plotting Position



We do not consider that it is appropriate to use the Inglewood gauge data and any subsequent at site frequency analysis extreme rainfall estimates, because the recorded rainfall at Inglewood is typically significantly higher than that experienced within the catchment (for more detail refer to Appendix F - Calibration). However, for the purposes of comparison and to support the analysis from HIRDS we have undertaken an analysis using the Inglewood gauge (combined records).

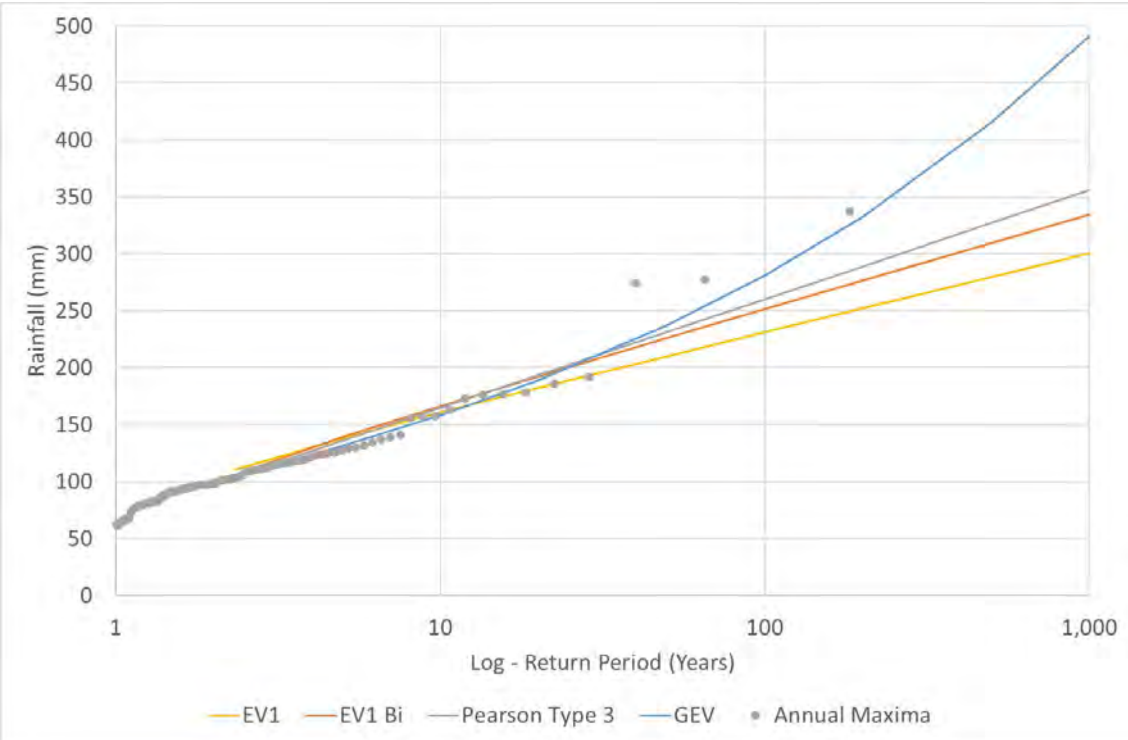
We have assessed the following frequency distributions:

- Extreme Value Type I (EV1)
- GEV
- Pearson Type 3
- Biennial EV1

The assessment of the biennial EV1 distribution is based on the findings of McKercher and Pearson (1989) who found "Of the annual series distributed as EV2, most are shown to be satisfactorily represented by EV1 when a maxima from two year intervals are used."

Figure 4 presents the results for the 24-hour duration event. We consider the Pearson Type 3 distribution provides the best comparison to the HIRDS assessment. We consider the GEV distribution is likely to overestimate the 1,000-year rainfalls depths.

Figure 4: Inglewood Frequency Distribution Results (24 Hour)



5.0 Griffiths Method

The Griffiths method factors the mean of annual maxima rainfall depths (for a certain duration) to determine a range of return period depths up the 1,000-year event. The entire North Island has the same factor. We have used the lake HIRDS data for the mean annual (6, 24 and 48-hour) rainfall depths to provide the base values for the Griffiths method.

Results are presented within Table 4 and Figure 5.

Table 4: Griffiths Method

Return Period (Years)	6-Hour Factor	24-Hour Factor	48-Hour Factor	6-Hour Depth (mm)	24-Hour Depth (mm)	48-Hour Depth (mm)
2.33	1.00	1.00	1.00	61	103	126
10	1.42	1.43	1.40	87	147	181
100	2.31	2.20	2.13	141	227	268
1,000	3.61	3.21	2.87	220	331	362

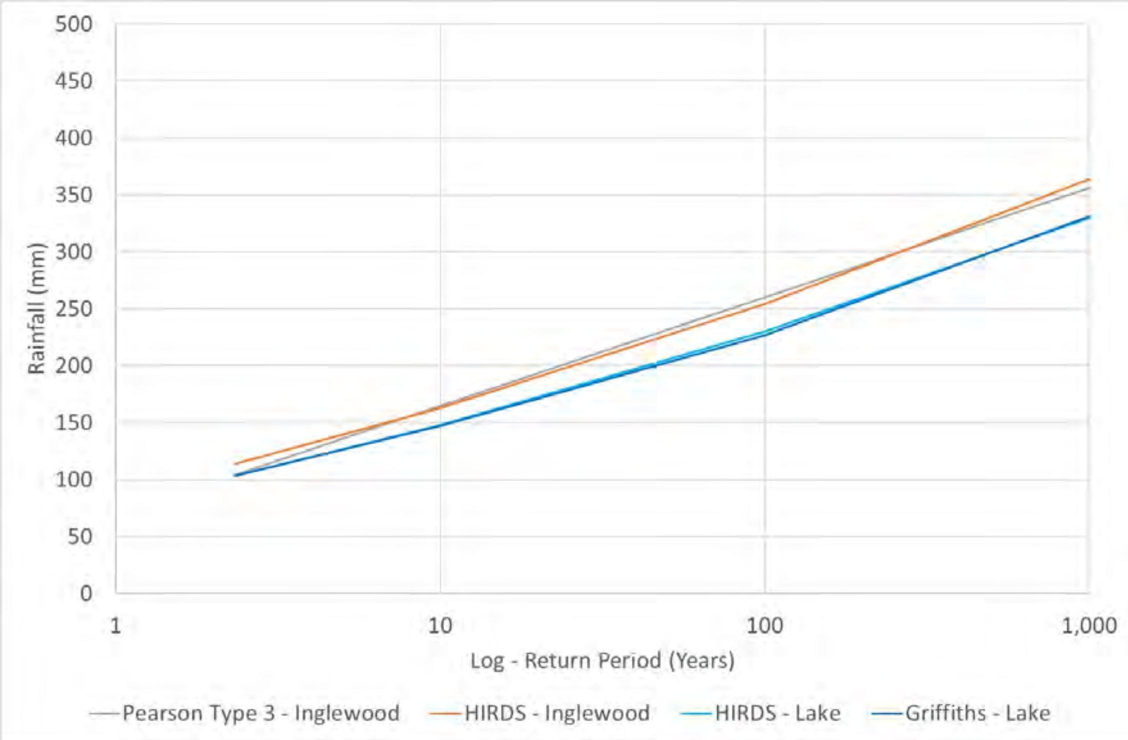
Figure 5: Griffiths Method

The Griffiths method provides similar results to HIRDS.

6.0 Discussion

The different methodologies provide a range of results. Figure 6 demonstrates that the frequency analysis (using Pearson Type 3) and the HIRDS data provides similar results for Inglewood. The HIRDS data for the lake location provides lower results, as the lake is in a lower rainfall area.

Figure 6: 24-Hour Duration Event - Rainfall Comparison



Overall, we consider that the HIRDS results for the lake are the most appropriate to be used within the model as the design rainfall depths, as the lake catchment is in a lower rainfall area than Inglewood. Within Appendix H – Inflow Design Flood, we have considered the effects of rainfall depths in excess of the design rainfall depths. Table 5 presents the selected design rainfall depths over a range of durations.

Table 5: Selected Rainfall Depths (mm)

Return Period (Years)	1-Hour	3-Hour	6-Hour	12-Hour	24-Hour	36-Hour	48-Hour
2.33	28	46	61	81	103	116	126
10	40	66	89	116	148	167	180
100	64	104	140	182	230	259	280
1,000	93	152	202	262	330	371	399

We note that the 1971 rainfall event could potentially have exceeded the selected 24-hour 100-year rainfall depth at the lake. We have attempted to source more information regarding the 1971 rainfall event such as rainfall distribution or lake level observations but were unable to find any information for inclusion in this analysis. Ideally, calibration against the 1971 event could be undertaken or alternatively the 1971 rainfall could be applied to the current model, as a sensitivity analysis. We note that there is no evidence that the dam has ever overtopped, and that the historical events within Table 1 occurred prior to the construction of the flood spillway.

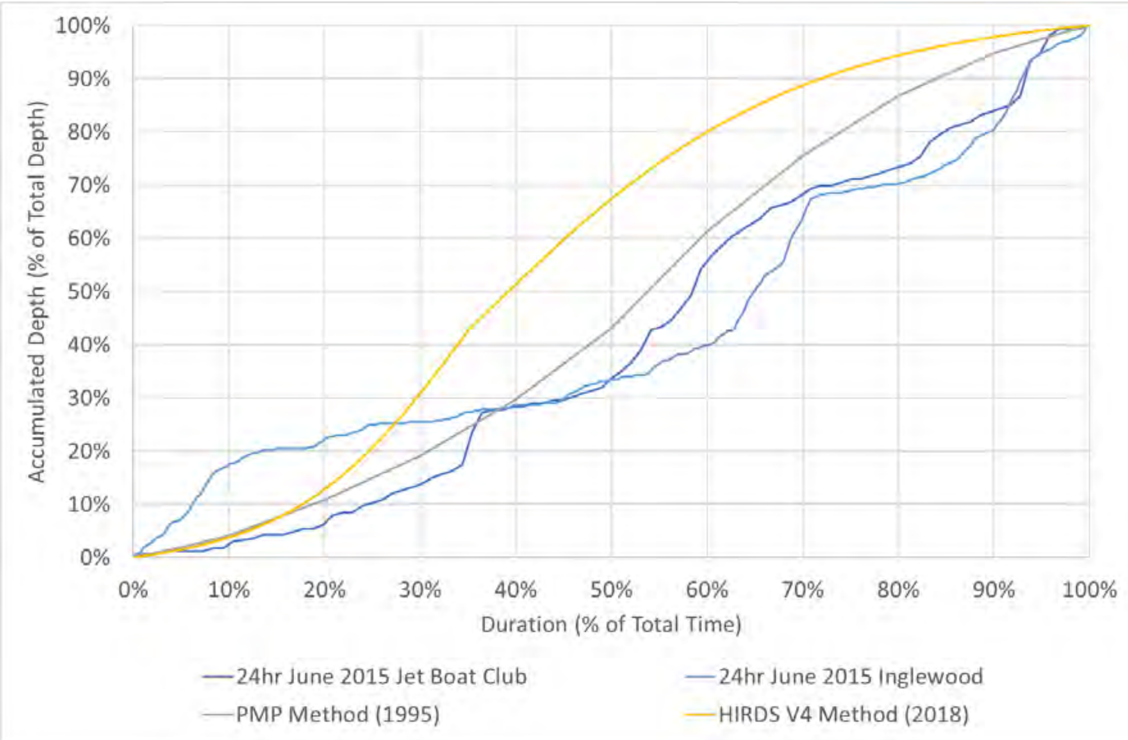
7.0 Spatial Distribution

We have not allowed for spatial distribution (i.e., a rainfall gradient) within the deign model to provide consistency with the calibration events.

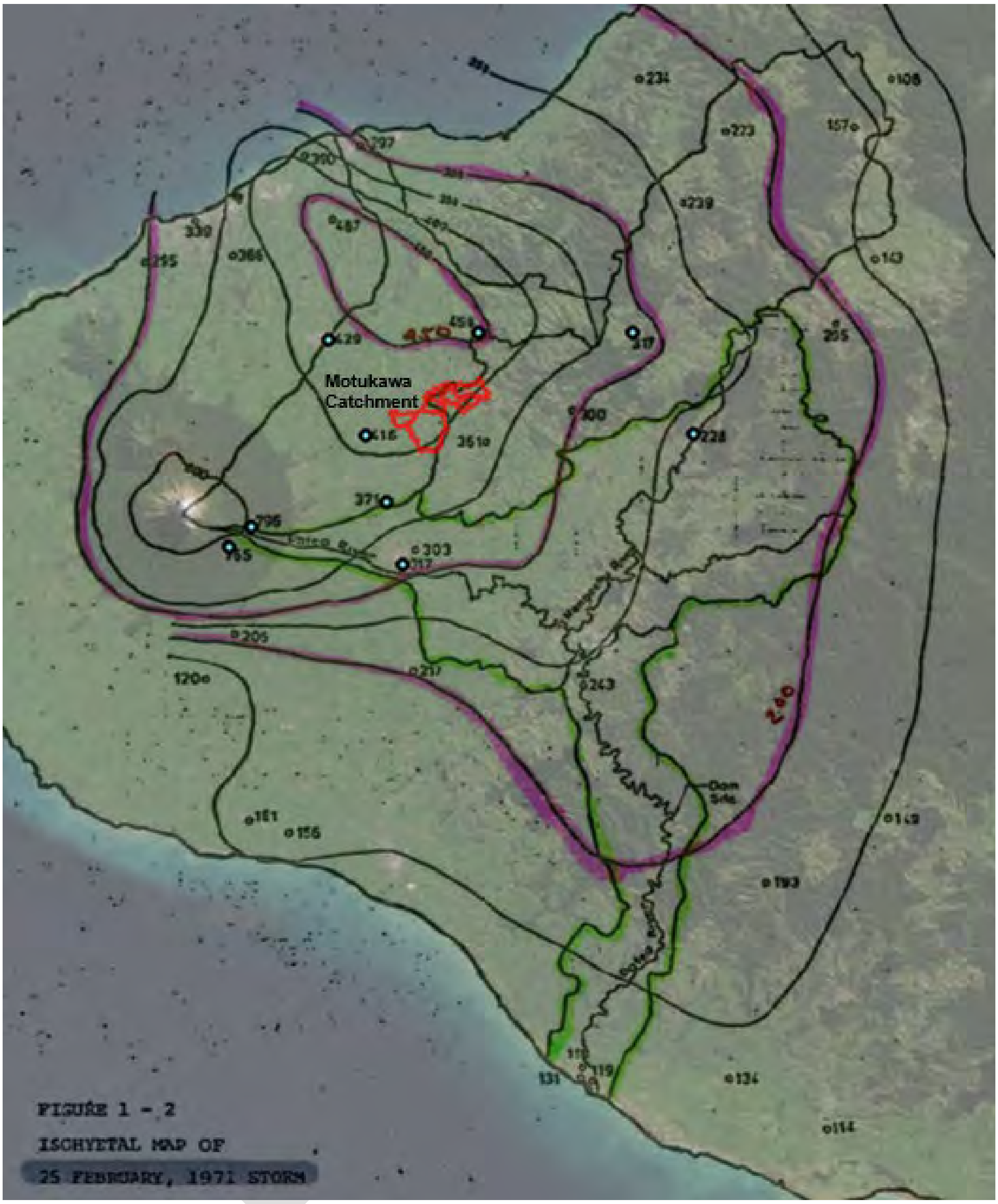
8.0 Temporal Distribution

We have used the 19 June 2015 recorded data (24-hour maximum recorded) from the Jet Boat Club Gauge to derive the rainfall distribution as presented with Figure 7. The 24 hour recorded depth of 167mm is the highest 24-hour depth recorded at either the Boyd or Jet Boat Club gauge. As a comparison, we have also presented the HIRDS V4 distribution, the PMP 1995 (TP19) distribution and the 19 June 2015 recorded data at the Inglewood gauge (24-hour duration). We consider the distributions are likely to obtain similar lake level results, although the June 2015 distribution has the steepest two hour period and therefore, we consider the June 2015 distribution is likely to result in the most conservative results. Further discussion on the rainfall distribution and sensitivity analysis is included within Appendix H.

Figure 7: 24-Hour Rainfall Distribution

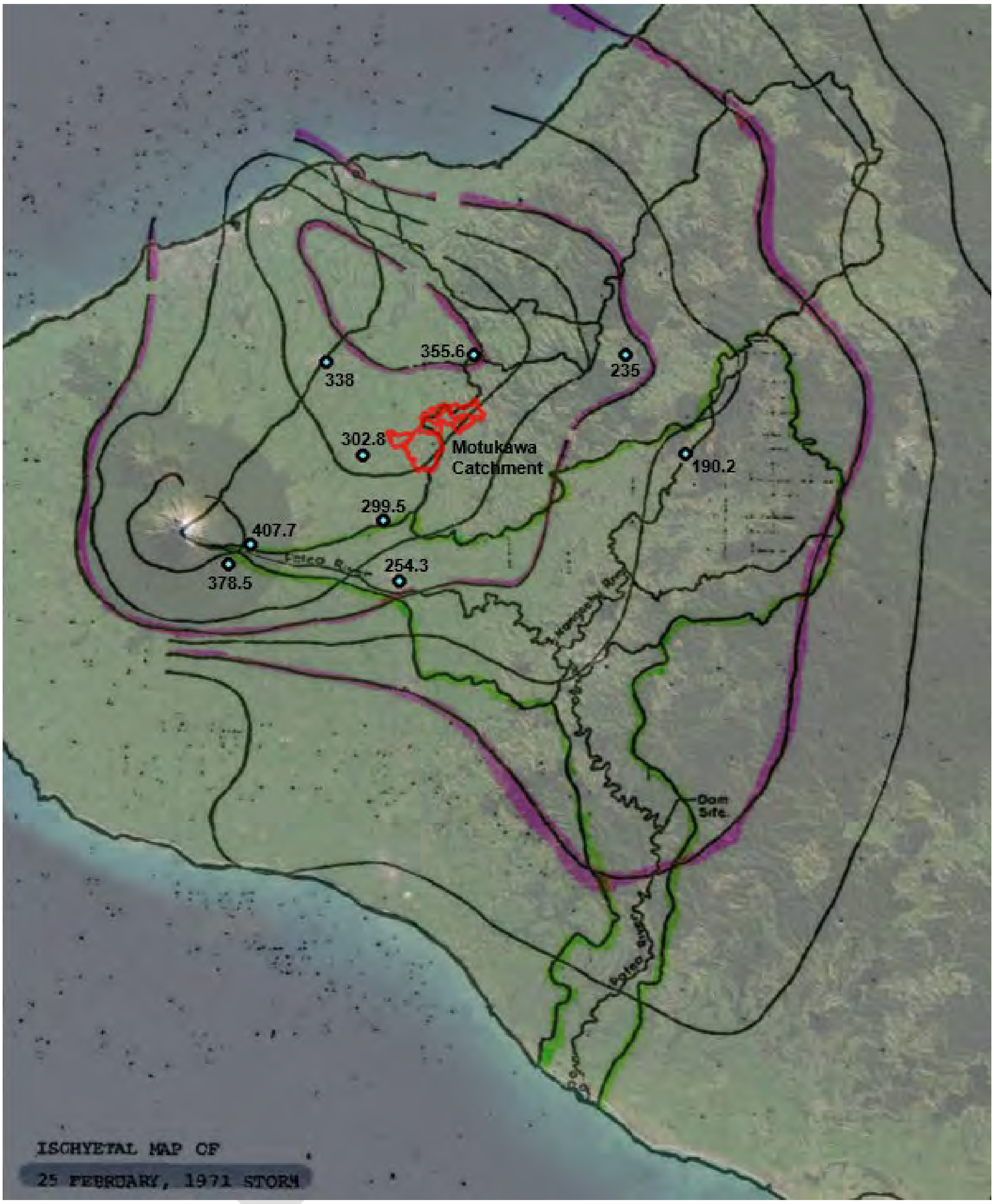


Map 1: Two Day Duration Isohyetal Map of Recorded Rainfalls for 24 and 25 February 1971.

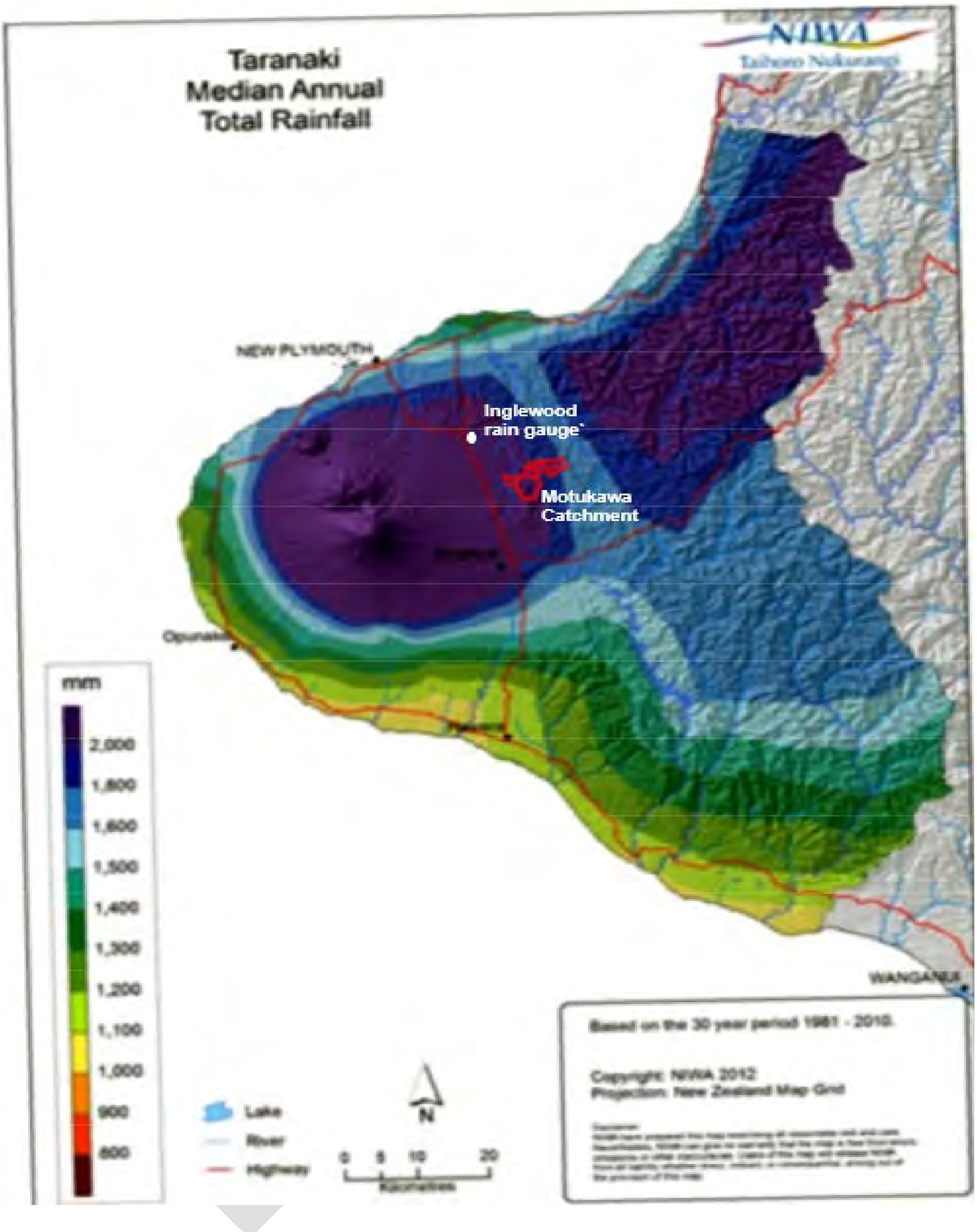


(Sourced from the 1979 Hydrology and Flood Study for the Patea Hydro-Electric Power Scheme.)

Map 2: One Day Duration (9am – 9am) Rainfall Depths for 25 February 1971 (NIWA CliFLO).



Map 3: Median Annual Rainfall Depths in the Taranaki Region (NIWA, 2012).



rain gauge`

APPENDIX H

Inflow Design Flood

Inflow Design Flood

1.0 General

The following methods have been used to derive the Inflow Design Flood (IDF):

- Calibrated models.
- Flood frequency analysis on a neighbouring flow gauge.
- A regional method assessment.
- Comparison with observed floods within the catchment.

2.0 Calibrated Models

Table 1 provides the parameters used within the HEC-HMS model for the baseline design events as derived in Appendix F. The HIRDS temporal pattern has been used for the baseline scenarios.

Table 1: Baseline Scenario HMS Parameters

Catchment	Initial (mm)	Constant (mm/hr)	PRF	Lag Time (minutes)	Initial Discharge ($m^3/s/km^2$)	Baseflow Coefficient (hr)	Baseflow Fraction	Baseflow Steps
Tariki	8	3.5	450	75	0.1	10	1.0	2
Salisbury	8	3.5	450	75	0.1	10	1.0	2
Mangaotea Road	8	3.5	450	75	0.1	10	1.0	2
Lower Mangaotea	8	3.5	450	75	0.1	40	1.0	2
Tunnel 3	8	3.5	450	75	0.1	10	1.0	2
Upper Lake	8	3.5	450	75	0.1	10	1.0	2
Lower Lake	8	3.5	450	75	0.1	10	1.0	2
Tributary 1	8	3.5	450	75	0.1	40	1.0	2
Tributary 2	8	3.5	450	75	0.1	20	1.0	2
Mangaotea	8	3.5	450	120	0.1	60	1.0	2

The HEC-HMS catchment runoff results, prior to any attenuation, are presented within Table 2. The critical duration events are highlighted.

Table 2: Peak HEC-HMS Catchment Runoff Results (m^3/s)

Duration (Hours)	2.33-Year	100-Year	1,000-Year
1	34	98	150
3	47	125	189
6	40	107	160
12	38	98	146
24	22	62	93

The HEC-RAS hydraulic model has been used to estimate the peak runoff inflows into the lower lake (i.e., attenuation effects accounted for). The peak inflow into the lower lake has been derived from the HEC-RAS results as follows:

Lower Lake Inflow = Ratapiko Road Connection Flow
+ Tributary 1 Connection Flow
+ Tributary 2 Connection Flow
+ Lower Lake Local Inflow

The peak HEC-RAS lower lake runoff inflow results are presented within Table 3, with the critical results highlighted. Knowing that the unattenuated inflows have a critical duration in the order of 3 hours, we would expect the attenuated inflows to have at least the same critical duration (likely to be higher). As a result, where the peak HEC-RAS lower lake runoff inflow is still increasing to the 3-hour duration, we have considered this duration critical.

Table 3: Peak HEC-RAS Lower Lake Runoff Inflow Results (m^3/s)

Duration (Hours)	2.33-Year	100-Year	1,000-Year
3	-	73	98
6	27	66	98
12	27	62	94
24	14	-	-

We note that during the critical 3-hour 1,000-year ARI event, 134mm (88%) of rainfall becomes direct runoff (i.e., direct runoff is high relative to the rainfall).

3.0 Flood Frequency Analysis

The Manganui flow gauge is in close proximity to the Ratapiko catchment, and a flood frequency analysis has been undertaken for the gauge. Table 3 summarises the gauge characteristics.

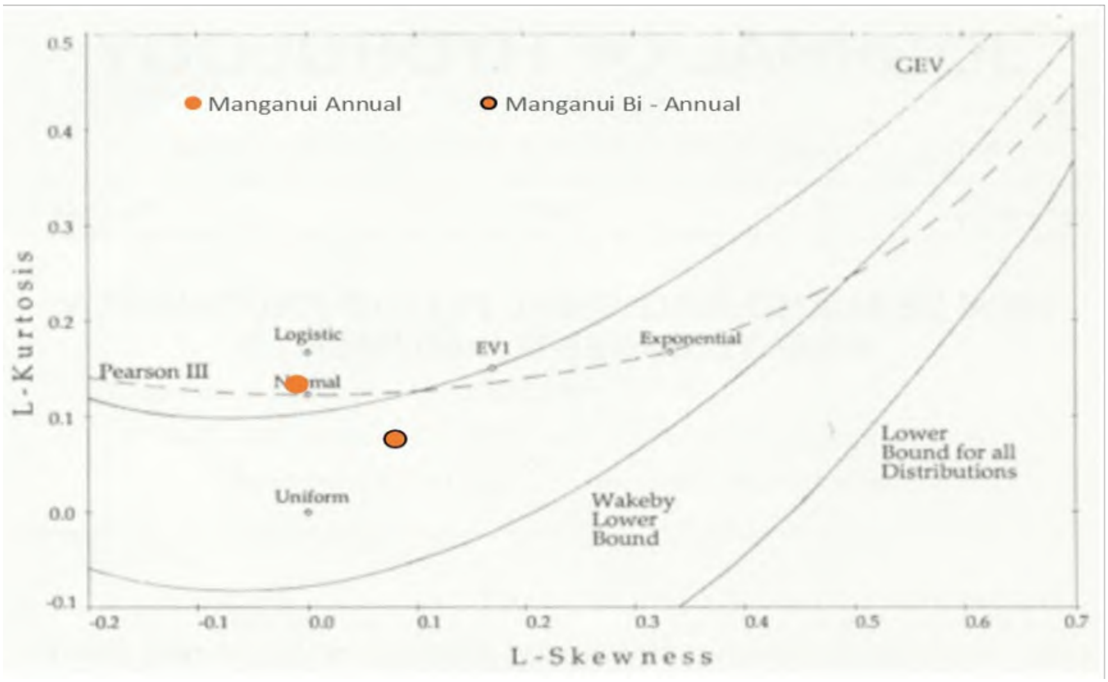
Table 3: Gauge Summary

Site Number	Site Name	Source	Period of Record	Length of Record (Years)	Catchment Area (km^2)
39508	Manganui at SH3	NIWA	May 1972 to 2021 (current)[1.]	48	11.3

Note: 1. 1972 not included within the analysis as a significant flood greater than mean is not included within period of record.

An L-moments analysis was undertaken for the Manganui flow record as shown in Figure 1.

Figure 1: L-moments Analysis

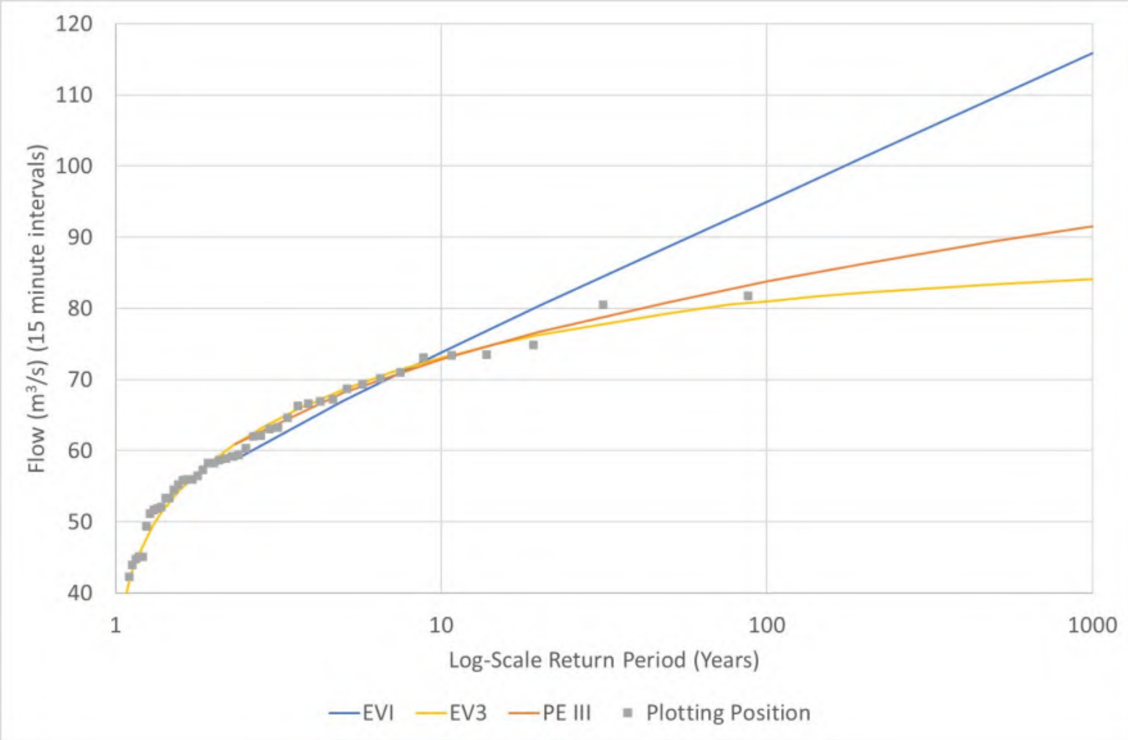


This analysis suggests that the Pearson Type III distribution is likely to be most representative of the Manganui at SH3 annual maxima series. The EV1 distribution is potentially overconservative for the Manganui at SH3 record. The EV3 distribution is likely to be more representative than the EV1. A transformation was applied to the dataset, to consider the effects of a biennial maxima series. This was not found to significantly improve or deplete the fit of the EV1 distribution.

Figure 2 presents the fit of the selected distributions. As expected, the Pearson Type III distribution appears to provide a good match to the Manganui at SH3 annual maxima series.

The downward trending EV3 distribution appears to provide a better match than the EV1 distribution to the Manganui at SH3 annual maxima series, which is comparable to the Pearson Type III distribution. Overall, we consider the Pearson Type III distribution provides the most appropriate distribution.

Figure 2: Annual Partition – Manganui Flow Gauge



The results of the analysis are presented in Table 4. The flows have been factored to Lake Ratapiko catchment area of 12.38km^2 (excluding the Mangaotea catchment).

Table 4: Flood Frequency Analysis Results – Flow (m^3/s)

Flood Event	Manganui (Pearson Type III)	Ratapiko[1] (Pearson Type III)
Mean Annual Flood ($Q_{2.33}$)	61	66
100-year Flood (Q_{100})	84	90
1,000-Year (Q_{100})	92	99

Note: 1. Manganui flows have been factored by $(12.38/11.3)^{0.8}$ to obtain Ratapiko Flows

The Manganui catchment extends to the upper regions of Mount Taranaki (some 2,500m above sea level). In contrast, the Ratapiko catchment extends to an elevation of around RL 240m and has significant storage. We consider the above analysis is conservative when applied to the Ratapiko catchment because we would expect the runoff per unit area would be less for the Ratapiko catchment due to storage effects, and the much flatter catchment and lower rainfall intensity.

4.0 Regional Methods

A regional method is detailed within Flood Frequency in New Zealand (McKerchar and Pearson, 1989)[1]. The method essentially provides contours over New Zealand to determine the mean annual flood and subsequently the 100-year flood.

1 Flood Frequency in New Zealand (McKerchar and Pearson, Publication No. 20 of the Hydrology Centre, 1989)

The McKerchar and Pearson method results in a wide range of possible mean annual flood results. Ryder (2010) used a $Q_{2.33}/A^{0.8}$ ratio of 3 to 4, with a catchment area of 9.1km^2 to determine a mean annual flood range of 18m^3/s to 23m^3/s. We consider a ratio of 4.0 is appropriate, however, we consider a catchment area of 12.38km^2 is more appropriate (i.e., excluding the Mangaotea catchment area) to allow a comparison of unattenuated catchment runoff. Using the McKerchar and Pearson method, we consider the mean annual flood is approximately 30m^3/s.

The Manganui gauge was included in the McKerchar and Pearson methodology. At the time, only 14 years of data was available compared to the 45 years available now. Although a $Q_{100}/Q_{2.33}$ ratio of 1.45 was calculated by McKerchar and Pearson, a ratio of 2.0 was recommended when results from other catchments were considered. We note that Ryder (2010) used a $Q_{100}/Q_{2.33}$ ratio of 2.0 using the McKerchar and Pearson method, and the ratio from the presented results was 2.1 (i.e., 51/24). Overall, we consider that a ratio of 2.0 is the most appropriate ratio to use for the McKerchar and Pearson method.

NIWA also has an online application (New Zealand River Flood Statistics) which provides flood estimates for various catchments within New Zealand up to the 1000-year event. The NIWA method estimates a mean annual flood of 11m^3/s and a 100-year flood of 23m^3/s for a catchment area of 9.24km^2 to the dam (i.e., excludes the canal catchment), with a $Q_{100}/Q_{2.33}$ ratio of 2.14. Table 5 summarises the McKerchar and Pearson and NIWA methods and provides a comparison to the flood frequency analysis.

We consider that the unattenuated peak flows from the calibrated model provide conservative results, as also presented within Table 5. As discussed previously, the peak inflow to the lake from the canal and Mangaotea catchments, is constrained by the canal capacity of approximately 12m^3/s (varies with lake level).

The largest flood on record at the Manganui flow gauge is the 2004 flood (1973 to 2020 inclusive).

Table 5: Regional Method Summary - Flow (m^3/s)

Item	Manganui Flood Frequency[1.] (Pearson Type III)	McKerchar and Pearson	NIWA	HEC-HMS Calibrated Model (Unattenuated)
Mean Annual Flood ($Q_{2.33}$)	66	30	14	47
Ratio ($Q_{100}/Q_{2.33}$)	1.4	2.0	2.1	2.7
100-Year Flood (Q_{100})	90	60	29	125

Note: 1. Manganui flows have been factored by $(12.38/11.3)^{0.8}$
2. NIWA flows have been factored by $(12.38/9.24)^{0.8}$

5.0 Observed Events Prior to 2001

We have reviewed historical information to source any information regarding flooding at the dam prior to 2001, when continuous lake level records began.

We note that a 1968 report (Engineering Report, Beca, North and MacDonald Consultants) states that in a recent flood, the service spillway was overtopped by 500mm. No other information was found.

6.0 Discussion

Table 6 presents a summary of the flood estimates. The selected estimates for the purposes of deriving the IDF are those from the HEC-RAS Calibrated Model (Attenuated). We believe the results from this model best represents the catchment and reservoir response.

Table 6: Summary of Flood Estimates

Flood Event	Manganui Flood Frequency[1.] (Pearson Type III)	McKerchar and Pearson	NIWA[2.]	HEC-HMS Calibrated Model (Unattenuated)	HEC-RAS Calibrated Model (Attenuated)
Mean Annual Flood ($Q_{2.33}$)	66	30	14	47	27
100-Year Flood (Q_{100})	90	60	29	125	73
1,000-Year Flood (Q_{100})	99	-	-	189	98

Note: 1. Manganui flows have been factored by $(12.38/11.3)^{0.8}$
2. NIWA flows have been factored by $(12.38/9.24)^{0.8}$

We note that the flood frequency analysis for Manganui River indicates that the mean annual flood for Lake Ratapiko is 66m^3/s, which is greater than the actual mean annual flood derived from the model. We therefore consider any comparison to the Manganui flood frequency analysis should be made with caution.

Table 7 presents a comparison of peak lower lake inflows with previous assessments.

Table 7: Lower Lake Peak Inflows - Comparison with Previous Assessments

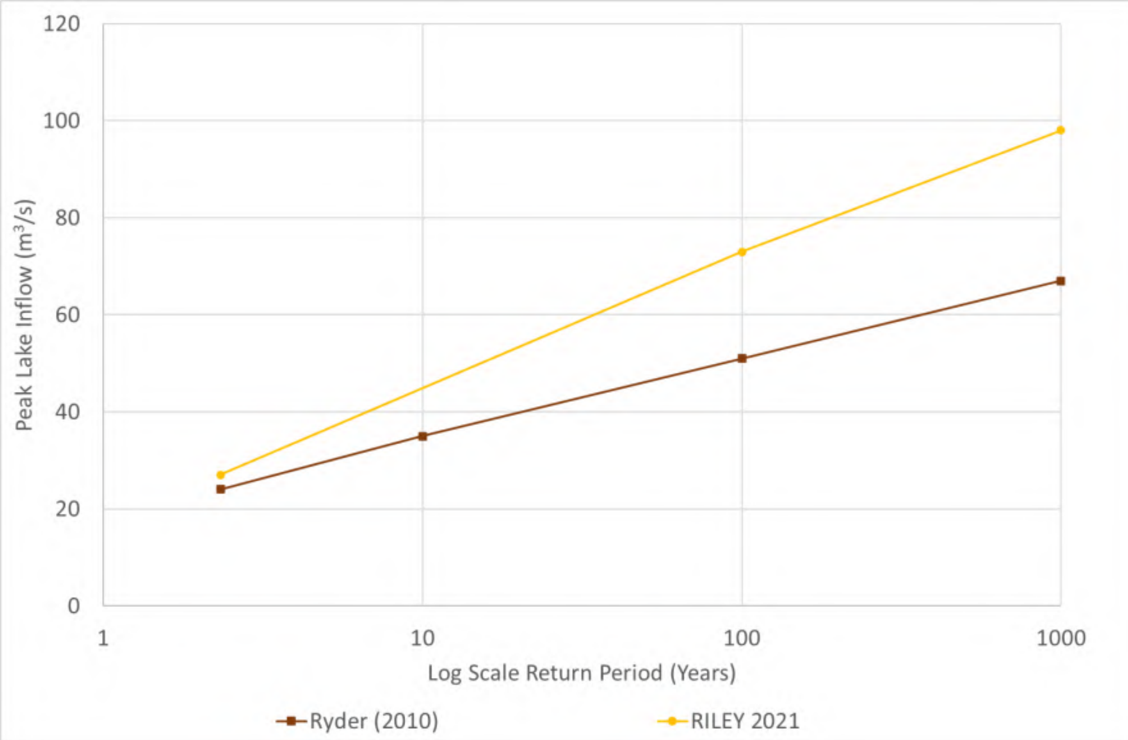
Flood Event	T&T (1994)	T&T (1999)	Ryder (2010)	RILEY (Attenuated)
Mean Annual Flood ($Q_{2.33}$)	13	-	24	27
100-Year Flood (Q_{100})	37	48	51	73
1,000-Year Flood (Q_{100})	-	-	67	98

Overall, we consider that a 1,000-year flood estimate of 98m^3/s derived from the calibration models for the Lake Ratapiko IDF is appropriately conservative. The peak flow is higher than previous estimates, which is due to more conservative loss values being used and the inclusion of the upstream culverts/embankments within the model have resulted in higher flood estimates for extreme flood events.

We consider the calibrated models provide an appropriately conservative estimate of the mean annual flood. The effective $Q_{100}/Q_{2.33}$ ratio is 2.7, which is also appears appropriately conservative compared to the ratios from the regional methods.

We note that there is a large range of uncertainty when predicting extreme return period floods. Figure 3 presents a summary of the design floods compared to the previous Ryder assessment.

Figure 3: Lower Lake Peak Inflows - Design Flood Summary



APPENDIX I

Canal Erosion Assessment

Race Erosion Assessment

This Appendix provides physical information on each section of the Race, including observations during inspections.

Section 1 – Manganui River to Tariki Weir

Section 1 is approximately 725m long and extends from the Manganui River to Tariki Weir. The section is predominantly concrete lined (apart from one section shown in Photograph 1 and the Sediment Pond). We did not observe any significant existing erosion issues during the June 2020 inspection.

Photograph 1: Intake screens, with concrete lined Race downstream (June 2020).

Photograph 2: View from CH150 upstream with the intake gates visible (June 2020).

Section 2 – Tariki Weir to End of Concrete Lining at CH 1075

Section 2 is approximately 350m long and extends from the Tariki Weir (Chainage (CH) 725) to the end of the concrete lining at CH 1075. The section appears to be concrete lined along its entire length. We did not observe any significant existing erosion issues during the June 2020 inspection.

Photograph 3: View from CH 1075 upstream (June 2020).

Section 3 – CH 1075 to CH 1600

Section 3 is approximately 525m long and extends from the end of the concrete lining at CH 1075 to the end of the steep section at CH 1600. This section has a relatively steep longitudinal grade (including a drop structure) and has variable geometry, lining and flow conditions/velocities. During the June 2020 inspection no significant slumping or erosion was identified.

Photograph 4: View from CH 1075 (end of concrete lining) downstream (June 2020). Note the velocities reducing downstream.

Photograph 5: View from CH 1225 upstream (June 2020). Note the low velocities and the mass concrete block repair.

Photograph 6: View from Salisbury Road Culvert CH 1325 upstream (June 2020). Note the drop structure and the higher velocities downstream of this (concrete lined).

Photograph 7: View from Salisbury Road Culvert CH 1325 downstream (June 2020). Note the rapids further downstream (riprap lined).

Section 4 – CH 1600 to In-Race Generator (CH 2225)

Section 4 is approximately 625m long and extends from the end of the steep section at CH 1600 to the In-Race generator. Velocities are generally low.

Photograph 8: Tunnel 1 Entrance. Note the low velocities and recent instability downstream of the mass concrete blocks.

Photograph 9: View upstream from In-Race Generator. Note the low velocities.

Section 5 – In-Race Generator to Upper Mangaotea Bridge (CH 2550)

Section 5 is approximately 325m long and extends from the In-Race Generator to the Upper Mangaotea Bridge. We did not observe any significant existing erosion issues during the June 2020 inspection, however the uneven appearance of the Race banks in the reach downstream of the In-Race Generator (refer Photograph 15) indicates that some historical erosion may have occurred through this reach. Some mass concrete block remedial works have also been undertaken in the past further downstream.

Photograph 10: View downstream from In-Race Generator (June 2020). Note the uneven appearance of the Race banks. The Upper Mangaotea Bridge is not visible (located around a left-hand bend).

Photograph 11: View upstream from the Upper Mangaotea Bridge (June 2020). Velocities of up to 2.0 m/s were estimated during the inspection.

During the June 2020 inspection, we observed velocities of up to 2.0m/s immediately upstream of the bridge.

Section 6 – Upper Mangaotea Bridge to Mangaotea Aqueduct

Section 6 is approximately 300m long and extends from the Upper Mangaotea Bridge to the Mangaotea Aqueduct. The section is located within the broad Mangaotea floodplain and is cut within soft, highly organic material prone to slumping due to frequent variations in water levels (i.e. independent of velocities). Water levels can also be affected by lake levels within this section. Observed flow velocities are less than 1.0m/s. Site personnel confirmed that within this section, slumping does not appear to occur due to the low water velocity.

Tree stumps are visible when the Race is dewatered. Site personnel have observed that, in places, these tree stumps cause eddies to occur which can also lead to erosion.

Through this section, extensive erosion remedial works have been undertaken over recent years. The repairs consist of mass concrete blocks and a timber retaining wall section. We understand the retaining wall was constructed instead of the mass concrete blocks due to settlement issues relating to the low strength of the organic founding material. To date, the remedial works appear to be performing adequately.

Photograph 12: Recent mass concrete block remedial works (June 2020). Note the relatively low velocities within the Race.

Photograph 13: Recent timber retaining wall remedial works (June 2020). Note the relatively low velocities within the Race and the low lowing nature of the surrounding floodplain. Upper Mangaotea Bridge is beyond the upstream extent of the wall.

Section 7 – Mangaotea Aqueduct CH 2850- CH 3000

This section was not inspected during the June 2020 inspection; however, we understand from Trustpower Site Personnel that existing erosion remediation works consisting of mass concrete blocks are performing adequately.

Photograph 14: View downstream from Mangaotea Aqueduct. Aqueduct is a concrete structure. Mass concrete blocks located on true right bank immediately downstream (February 2019 – low flows).

Section 8 – Lower Race CH 3000 to Lake

This section is over 1600m long and extends along the remainder of the Race to the lake.

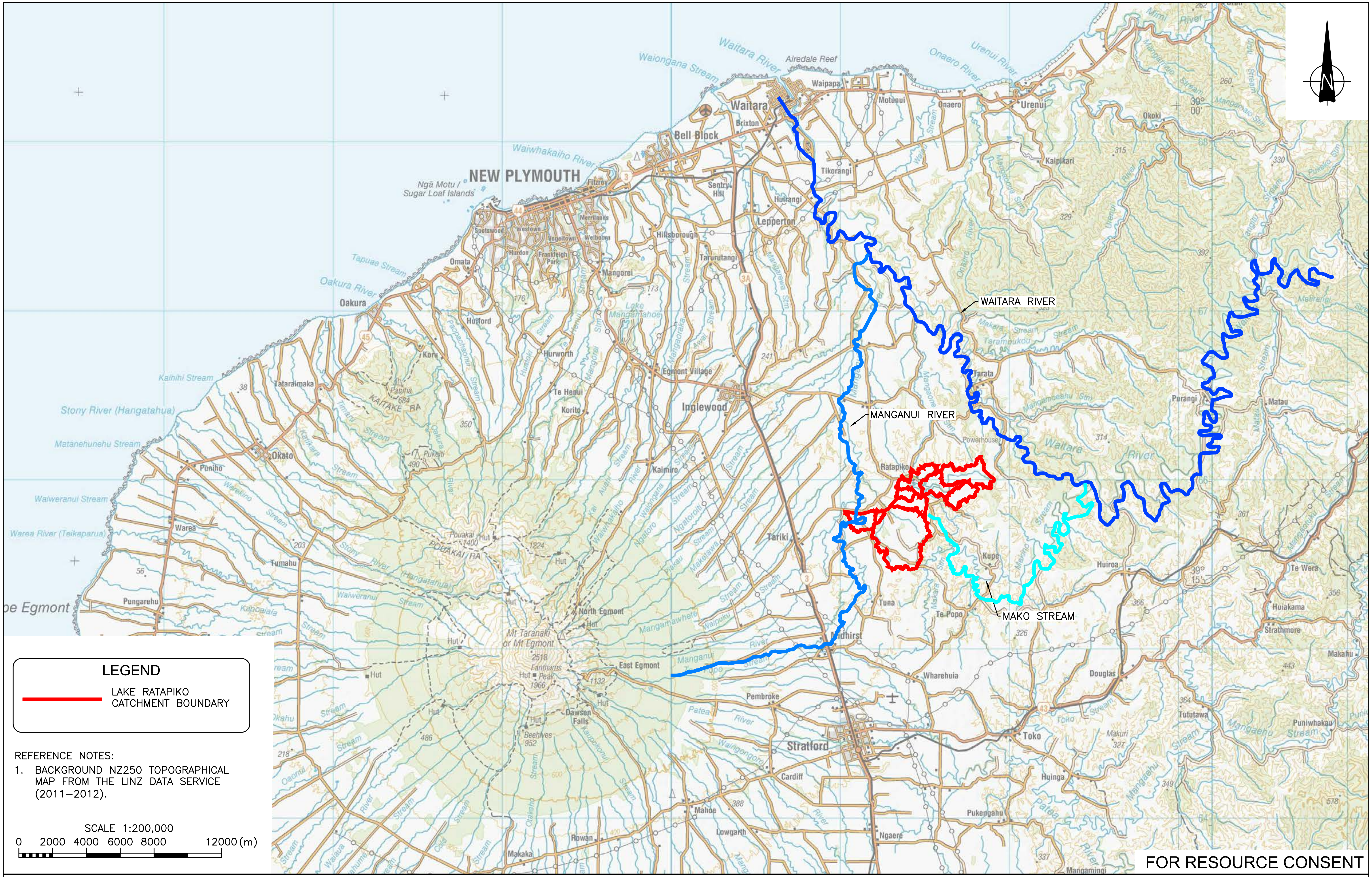
Photograph 15: View upstream from Lower Mangaotea Bridge (June 2020). Note the low velocities.

Photograph 16: View upstream from Tunnel 2 (June 2020). Note the low velocities and the well vegetated banks.

Photograph 17: View downstream from Tunnel 2 (June 2020).

APPENDIX J

Hydraulic Model Result Maps



LEGEND

LAKE RATAPIKO CATCHMENT BOUNDARY

REFERENCE NOTES:

1. BACKGROUND NZ250 TOPOGRAPHICAL MAP FROM THE LINZ DATA SERVICE (2011–2012).

SCALE 1:200,000

0 2000 4000 6000 8000 12000 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
MAY 2021		15 / 11 / 21

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	SCHEME LOCATION - REGIONAL CONTEXT

ACENZ
ISO 9001 CERTIFIED GCS

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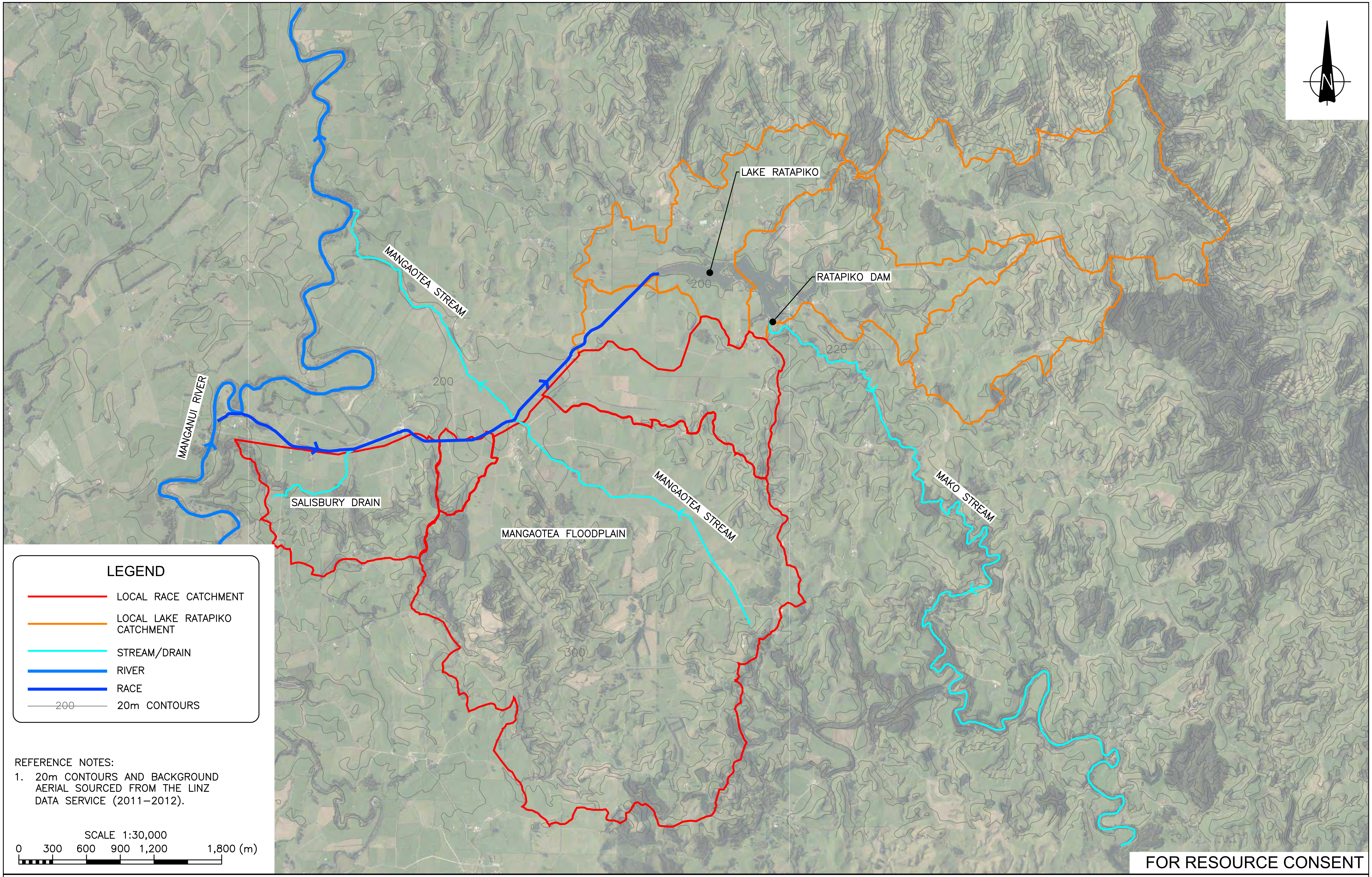

LEGEND

- LOCAL RACE CATCHMENT
- LOCAL LAKE RATAPIKO CATCHMENT
- STREAM/DRAIN
- RIVER
- RACE
- 200 20m CONTOURS

REFERENCE NOTES:

1. 20m CONTOURS AND BACKGROUND AERIAL SOURCED FROM THE LINZ DATA SERVICE (2011–2012).

SCALE 1:30,000

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FOR RESOURCE CONSENT

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: JUNE 2021		ISSUE DATE: 15 / 11 / 21

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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	LANDFORM AND ASSOCIATED DRAINAGE SYSTEM

ACENZ
ISO 9001 CERTIFIED GCS

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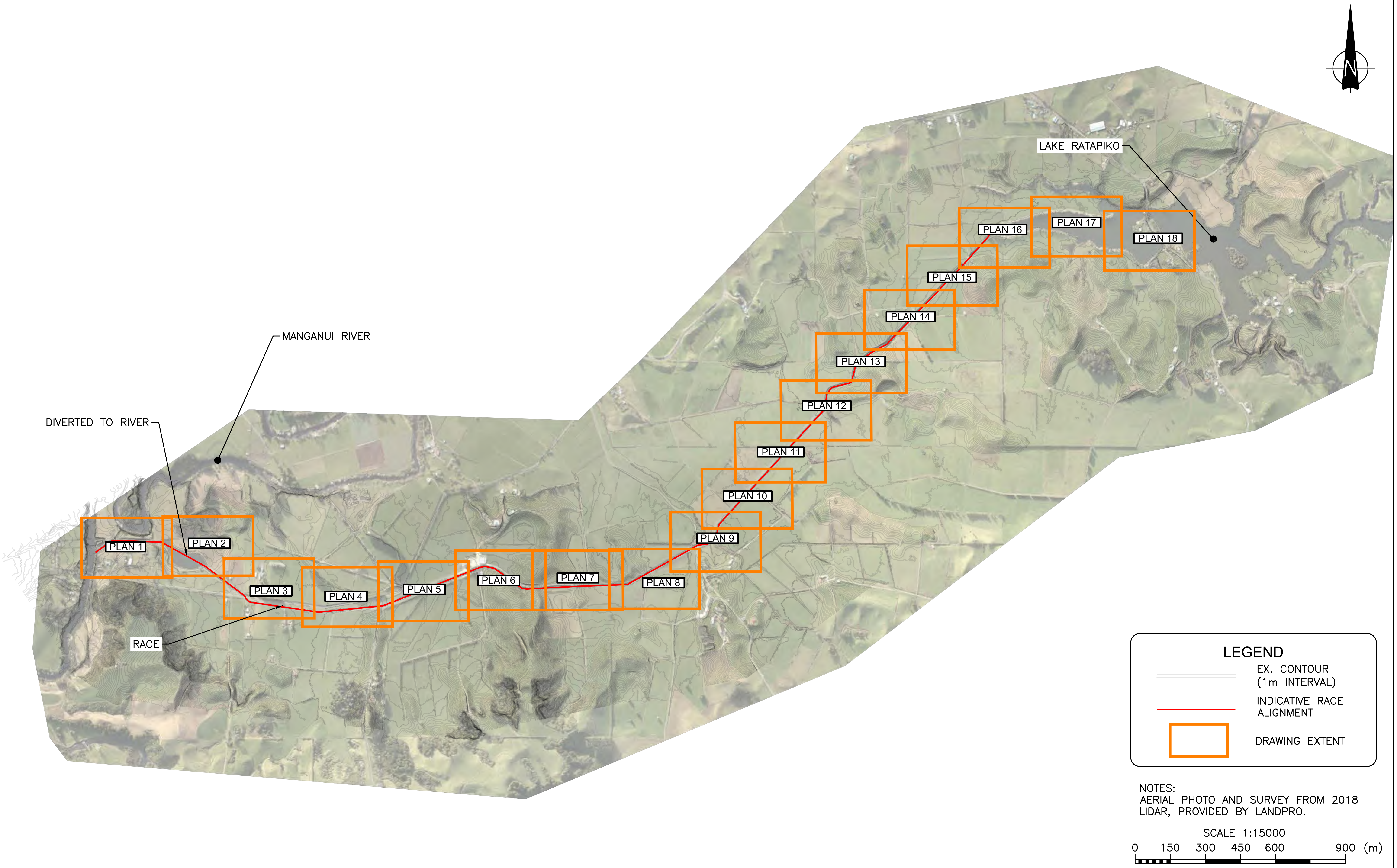
DRAWING No.	18MTK_ENH-202
REV.	1



NOTES:
AERIAL PHOTO AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
MARCH 2019		15 / 11 / 21

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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: OVERALL SITE PLAN

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–209.dwg
SCALE (A3): 1:15000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-209
REV.: 1

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 1

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-210
REV.: 1

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN: OOBG | CAD CHECK: FY
DATE DRAWN: AUG 2021 | ISSUE DATE: 15 / 11 / 21

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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 2

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ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000 | ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-211 | REV.: 1

LEGEND

WATER VELOCITY (m/s)
- 0–1
- 1–2
- 2–3
- 3–4
- 4+

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 3

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-212
REV.: 1

LEGEND

- 0–1, 1–2, 2–3, 3–4, 4+: WATER VELOCITY (m/s)
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

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REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM | DES CHECK: VM | DRAWN: OOBG | CAD CHECK: FY | DATE DRAWN: AUG 2021 | APPROVED FOR ISSUE: P LILLEY | ISSUE DATE: 15 / 11 / 21

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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 4

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000 | ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-213 | REV.: 1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT TRUSTPOWER LTD
ADDRESS MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 5

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE 18MTK_ENH–210to227.dwg
SCALE (A3) 1:1000
ORIG. SHEET SIZE A3
DRAWING No. 18MTK_ENH-214
REV. 1

LEGEND

	0–1	WATER VELOCITY (m/s)
	1–2	
	2–3	
	3–4	
	4+	
■	WATER LEVEL INSTRUMENT	
	EX. CONTOUR(1m INTERVAL)	
	HEC–RAS MODEL EXTENT	
	CANAL CENTRE LINE	
	HYDRO BREAKLINES	
	POTENTIAL DISCHARGE FROM LAND INTO RACE	
	FLOW DIRECTION	

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

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DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 6

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–210to227.dwg
SCALE (A3) 1:1000	ORIG. SHEET SIZE A3
DRAWING No. 18MTK_ENH-215	REV. 1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 7

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-216
REV.: 1

LEGEND

0–1 WATER VELOCITY (m/s)
1–2
2–3
3–4
4+

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

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OOBG	FY	
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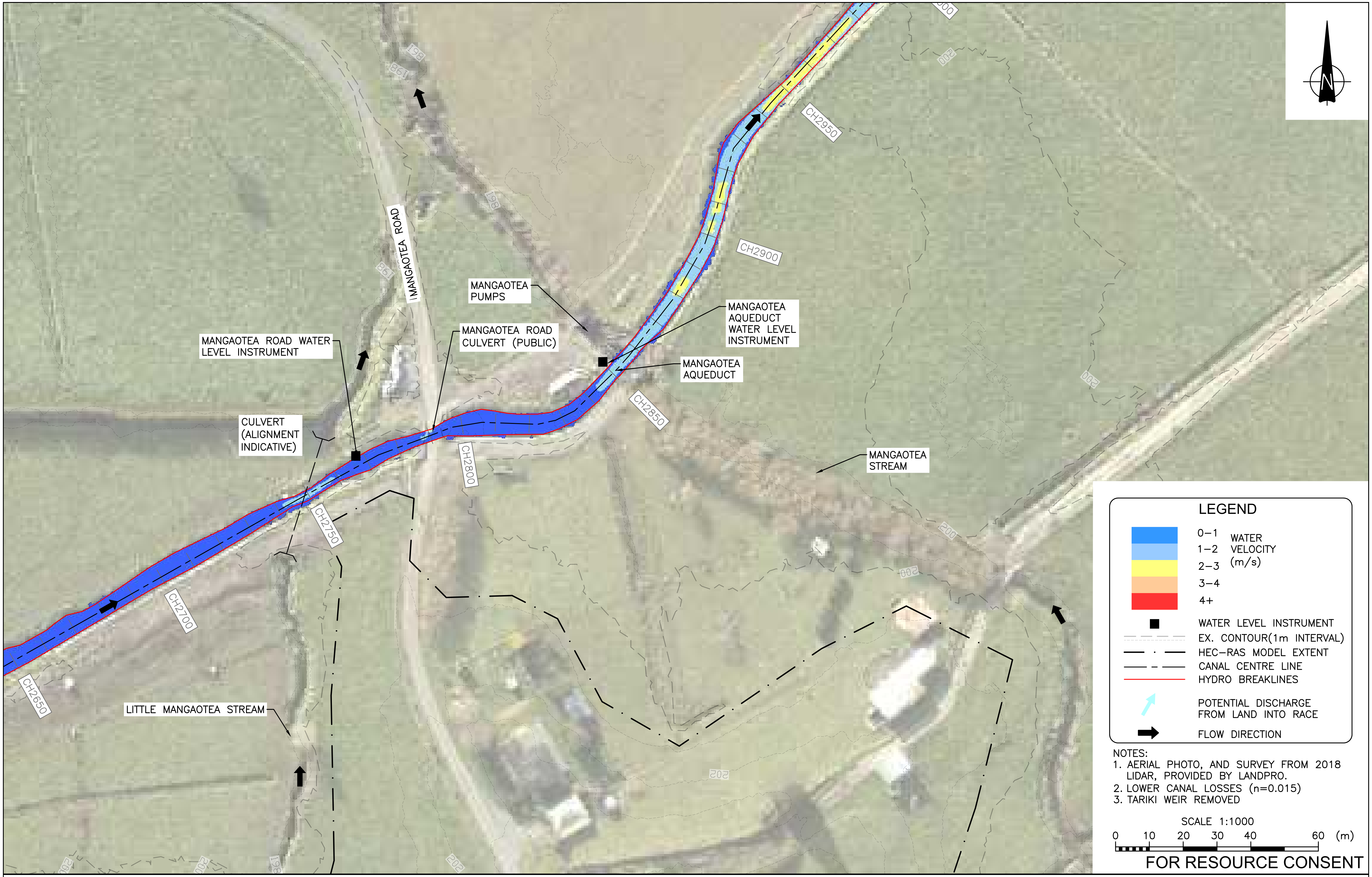
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 8

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-217
REV.: 1



LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	
■	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 9

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CADFILE	
18MTK_ENH–210to227.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-218	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 10

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ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-219
REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN	CAD CHECK	
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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 11

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–210to227.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-220	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 12

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000 | ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-221 | REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 13

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CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-222
REV.: 1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 14

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ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-223	REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)	
1–2		
2–3		
3–4		
4+		

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 15

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–210to227.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-224	1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 16

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–210to227.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-225
REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)	
1–2		
2–3		
3–4		
4+		

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 17

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–210to227.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-226	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	
■	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 5.2m³/s ABSTRACTION - VELOCITIES - PLAN 18

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–210to227.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-227	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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Trust power

CLIENT: TRUSTPOWER LTD

ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME

PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT

SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 1

ACENZ

ISO 9001 CERTIFIED GCS

CADFILE		
18MTK_ENH–230to247.dwg		
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3	
DRAWING No.: 18MTK_ENH-230		REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 2

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-231	REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 3

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–230to247.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-232	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	FY	
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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 4

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CADFILE	
18MTK_ENH–230to247.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-233	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN	CAD CHECK	
OOBG	FY	
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CLIENT	TRUSTPOWER LTD
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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 5

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-234
REV.	1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 6

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-235
REV.: 1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 7

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-236
REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
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OOBG	FY	
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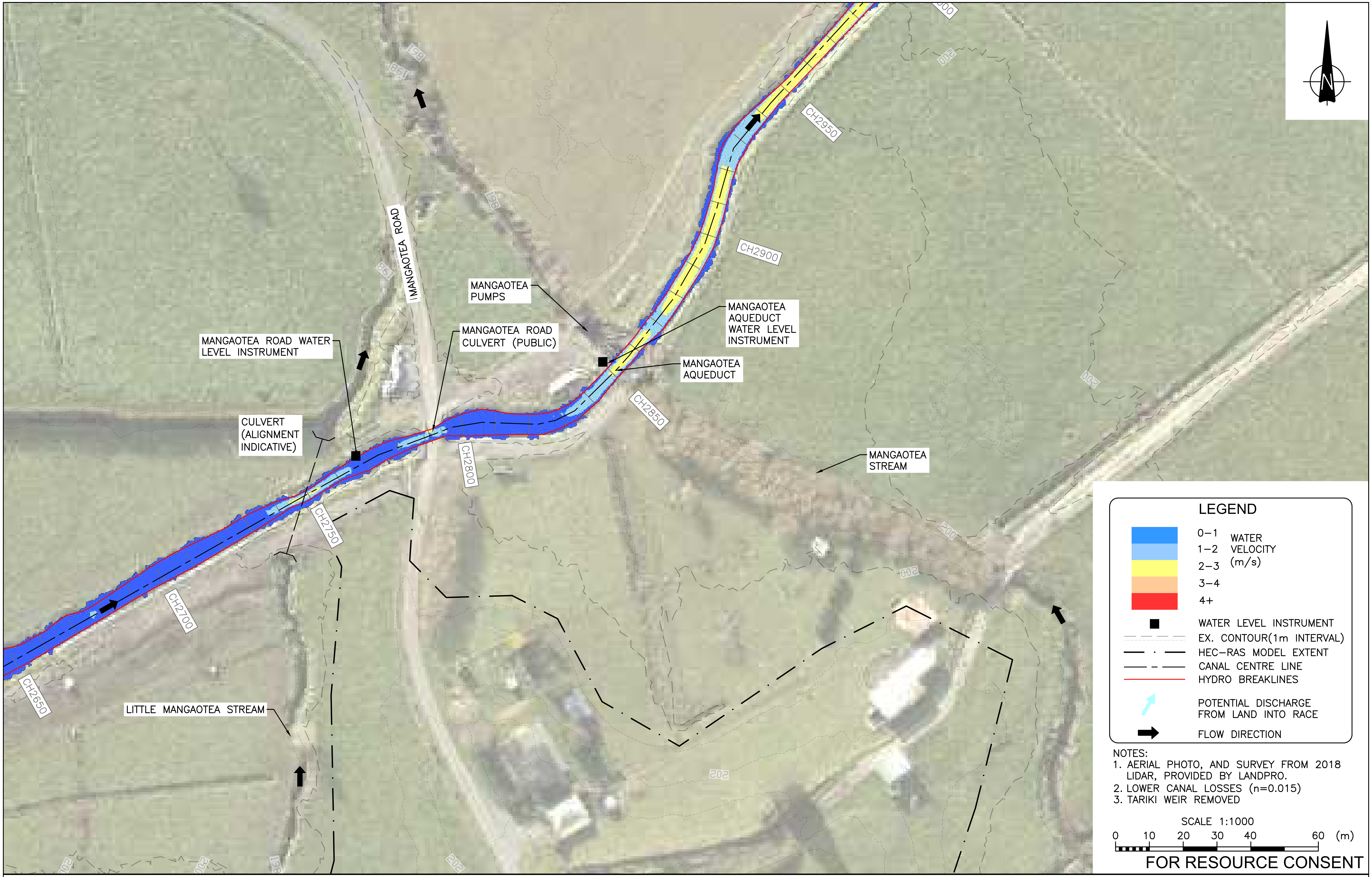
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 8

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-237
REV.: 1



NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

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DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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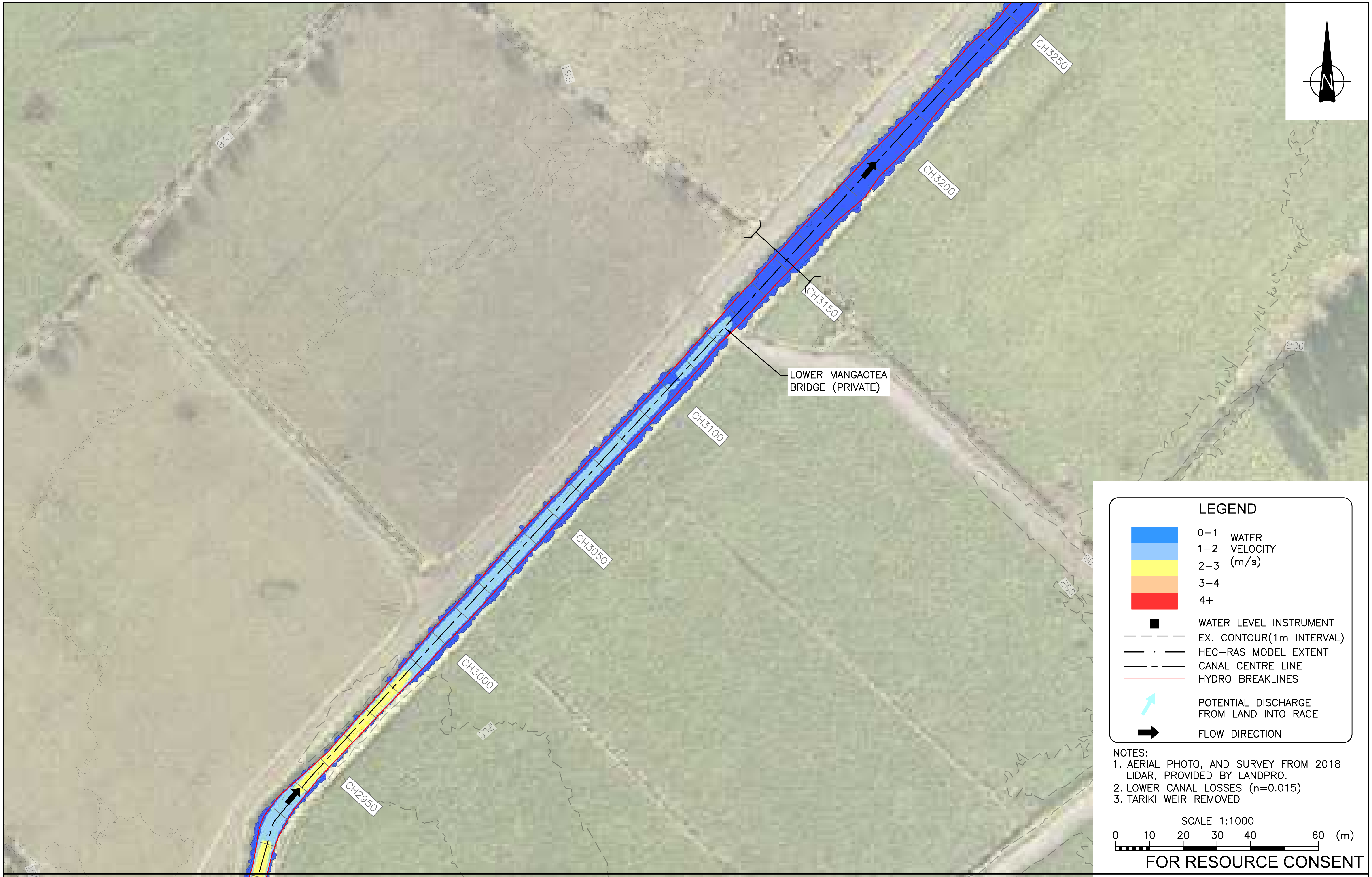
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 9

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-238
REV.	1



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REV	DATE	ISSUE	BY
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DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
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DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 10

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-239	REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 11

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-240	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 12

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–230to247.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-241
REV.	1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 13

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-242
REV.: 1

LEGEND

WATER VELOCITY (m/s)
- 0–1
- 1–2
- 2–3
- 3–4
- 4+

- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN	CAD CHECK	
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 14

ACENZ

ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–230to247.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-243	1

LEGEND

0–1	WATER VELOCITY (m/s)
1–2	
2–3	
3–4	
4+	

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

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PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 15

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-244
REV.: 1

LEGEND

0–1	WATER VELOCITY (m/s)	
1–2		
2–3		
3–4		
4+		

WATER LEVEL INSTRUMENT
EX. CONTOUR(1m INTERVAL)
HEC–RAS MODEL EXTENT
CANAL CENTRE LINE
HYDRO BREAKLINES
POTENTIAL DISCHARGE FROM LAND INTO RACE
FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN: OOBG	CAD CHECK: FY	
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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 16

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-245
REV.: 1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 17

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-246
REV.: 1

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: NORMAL OPERATING CONDITIONS 7.5m³/s ABSTRACTION - VELOCITIES - PLAN 18

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–230to247.dwg	
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-247	REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 3

ACENZ

ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-252
REV.	1

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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Trust power

CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 4

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–250to267.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-253
REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY	
DRAWN: OOBG	CAD CHECK: FY		
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21	

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Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 5

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-254
REV.	1

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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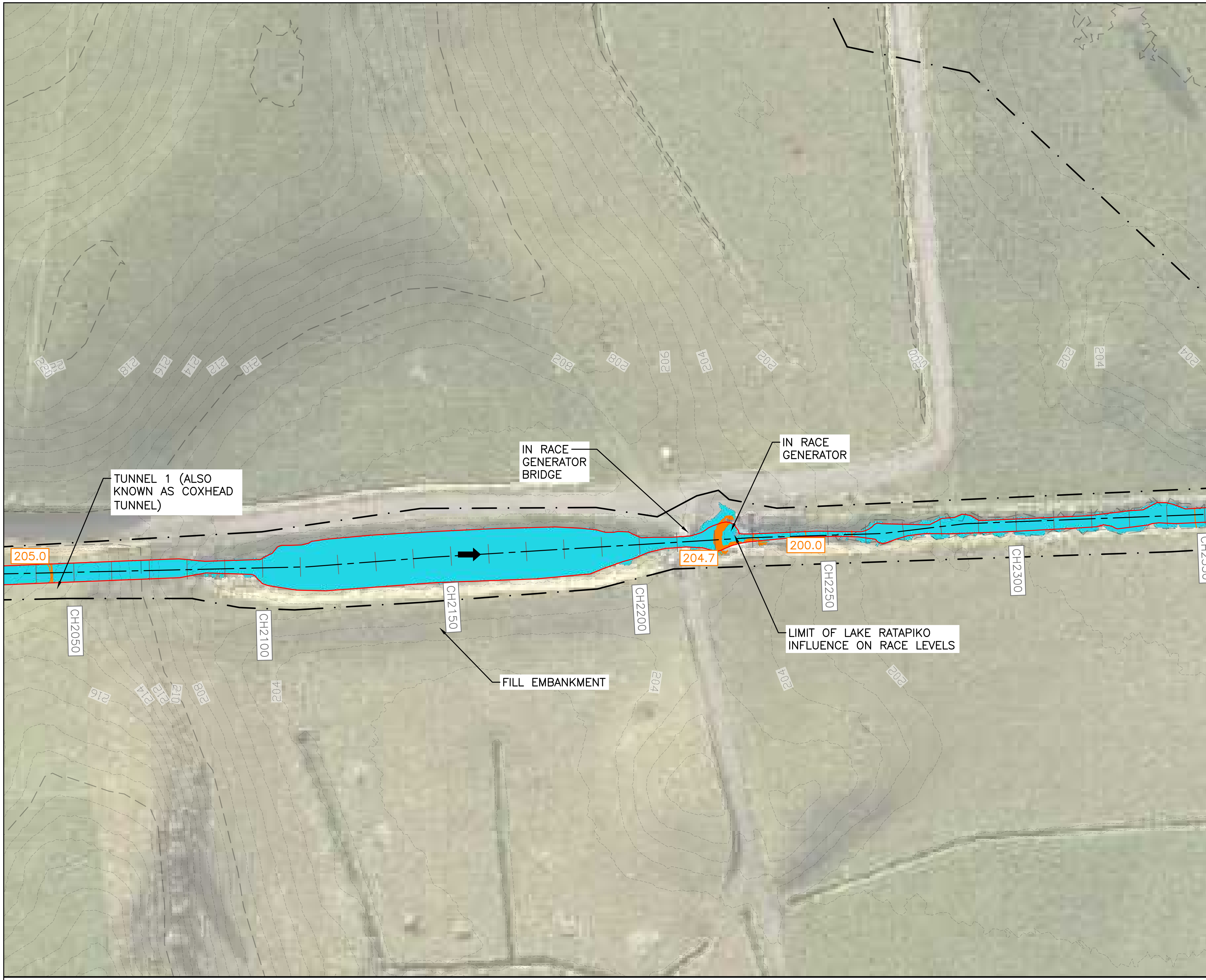
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 6

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-255	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
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CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 7

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–250to267.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-256
REV.: 1

LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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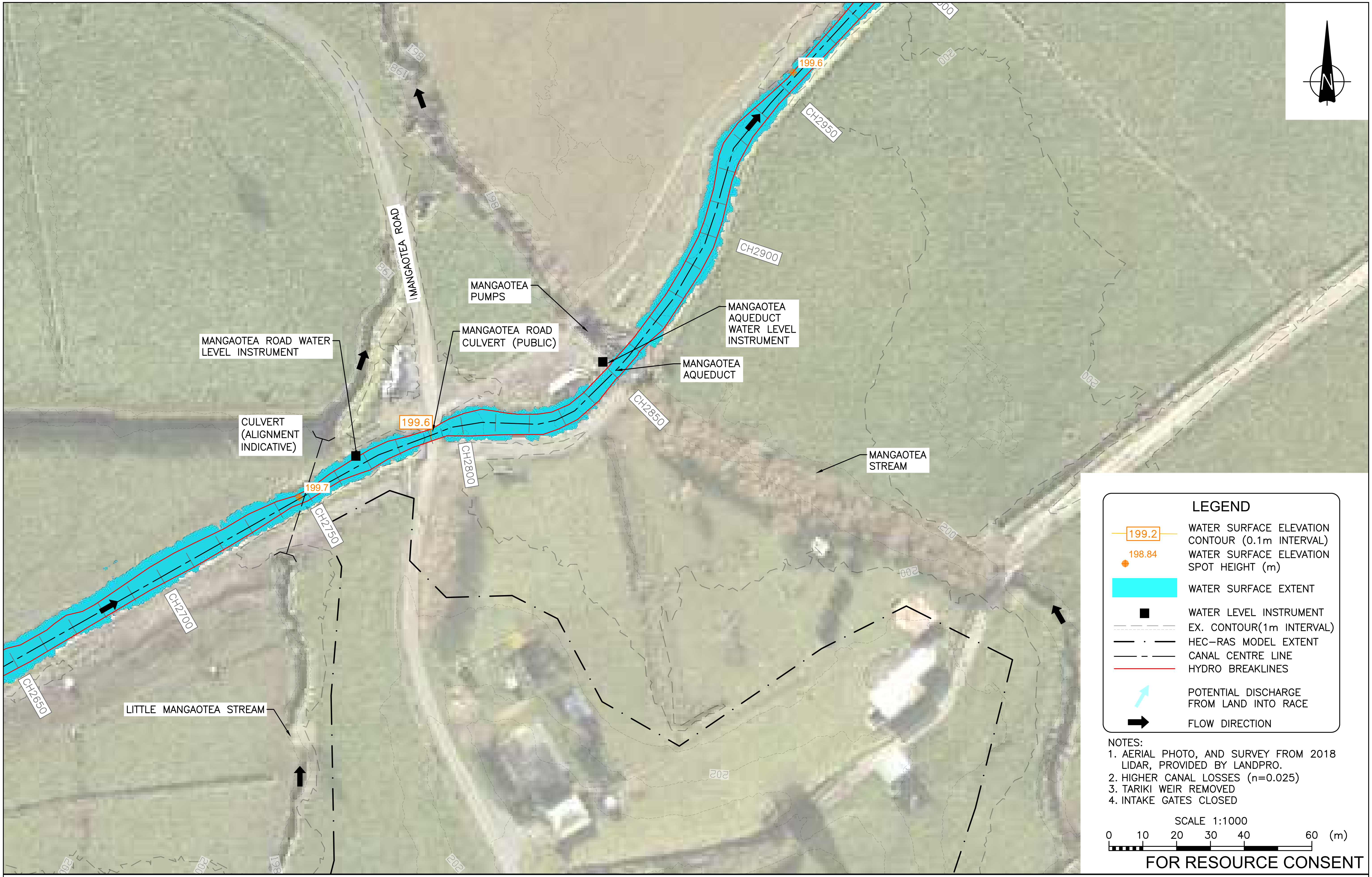
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 8

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-257
REV.	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 9

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-258
REV.	1

LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 10

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-259	1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC-RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	FY	
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 11

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH-250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-260
REV.	1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	VM
DES CHECK	VM
DRAWN	OOBG
CAD CHECK	FY
APPROVED FOR ISSUE	P LILLEY
DATE DRAWN	AUG 2021
ISSUE DATE	15 / 11 / 21

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Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 12

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-261
REV.	1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 13

ACENZ

ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-262	REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000
0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	VM
DES CHECK	VM
DRAWN	OOBG
CAD CHECK	FY
DATE DRAWN	AUG 2021
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 14

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-263
REV.	1

LEGEND

- 199.2 WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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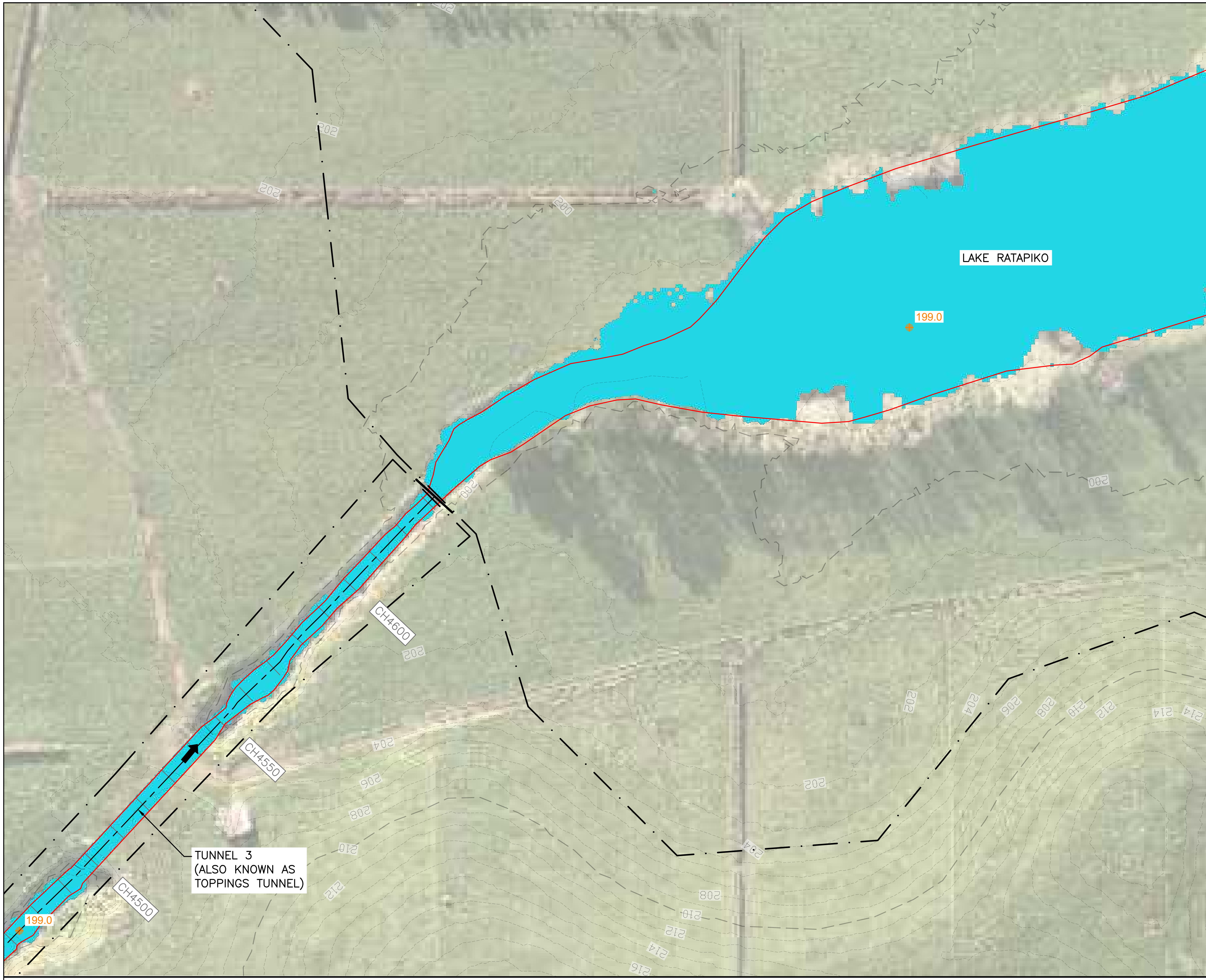
CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 15

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH–250to267.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-264
REV.: 1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:

1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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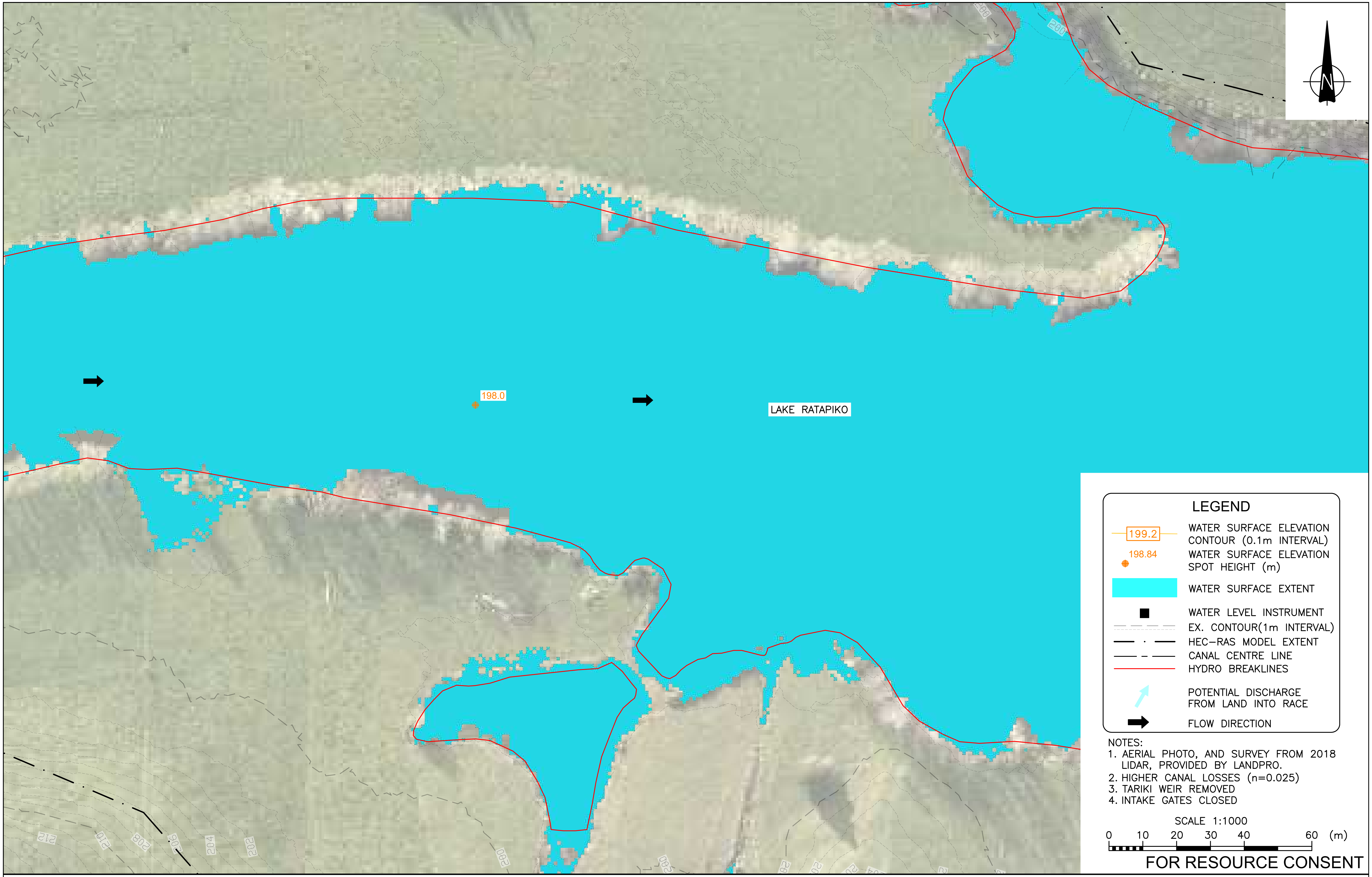
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 16

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-265
REV.	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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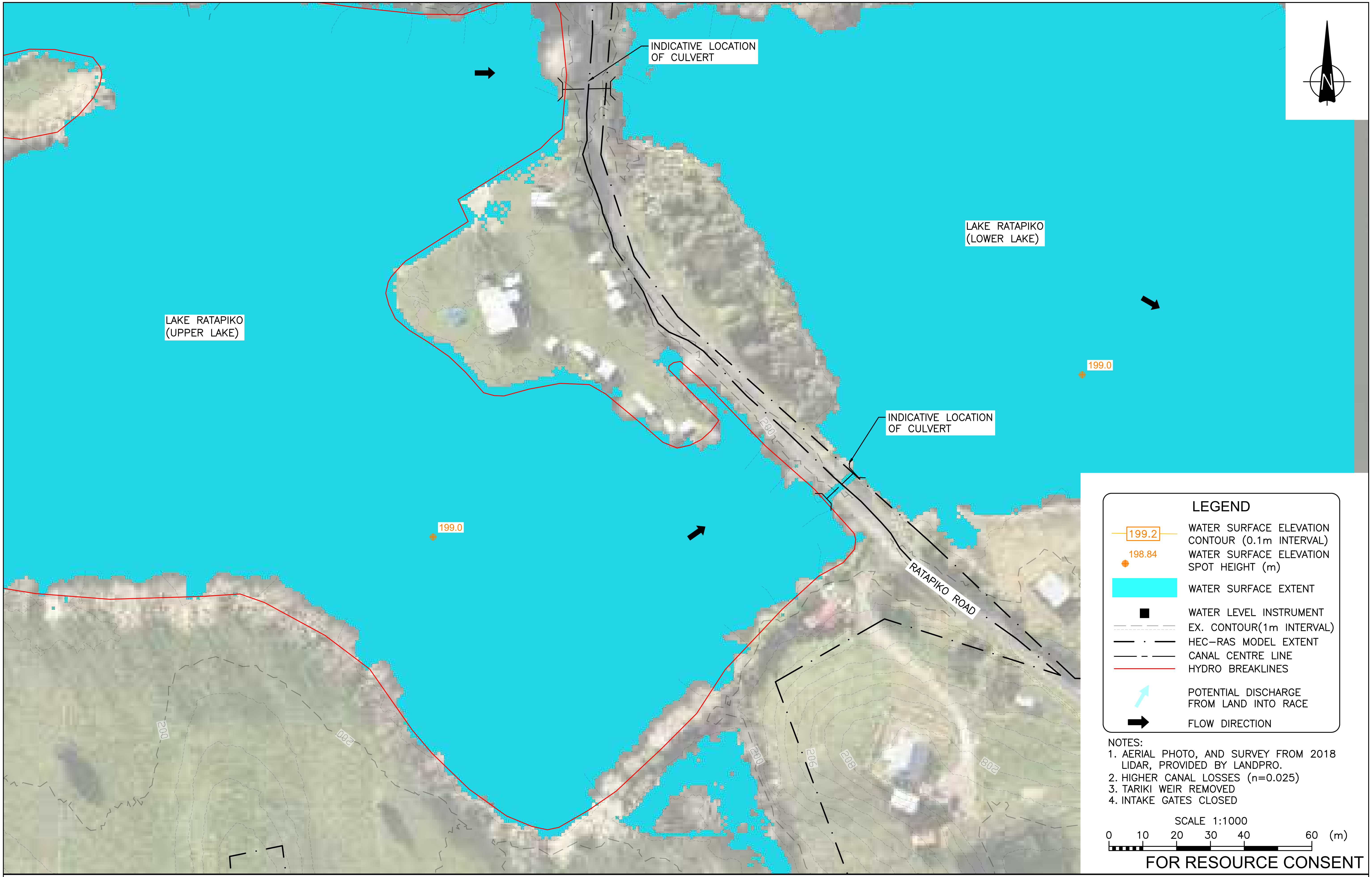
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 17

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–250to267.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-266	REV.: 1



FOR RESOURCE CONSENT

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	MEAN ANNUAL FLOOD EVENT - LEVELS - PLAN 18

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH-250to267.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-267	1

LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC-RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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Trust power.

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 3

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE		
18MTK_ENH-270to287.dwg		
SCALE (A3)	ORIG. SHEET SIZE	
1:1000	A3	
DRAWING No.		REV.
18MTK_ENH-272		1

LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 4

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–270to287.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-273	1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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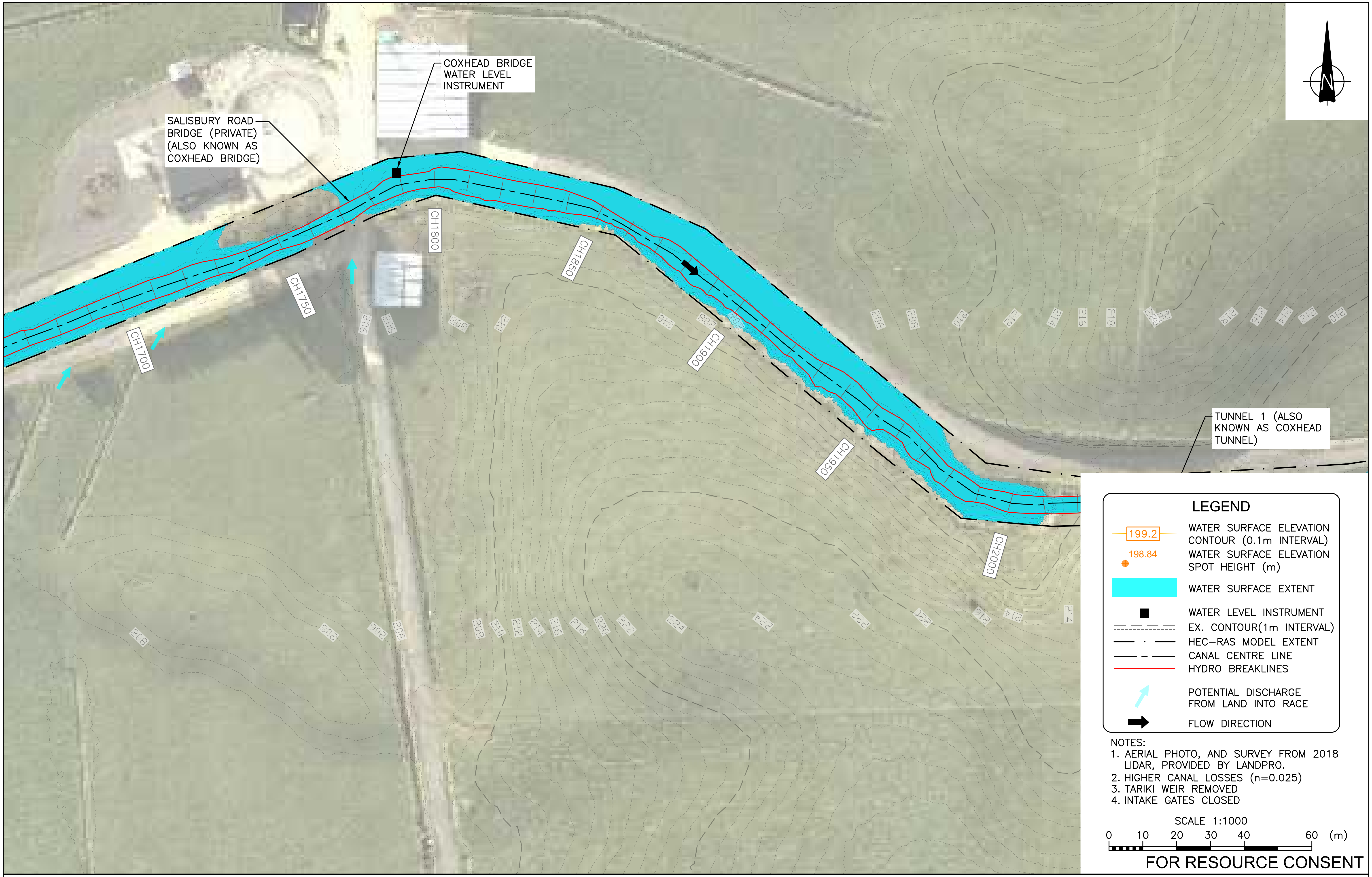
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 5

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-274	1



FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 6

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CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-275	REV.: 1

LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 18 / 10 / 21

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 7

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-276	REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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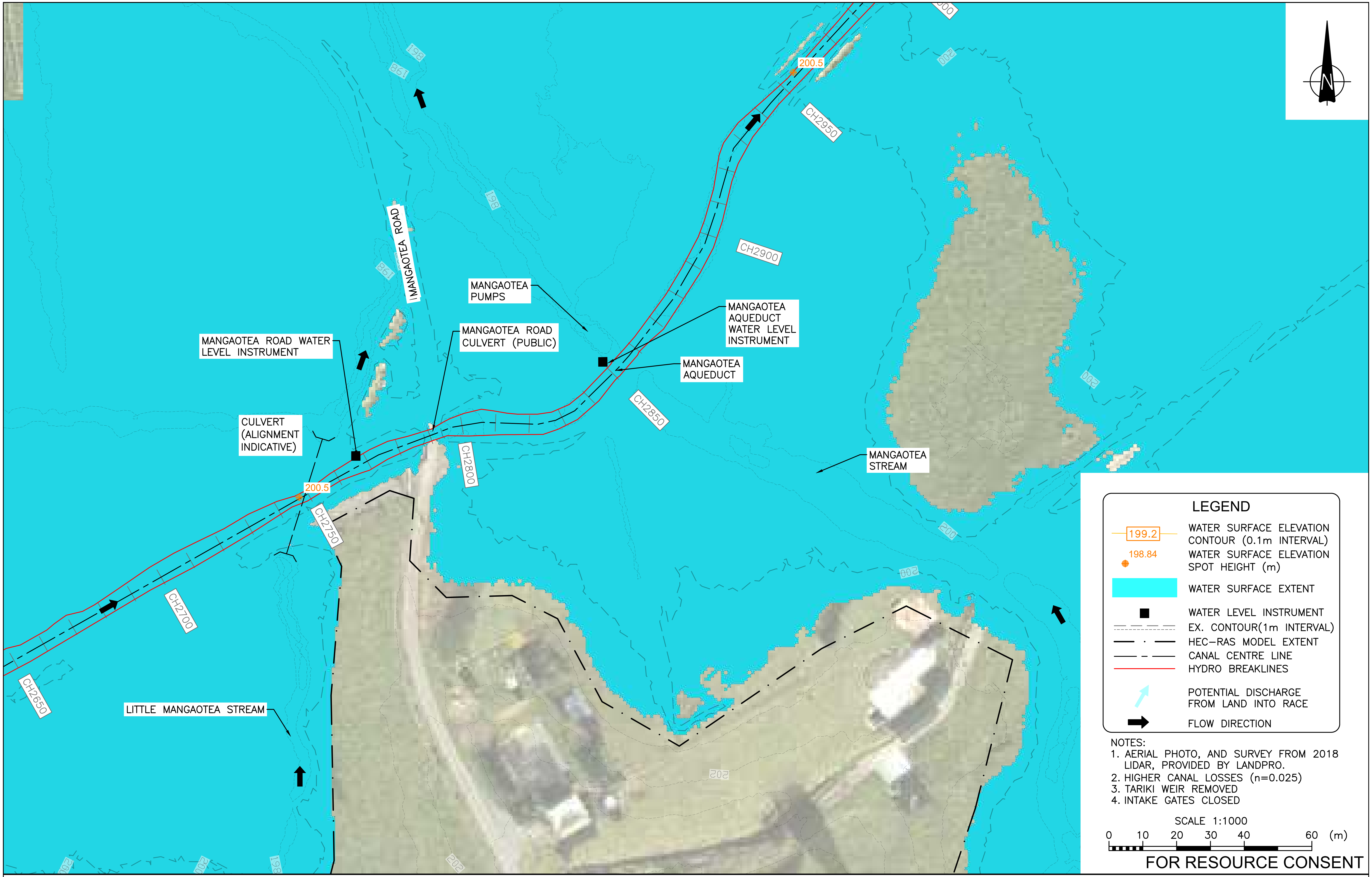
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 8

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-277
REV.	1



NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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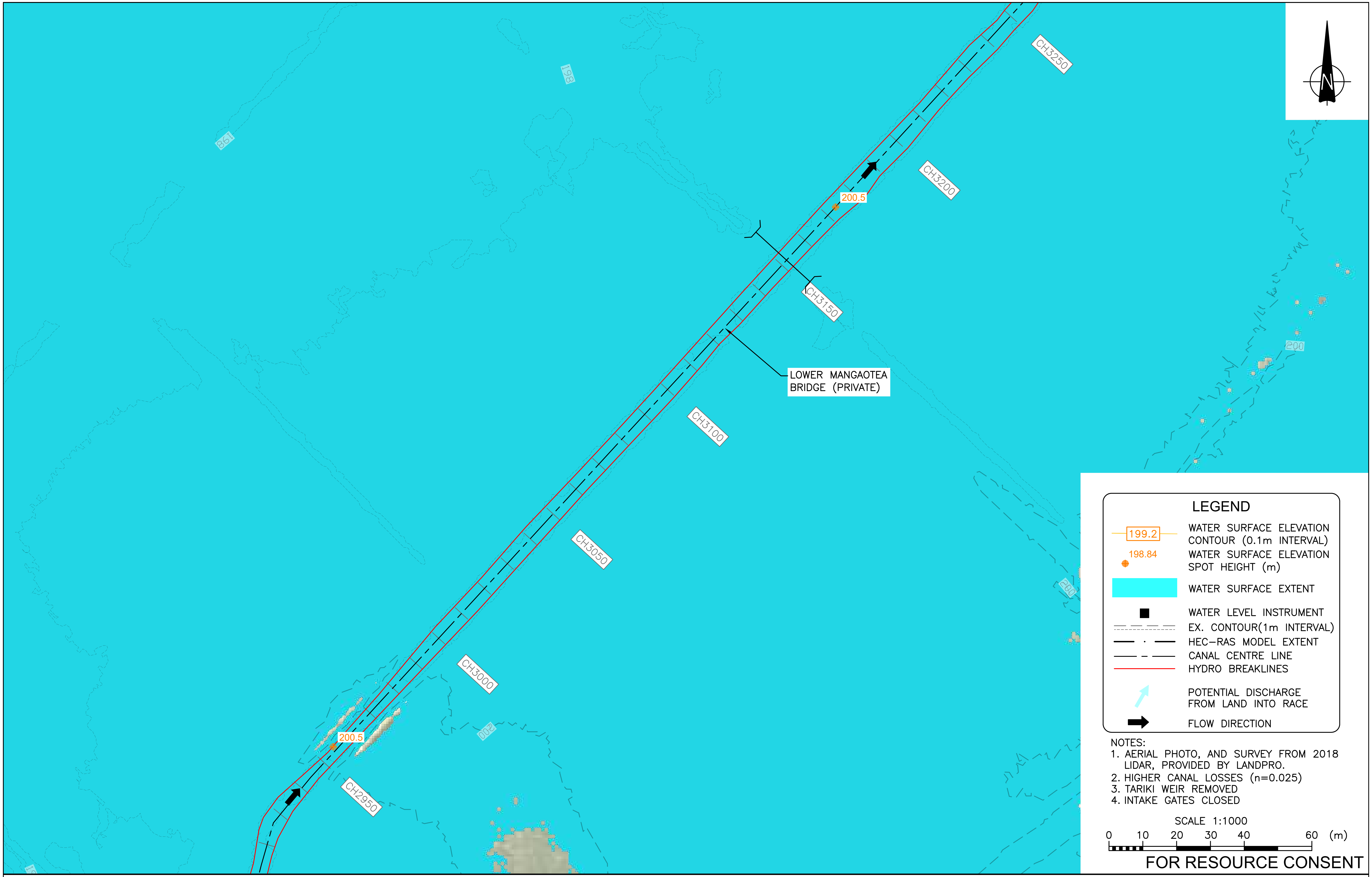
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 9

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-278
REV.	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
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DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 10

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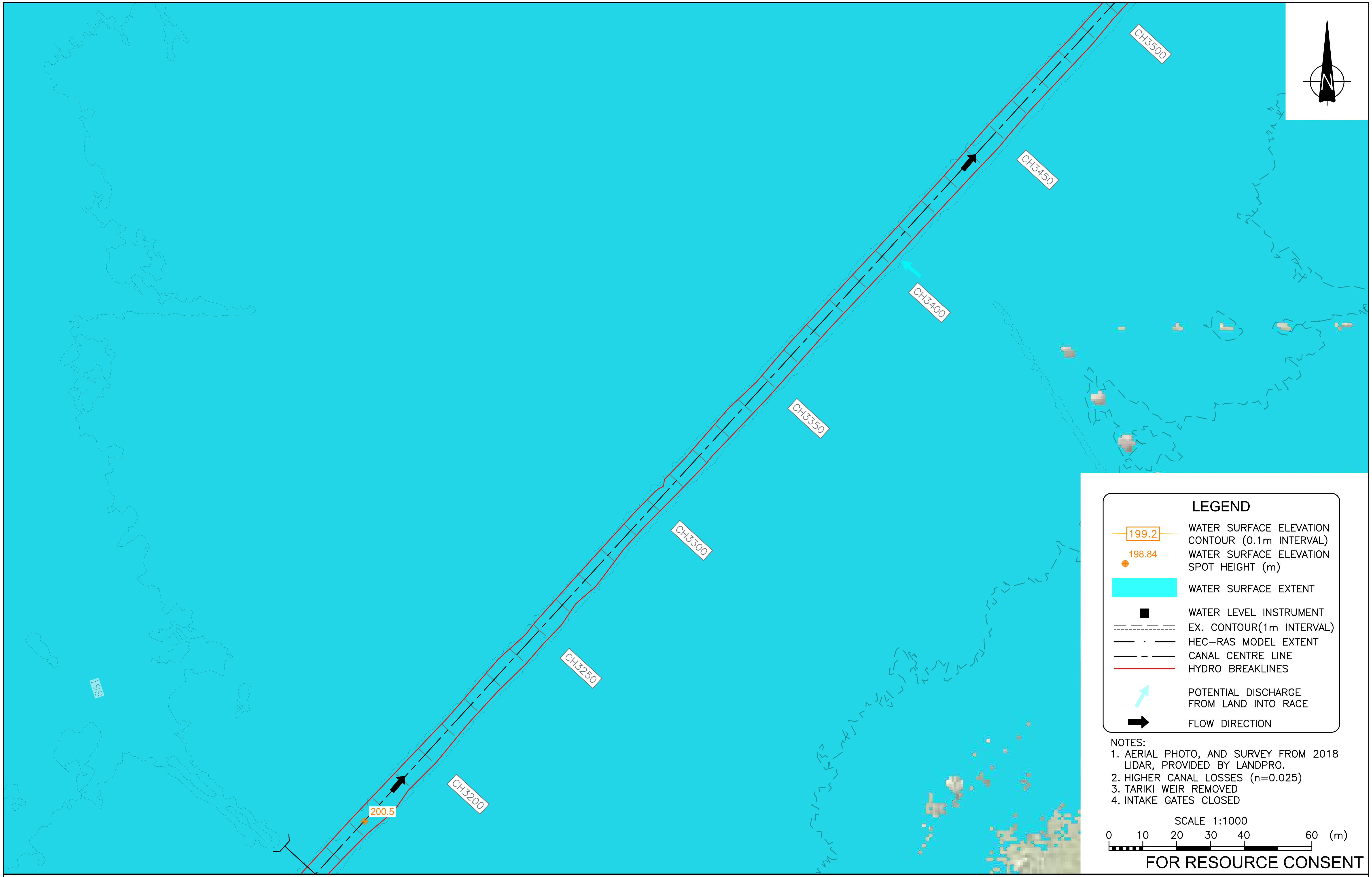
CADFILE	
18MTK_ENH–270to287.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-279	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	
DRAWN	CAD CHECK	P LILLEY
OOBG	FY	
DATE DRAWN		ISSUE DATE
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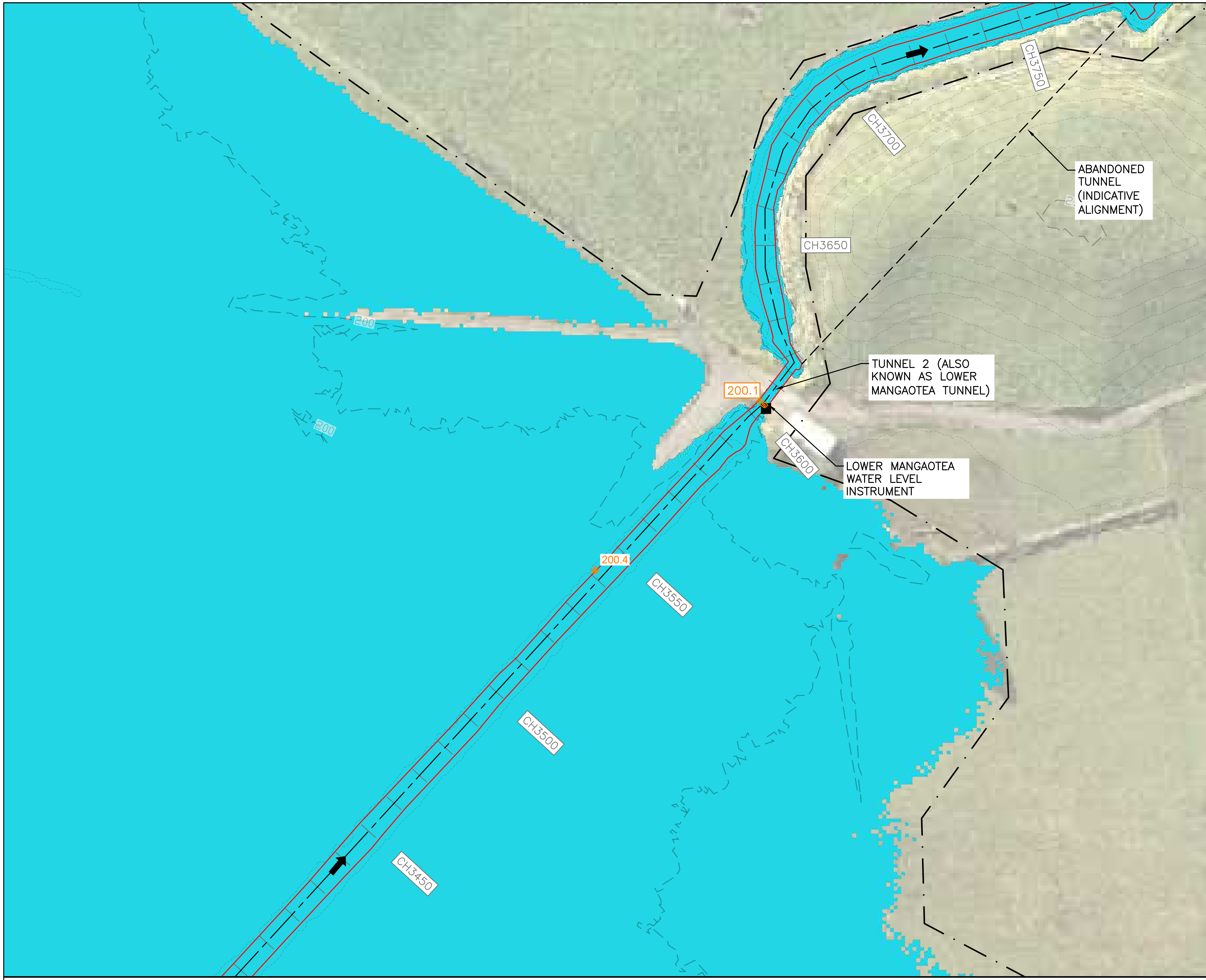
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 11

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	
18MTK_ENH–270to287.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-280	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 12

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ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	1:1000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-281
REV.	1

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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Trust power

CLIENT: TRUSTPOWER LTD
ADDRESS: MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT: RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE: 100 YEAR FLOOD EVENT - LEVELS - PLAN 13

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ISO 9001 CERTIFIED GCS

CADFILE: 18MTK_ENH-270to287.dwg
SCALE (A3): 1:1000
ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-282
REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 14

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-283	REV.: 1

LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 15

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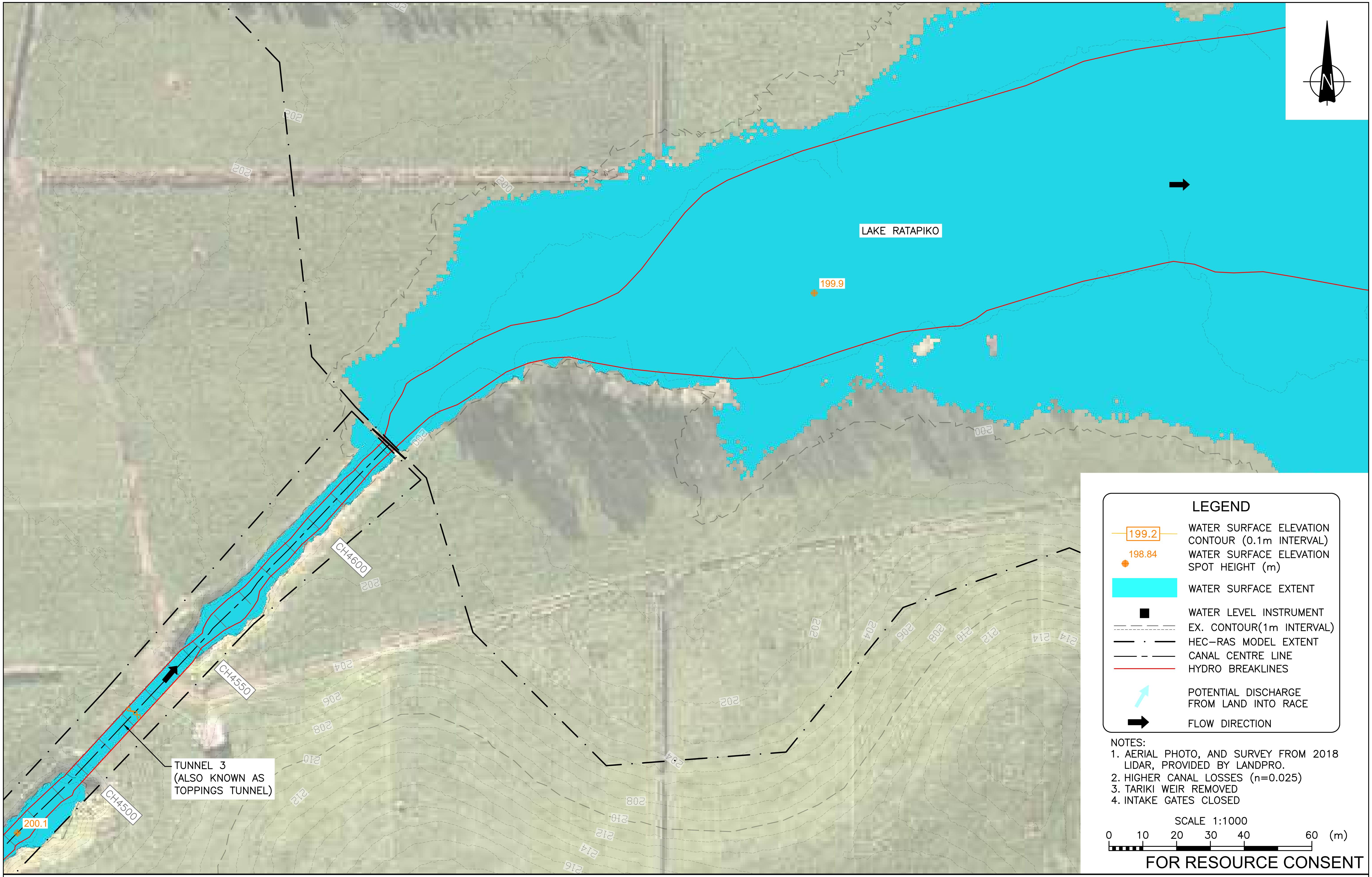
CADFILE	
18MTK_ENH–270to287.dwg	
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-284	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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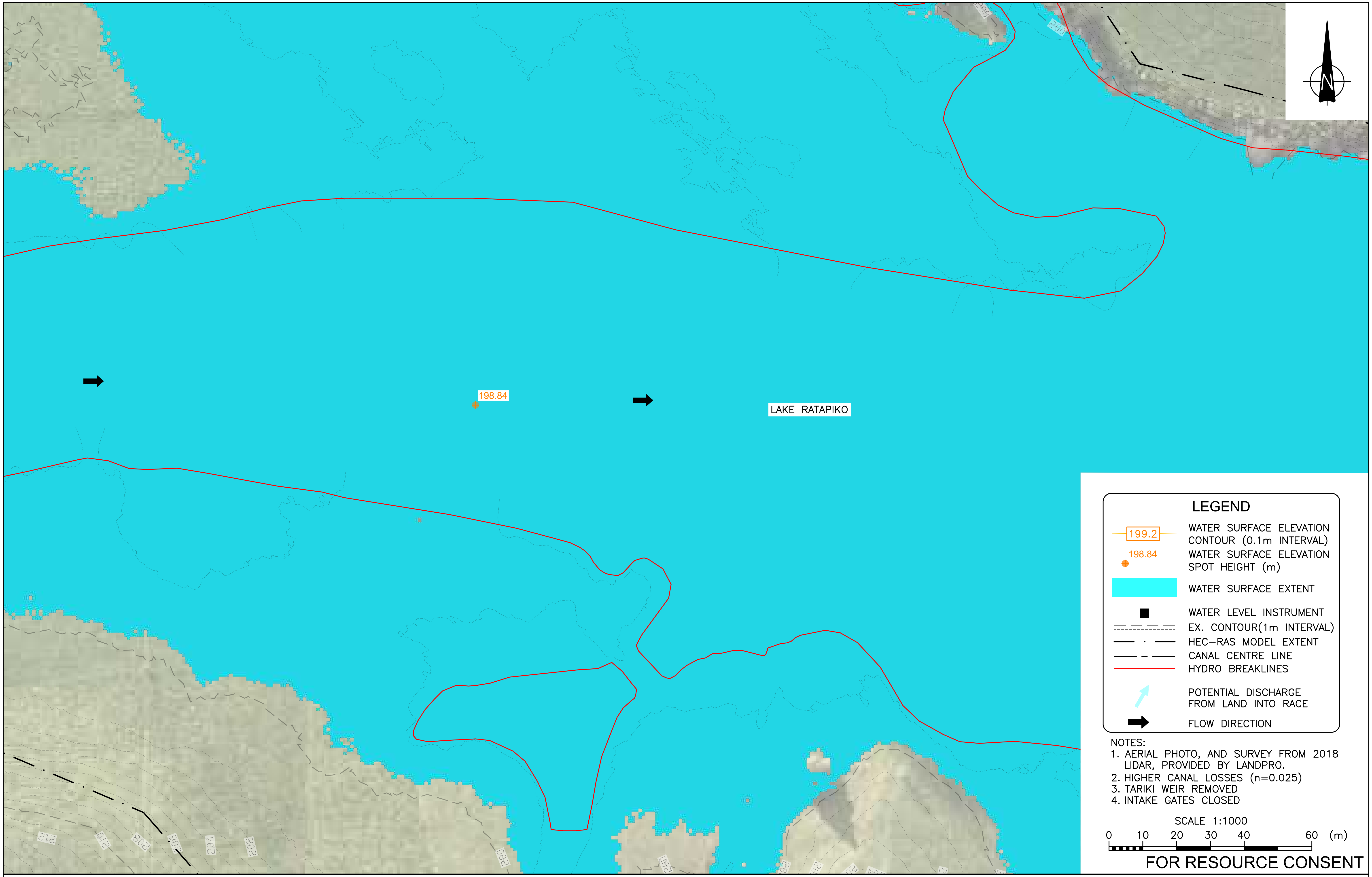
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 16

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-285	1



LEGEND

199.2	WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
198.84	WATER SURFACE ELEVATION SPOT HEIGHT (m)
	WATER SURFACE EXTENT
	WATER LEVEL INSTRUMENT
	EX. CONTOUR(1m INTERVAL)
	HEC–RAS MODEL EXTENT
	CANAL CENTRE LINE
	HYDRO BREAKLINES
	POTENTIAL DISCHARGE FROM LAND INTO RACE
	FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000

0 10 20 30 40 60 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
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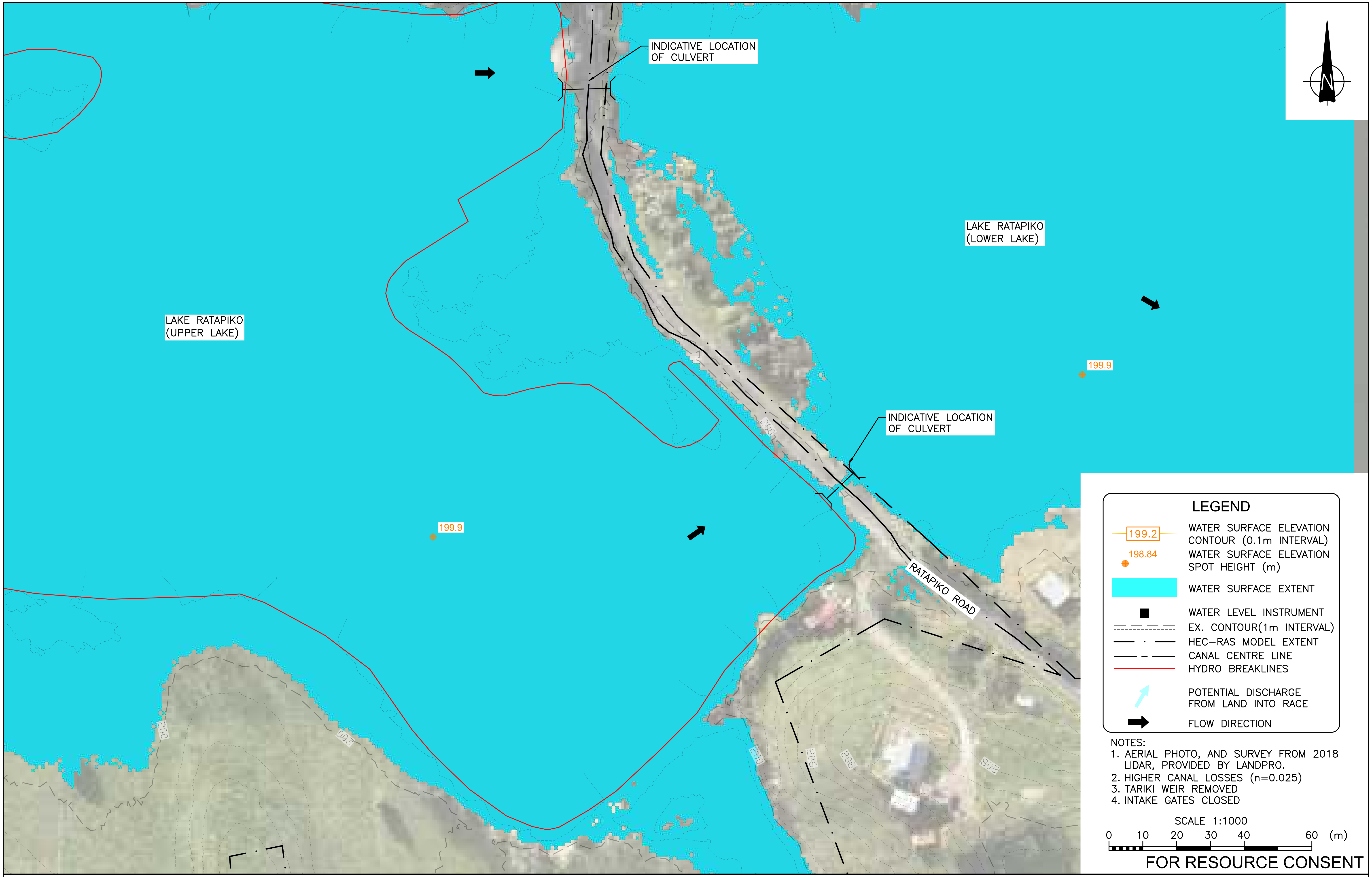
CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 17

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:1000	A3
DRAWING No.	REV.
18MTK_ENH-286	1



LEGEND

- 199.2 — WATER SURFACE ELEVATION CONTOUR (0.1m INTERVAL)
- 198.84 — WATER SURFACE ELEVATION SPOT HEIGHT (m)
- WATER SURFACE EXTENT
- WATER LEVEL INSTRUMENT
- EX. CONTOUR(1m INTERVAL)
- HEC–RAS MODEL EXTENT
- CANAL CENTRE LINE
- HYDRO BREAKLINES
- POTENTIAL DISCHARGE FROM LAND INTO RACE
- FLOW DIRECTION

NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. HIGHER CANAL LOSSES (n=0.025)
3. TARIKI WEIR REMOVED
4. INTAKE GATES CLOSED

SCALE 1:1000
0 10 20 30 40 60 (m)

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REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

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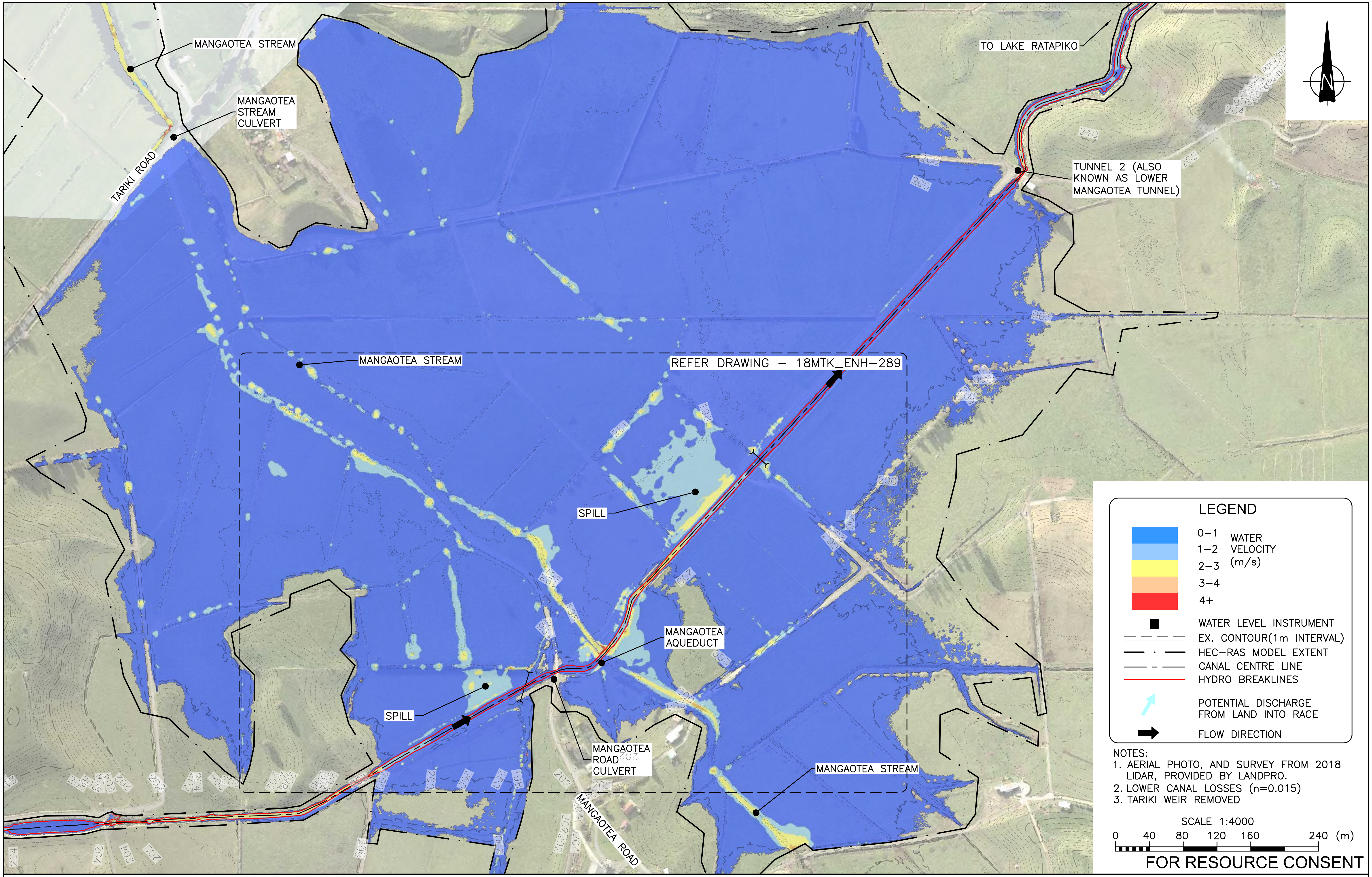
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - LEVELS - PLAN 18

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–270to287.dwg
SCALE (A3): 1:1000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-287	REV.: 1



NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:4000

0 40 80 120 160 240 (m)

FOR RESOURCE CONSENT

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1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

RILEY CONSULTANTS
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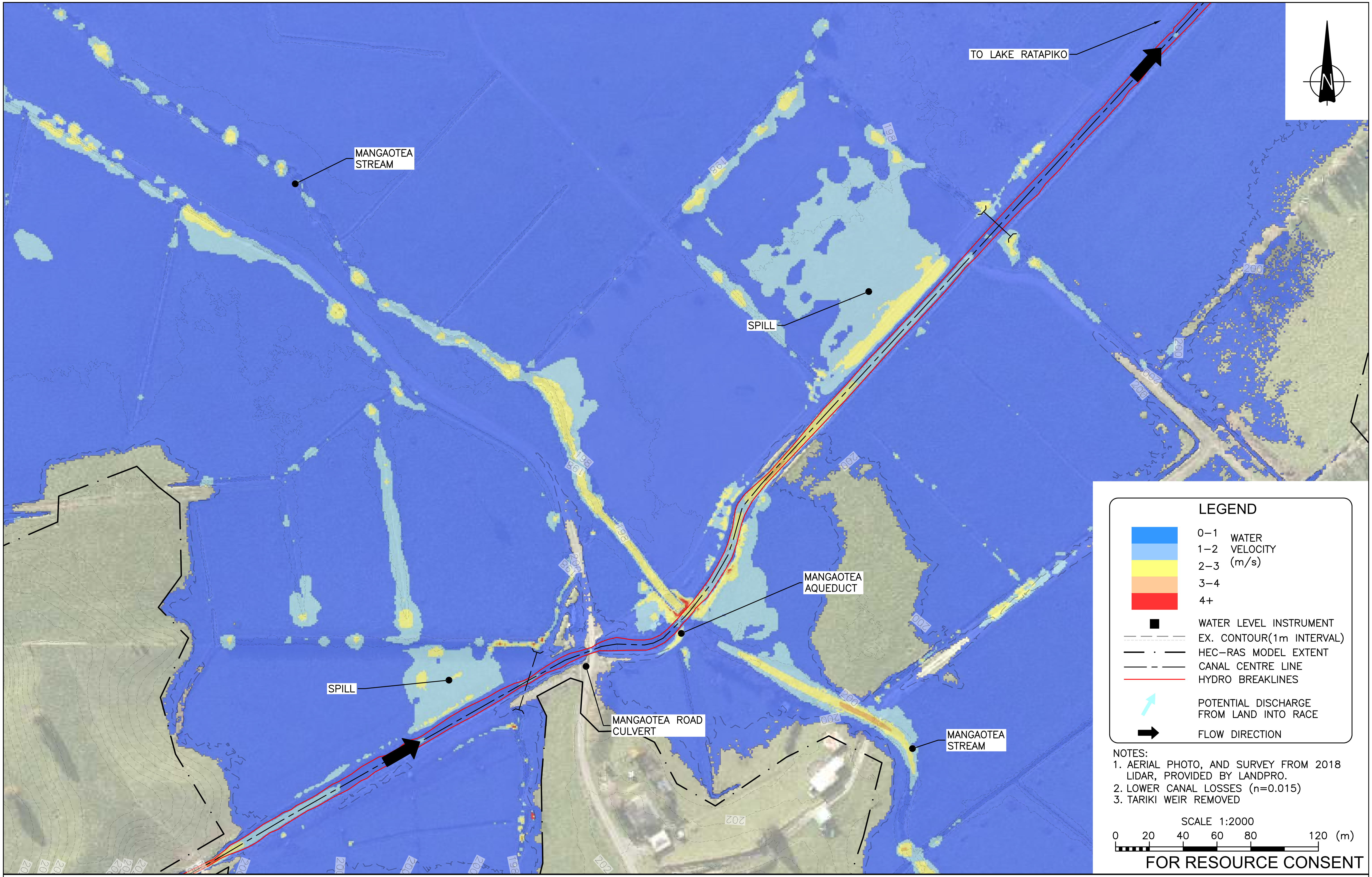
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - VELOCITY - PLAN

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–288to291.dwg
SCALE (A3)	1:4000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-288
REV.	1



NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:2000
0 20 40 60 80 120 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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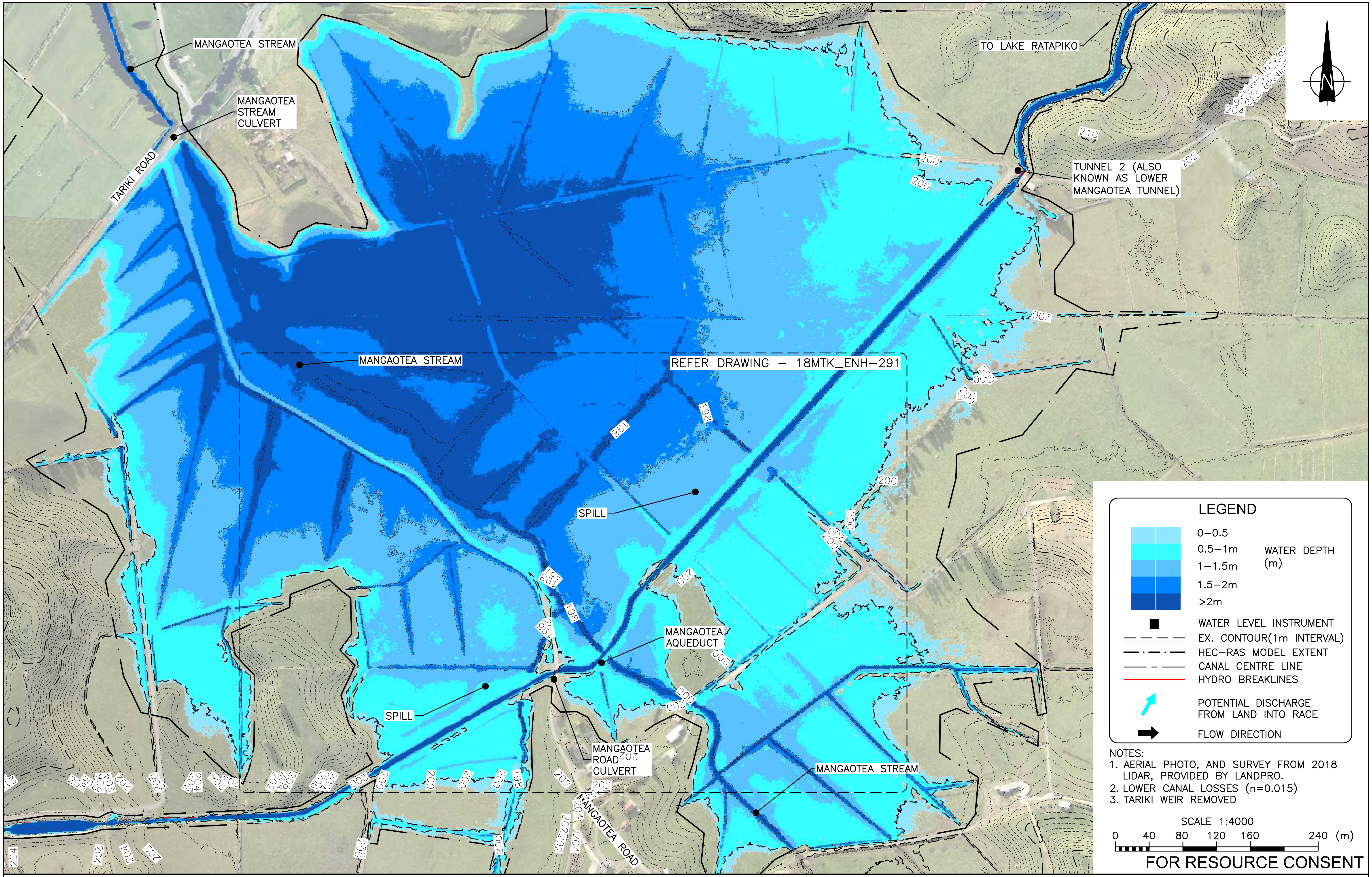
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - VELOCITY - PLAN

ACENZ
ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–288to291.dwg
SCALE (A3)	1:2000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-289
REV.	1



NOTES:
1. AERIAL PHOTO, AND SURVEY FROM 2018 LIDAR, PROVIDED BY LANDPRO.
2. LOWER CANAL LOSSES (n=0.015)
3. TARIKI WEIR REMOVED

SCALE 1:4000

0 40 80 120 160 240 (m)

FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
VM	VM	P LILLEY
DRAWN	CAD CHECK	
OOBG	FY	
DATE DRAWN		ISSUE DATE
AUG 2021		15 / 11 / 21

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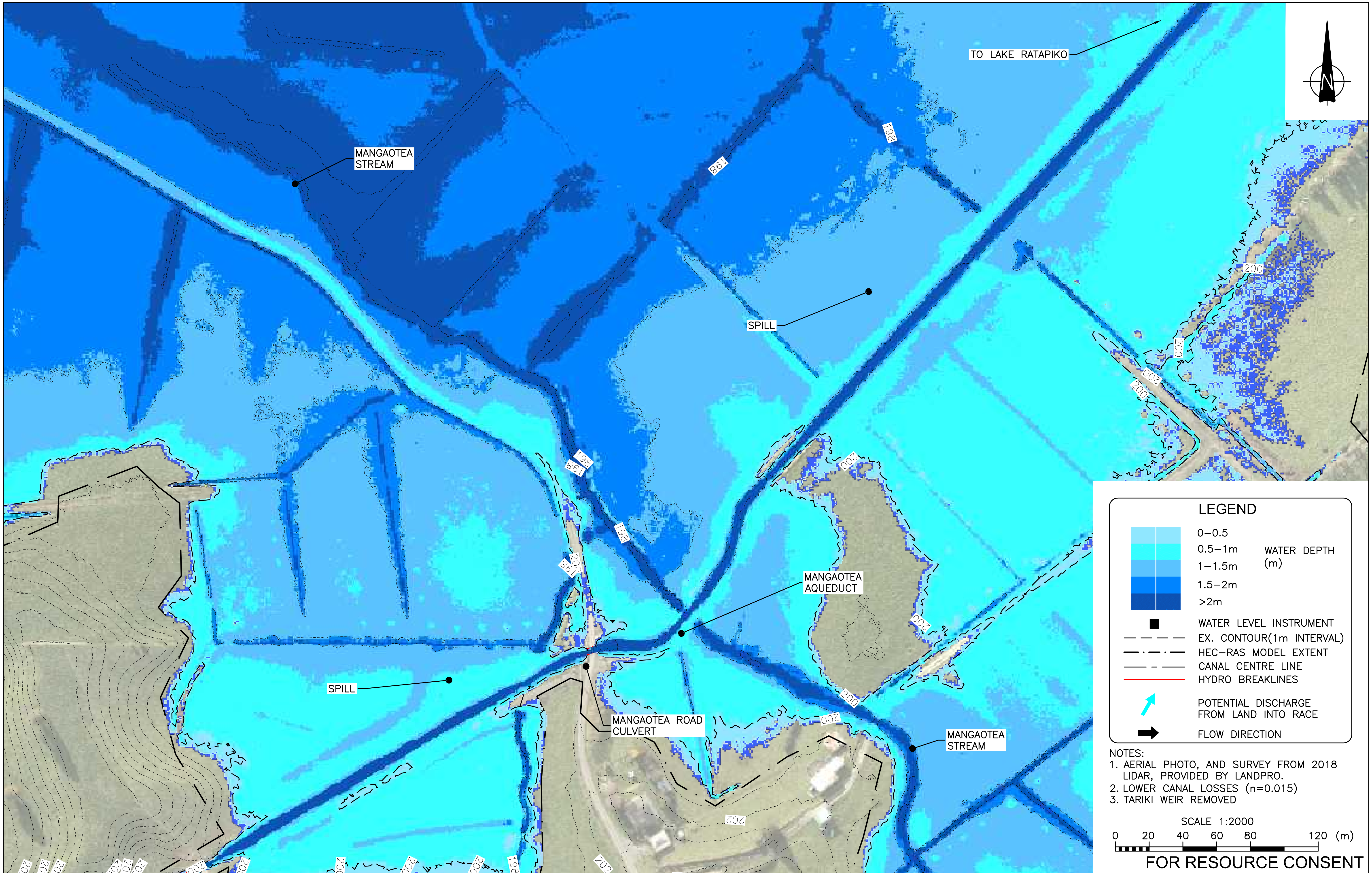
Trust power

CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - DEPTH - PLAN

ACENZ ISO 9001 CERTIFIED GCS

CADFILE	18MTK_ENH–288to291.dwg
SCALE (A3)	1:4000
ORIG. SHEET SIZE	A3
DRAWING No.	18MTK_ENH-290
REV.	1



FOR RESOURCE CONSENT

REV	DATE	ISSUE	BY
1	15.11.21	ISSUED FOR RESOURCE CONSENT	MD

DESIGN: VM	DES CHECK: VM	APPROVED FOR ISSUE: P LILLEY
DRAWN: OOBG	CAD CHECK: FY	
DATE DRAWN: AUG 2021		ISSUE DATE: 15 / 11 / 21

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CLIENT	TRUSTPOWER LTD
ADDRESS	MOTUKAWA HYDRO ELECTRIC POWER SCHEME
PROJECT	RACE HYDRAULIC AND GEOTECHNICAL ASSESSMENT
SHEET TITLE	100 YEAR FLOOD EVENT - DEPTH - PLAN

CADFILE	18MTK_ENH–288to291.dwg
SCALE (A3): 1:2000	ORIG. SHEET SIZE: A3
DRAWING No.: 18MTK_ENH-291	REV.: 1