Lake Pūkaki Reservoir Hydro Storage and Dam Resilience Works

Engineering Structures Assessment

Meridian Energy Limited

29 October 2025

→ **The Power of Commitment**

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1. Introduction

Meridian Energy Limited (Meridian) have engaged GHD Limited (GHD), to assist with obtaining consents to authorise the operation of Lake Pūkaki below the current normal minimum level of 518.0 m above mean sea level (m RL), for a three-year period, and for civil works at Pūkaki Dam to improve the structures resilience to wave action during lower lake operational levels.

1.1 Project Background

1.1.1 Waitaki Power Scheme

The Waitaki Power Scheme (WPS) is a nationally and regionally significant component of New Zealand's electricity supply infrastructure. It is New Zealand's largest and most flexible hydroelectricity power scheme and therefore has a critical role to play in the electricity system and economy. It consists of eight power stations (two owned by Genesis Energy and six owned by Meridian Energy), commissioned between 1935 and 1985, together having an installed capacity of 1,761 MW, being ~32% of New Zealand's installed hydro capacity.

Lake Pūkaki is a modified natural lake and is managed as part of the WPS. It is New Zealand's largest hydro storage lake and provides an average of 1,767 GWh of stored water in normal operating conditions, with an additional 545 GWh available during a national electricity shortage.

Meridian is currently authorised to dam the Pūkaki River to control and operate Lake Pūkaki between the levels of 518.0 m RL (normal consented minimum lake level) and 532.5 m RL (maximum consented storage level).

1.1.2 Previous Plan Changes - Waitaki Catchment Allocation Regional Plan (WAP)

The WAP is a sub-regional plan and provides objectives, policies and rules for the use and development of water resources within the Waitaki Catchment. Prior to 2012, it was a prohibited activity in the WAP for Meridian to draw the lake level below 518.0 m RL.

1.1.2.1 Plan Change 1 (PC1)

In 2012, Meridian initiated Plan Change 1 (PC 1) to the WAP which sought to introduce a new minimum lake level for Lake Pūkaki during circumstances when the System Operator (SO) had commenced an Official Conservation Campaign (OCC) in regard to electricity supply. PC1 allowed additional water from Lake Pūkaki to be used for generating electricity as a permitted activity when an OCC is declared by the SO.

When assessing the potential operation of Lake Pūkaki below 518.0 m for PC1, the duration of an entire event (time below 518.0 m RL) was considered likely to be between 4-7 months (this includes the time spent operating below 518.0 m RL while the OCC was in place, as well as the time required to restore the lake level to above 518.0 m RL once an electricity supply emergency ended). Supporting technical effects assessments were submitted as part of this plan change process. It was ultimately concluded that allowing access for electricity generation purposes to water stored between 513.0 and 518.0 m RL, as a permitted activity once an OCC had been declared, was appropriate and promoted the sustainable management purpose of the RMA. PC1 was adopted by Environment Canterbury on 27 September 2012.

The technical studies completed for this project have relied on the PC1 2012 effects assessments as being appropriate and have focused on both the changes that have occurred in the environment since 2012, and the differences between the activities permitted by PC 1 and the proposed activities. This is the 'Baseline' that is referred to throughout this report.

1.1.2.2 Plan Change 3 (PC3)

PC3 included a new rule regarding the use of Lake Pūkaki between 518.0 m RL and 515.0 m RL. In addition to the PC1 Permitted Activity rule, at times of a Security of Supply Alert (SSA) initiated by the SO, the lake may be operated between the alert minimum control level of 515.0 m RL and 518.0 m RL. The rule is not a permitted activity and to implement this, Meridian applied for and was a granted resource consent in 2018 (CRC185833). This consent expired on 30 April 2025 but has been granted a section 124 continuance while the new replacement consent (CRC240441) is being processed.

1.1.3 Meridan's Application

Meridian is seeking approvals under the Fast Track Approvals Act (FTAA) to enable access to water stored in Lake Pūkaki below 518.0 m RL, without the currently applicable security of supply triggers, thereby enabling the better planning and utilisation of the available stored generating capacity. Further information on the background to the proposal and the benefits of allowing access to additional water is provided in the Substantive Application[1] document that supports the FTAA application.

Meridian is proposing to access the additional storage for a time-bound period of three years, until the end of 2028. For the purpose of this report 'Eased Access', refers to the ability to use water from Lake Pūkaki between 513.0 m RL and 518.0 m RL without a SSA or OCC being initiated by the SO. The ability to access stored water below 518.0 m RL will be incorporated into Meridian's electricity generation models and water stored in Lake Pukaki (both above and below 518.0 m RL) will continue to be managed to supply the market. The three-year period is to allow for additional generation capacity that is currently being built, to come online. For further clarification, the existing lake operation framework and proposed activity is detailed below in Table 1.

Table 1 Proposed Activity – Eased Access

Existing Framework	Proposed Activity
Operation of Lake above 518.0 m RL (CRC905321.7).	Operation of Lake above 518.0 m RL (CRC905321.7). UNCHANGED.
Operation of Lake between 518.0 m RL and 515.0 m RL as a discretionary activity at times of a Security of Supply Alert initiated by the System Operator (CRC185833).	Operation of Lake between 518.0 m RL and 513.0 m RL for a period of 3 years without a Security of Supply Alert or Official Conservation Campaign being initiated by the System Operator.
Operation of Lake between 518.0 m RL and 513.0 m RL as permitted activity during an Official Conservation Campaign initiated by the System Operator (Permitted Activity).	

In addition to the temporary ability to lower the lake level, Meridian seeks consent for the installation of rip-rap on the face of the Pūkaki dam and its left and right abutments to provide protection from wave erosion, when operating the lake below 518.0 m RL. Rip-rap will be placed to a maximum depth of 510.5 m RL, with earthworks/site preparation activities extending to a maximum depth of 509.6 m RL. Rock armouring will take a total of 12-18 weeks to complete but is expected to be done over multiple stages over several years and works may be required to be completed beyond 2028.

Meridian has stockpiled rock for this purpose on its land adjacent to the Pūkaki dam since 2014, but the rock armouring has not been undertaken due to the existing supply triggers never being initiated by the SO, with the result that the lake level has not been low enough over that period to allow the works to be completed

[1] GHD 2026 Fast Track Substantive Application "Lake Pūkaki Reservoir Hydro Storage and Dam Resilience Works"

1.2 Purpose of this report

This report summarises the potential impacts on existing lake edge engineered structures when operating Lake Pūkaki between 518.0 m RL and 513.0 m RL. It provides a description of the impacts, proposed mitigation measures and a high-level risk assessment for the sites assessed.

Additionally, the report also provides a summary of other potentially impacted existing structures and facilities, including:

- The Alps to Ocean Cycle Trail (A2O) and Te Araroa (TA) walking trail that crosses the Lake Pūkaki dam adjacent to State Highway 8 and passes through the area of proposed works associated with rip-rap placement on the upstream face of the dam
- New Zealand Transport Agency (NZTA) requirements for utilisation of State Highway 8 for transport of rip-rap rock from stockpiles to the work sites.

2. Scope and limitations

2.1 Scope of work

An initial assessment of potential existing engineered structures that may be affected by the operation of lake levels below 518.0 m RL was conducted in January 2024, consisting of a site visit and literature search. The initial assessment identified six potential sites that may be impacted, of which two were identified as being likely to be impacted and needing further assessment.

This report summarises the findings of the initial assessment and provides additional information (where available) and reassessment of all six sites to determine and describe the likely impacts on the structures.

In addition, Meridian Energy has engaged with A2O, Te Araroa Trust and NZTA to discuss the proposed approach to mitigation of construction effects.

2.2 Assumptions and Clarifications

No as-built plans/drawings for any of the structures have been sighted by the project team, except for the Catherine Fields Irrigation Intake.

The Pūkaki Dam outlet and spillway structures are noted in this report but excluded from specific assessment as these sites are assessed within the Lake Pūkaki Reservoir Hydro Storage and Dam Resilience Works - Pūkaki Dam Rip-Rap Design and Construction Methodology (GHD, 2025A).

The potential effects of lake lowering on Tekapo B power station are a summary of work completed by Damwatch (Damwatch 2025).

The assessment only considers impacts caused by lowering of the minimum operating level from 518.0 m RL to 513.0 m RL. Any existing impacts at higher lake levels are not assessed.

2.3 Report Author and Contributions

The qualifications and experience of the report authors are set out in Appendix A. The author confirms that they have read the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note (2023) and agree to comply with it. In that regard the lead author confirms that this structures assessment report is written within their expertise, except where stated that the author is relying on the assessment of another person. The author confirms that they have not omitted to consider material facts known to them that might alter or detract from the opinions expressed.

2.4 Limitations

This report has been prepared by GHD Limited on the instructions of Meridian Energy, in accordance with the agreed scope of work. It is intended to support Meridian's application under the Fast-track Approvals Act 2024 and may be relied upon by the Expert Panel and relevant administering agencies for the purposes of assessing the application.

While GHD Limited has exercised due care in preparing this report, it does not accept liability for any use of the report beyond its intended purpose. Where information has been supplied by the Client or obtained from external sources, it has been assumed to be accurate unless otherwise stated.

3. Proposed activities

Meridian is proposing two activities, being:

– Over a three-year period, having the ability to lower the lake levels below 518.0 m RL to a minimum level of 513.0 m RL, so that stored lake water can be used to generate electricity.
– When the lake levels are low, this enables civil works near the Pūkaki Dam, specifically extending rip-rap armouring to reduce the risk of erosion on the dam face and other critical infrastructure.

The focus of this report is both the impact of lowering the lake on existing lake-side structures and effects associated with the rip-rap construction works on the A2O trail, the Te Araroa trail and SH8.

3.1 Short-term change to lake operating level

Meridian undertook modelling to understand potential changes to lake levels from the proposed activity (Meridian, 2025). The modelling draws on 91 years of hydrological and meteorological data for the lake, and the current understanding of the NZ energy system (supply and demand analysis) resulting in forecasts of stored water (energy), which can be used to understand potential changes to lake levels (Figure 1). The Meridian modelling indicates the following:

Modelled First Year of Eased Operation (2026)

– Under eased conditions of operation, typically lake levels are held lower, but still within the normal operating range above 518.0 m RL most of the time, only falling below 518.0 m RL on occasion.
– There is approximately a 3% probability that lake levels in any given week will be below 518.0 m RL. Therefore, on average the lake level will be below 518.0 m RL for approximately 1.5 weeks in the first year of operation.
– 23% of the modelled hydrological sequences dip below 518.0 m RL in the first year. However, most instances are short duration and not deep. Of the 91 hydrological sequences modelled, 21 sequences fall below 518.0m and of these 21 sequences:
  - 9 fall between 518.0 – 517.0 m
  - 6 fall between 517.0 m – 516.5 m
  - 3 fall between 516.5 m – 516.0 m
  - 2 fall between 516.0 m – 515.0 m
  - 1 falls below 515.0 m
– In terms of duration, in the worst-case scenario, the lake level falls below 518.0 m RL in early September and does not return above 518.0 m RL until December (a duration of approximately 4 months). However, the likelihood of this scenario is extremely low – approximately 1% (1 of the 91 hydrological sequences modelled).

Modelled Subsequent Years of Eased Operation (2027 and 2028)

– The pattern is broadly the same in subsequent years although the probability of falling below 518 m RL in any given week increases very slightly to 3.5% in 2027 and 4% in 2028.

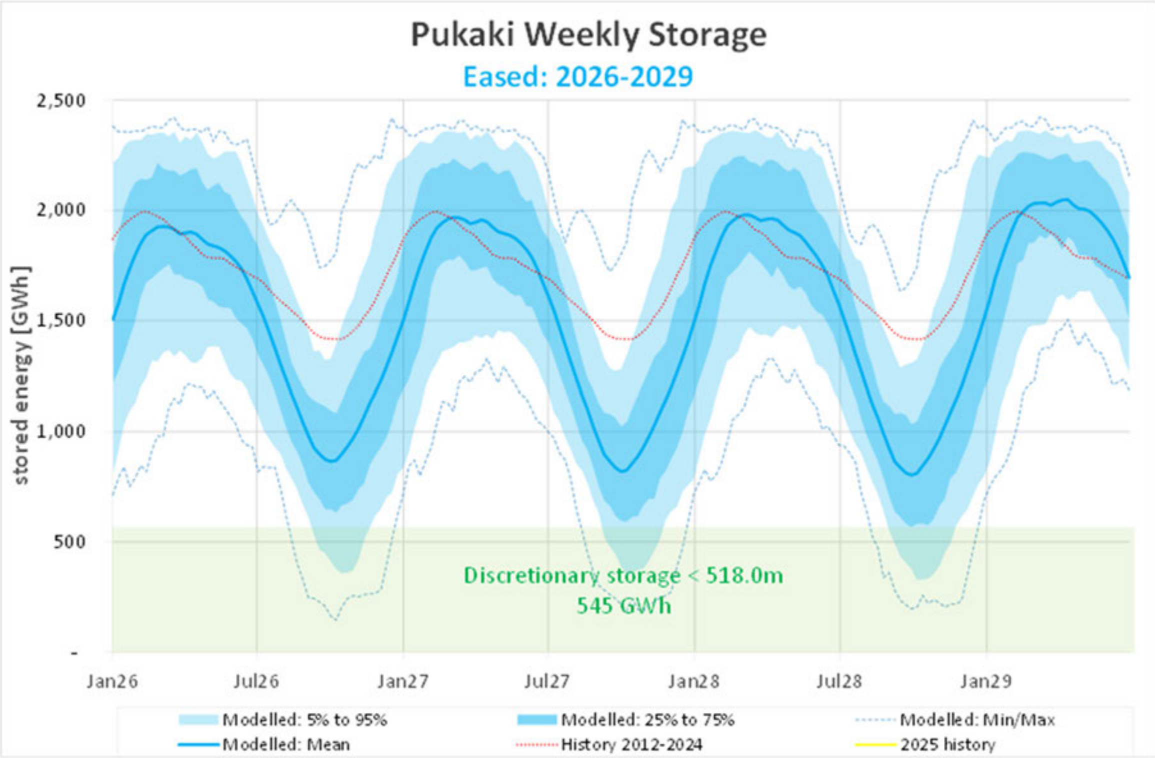

Figure 1 ***Meridian (2025) modelling results for stored lake water***

3.2 Plan Change 1

The PC1 application stated:

– The duration of an event (time below 518.0 m) is likely to be between 4-7 months, with 7 months being an extreme scenario. Refilling of the lake to return above 518.0 m was by late December and sometimes into early January.

– The rate of drawdown of lake levels was estimated to be 1.5 m to 3 m per month in low flow conditions.

– The PC1 application did not consider the frequency of lake levels going below 518.0 m RL

4. Literature review

GHD reviewed the following documentation relevant to this assessment:

– Plan Change request (2012) *Emergency electricity supply: managing the minimum level of Lake Pukaki*

– Aerial imagery & historic photographs (*Pukaki Contingent Storage* presentation, Nov 2019)

– Lake Pūkaki shoreline survey profiles

– Damwatch *Tekapo B Power Station (Hydraulic Review of Weir and Chute, September 2025)*

– Temporary Catherine Fields Irrigation Intake Drawings

– *Glentanner Agreement* (signed 10 August 2012)

5. Site visit

A site visit to Lake Pūkaki was completed on the 23rd and 24th of January 2024. The lake level on these dates was approximately 531 m RL as shown in Figure 2.

Figure 2 Lake level at the time of site visit (Source: Meridian website).

Six structures situated on the lake shoreline with the potential to be affected by the lowering of the lake level, were visually assessed and photographed. Five of the structures were constructed prior to the implementation of PC1 in 2012. The sixth structure, the Catherine Fields irrigation intake, was constructed after 2017.

Figure 3 shows the structures of interest and their location along the Lake Pūkaki shoreline.

Two of the six structures identified are the Lake Pūkaki Outlet and Spillway, situated on the southern end of the lake. The proposed dam protection works will provide erosion protection, and no further assessment has been completed regarding these structures or the surrounding shoreline. Refer to GHD, 2025A for further details in this regard.

The 2012 Private Plan Change request document lists three water take locations within the dam's impoundment area (all unconsented at the time – refer to Appendix 6 of the document). The report goes on to confirm that these takes will not be impacted by the lowering of the lake operational levels to 513.0 m RL, as the intakes mentioned would need to be designed to abstract water down to 513.0 m RL. No structures associated with these three intakes were observed during the site visit.

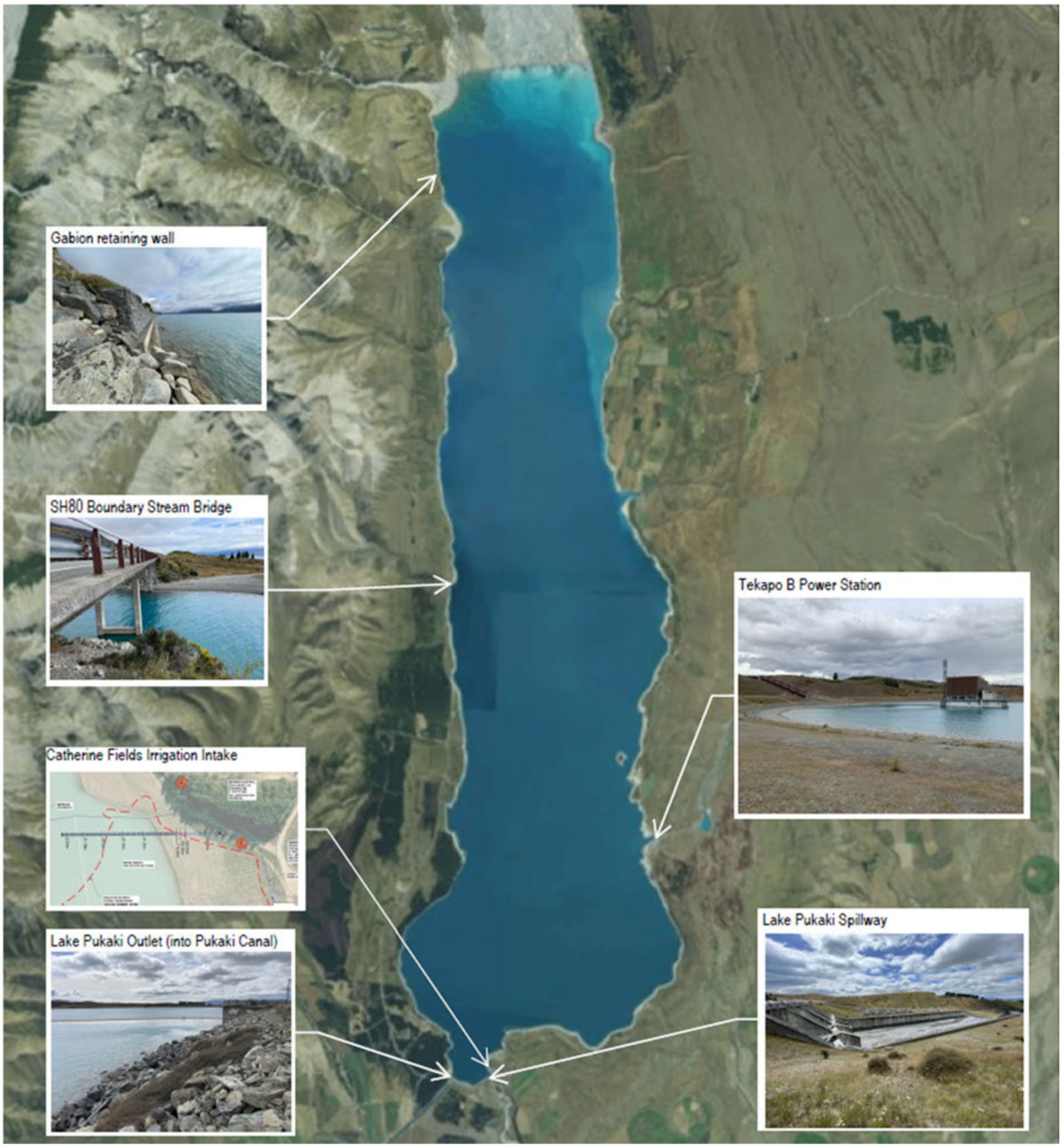

Figure 3 ***Map of Lake Pūkaki, showing the structures assessed on the site visit***

The following sections outline the potential impacts that a lower lake level may have on the four sites assessed.

5.1 Tekapo B Power Station

5.1.1 Overview

The Tekapo B Power Station is a 46 m-high structure situated within a tailrace pond and within the lake's current operating water levels. Commissioned in August 1977, it houses two 80 MW generators which are supplied with water via two large penstocks which are fed from Lake Tekapo via the Tekapo Canal. The peak discharge flowrate through the power station is limited to 130 m^3/s (typically occurring at high tailwater conditions). Tekapo B Power Station and the associated infrastructure is owned by Genesis Energy Limited.

The station is surrounded by water when the discharge is operational (refer to Figure 44) and has been designed for a Lake Pūkaki operational water level range of 14.3 m, with the lowest operational tailwater level at around 518.0 m RL.

Figure 4 Site visit photo of Tekapo B Power Station showing the two penstocks.

The usually submerged portion of the station structure is shown in a 1977 photo taken during construction (Figure 55).

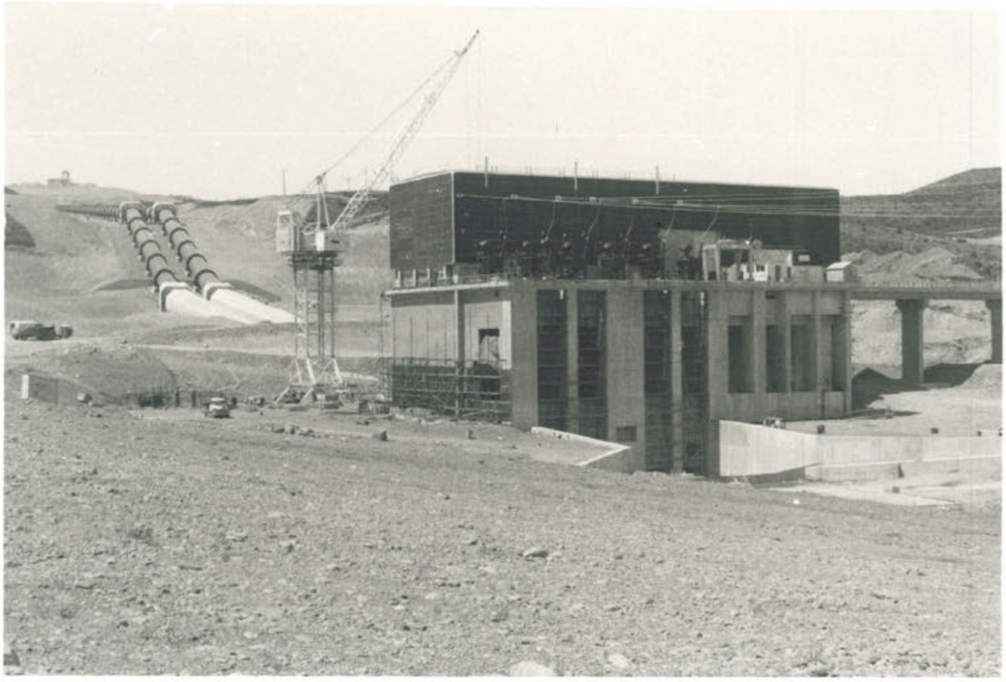

Figure 5 1977 image of Tekapo B Power Station under construction (Source: information board near the power station).

5.1.2 Tailrace Discharge Channel and Weirs

Tekapo B Power Station was constructed prior to raising of Lake Pūkaki. To allow the station to generate power before the lake was raised a tailrace weir with a crest of 517.85 m RL and rock chute were constructed to maintain the tailrace pond at around 518.0 m RL (see Figure 6)

The weir and chute is comprised of three linked components:

- A trapezoidal shaped earth lined channel from Tekapo B outlet to the weir;
- The weir at the head of the chute is formed by a concrete sill approximately 45.7m in length;
- The chute downstream of the weir comprises a further ten concrete sills (beams) spaced every 30.5m over the length of the structure. The overall length of the chute is 364m and it has an average slope of 19.3:1 (horizontal to vertical) except over the last 61 m where it increases to 8:1. The level of the last sill at the base of the chute is 502.23 m RL. The spaces between the sills are lined with rock boulders overlying gravel and sand filters.

Figure 6 shows an aerial view of the weir and chute in operation in the 1970's with Lake Pūkaki water level at around 504.3 m RL.

Figure 6 **Lake Pūkaki at 504.3 m RL showing the full extent of the tailrace and weir.**

Figure 7 shows a plan view of the Tekapo B Power Station, tailwater pond and discharge channel with the 11 sills (crest weir and ten sills) and key reduced levels.

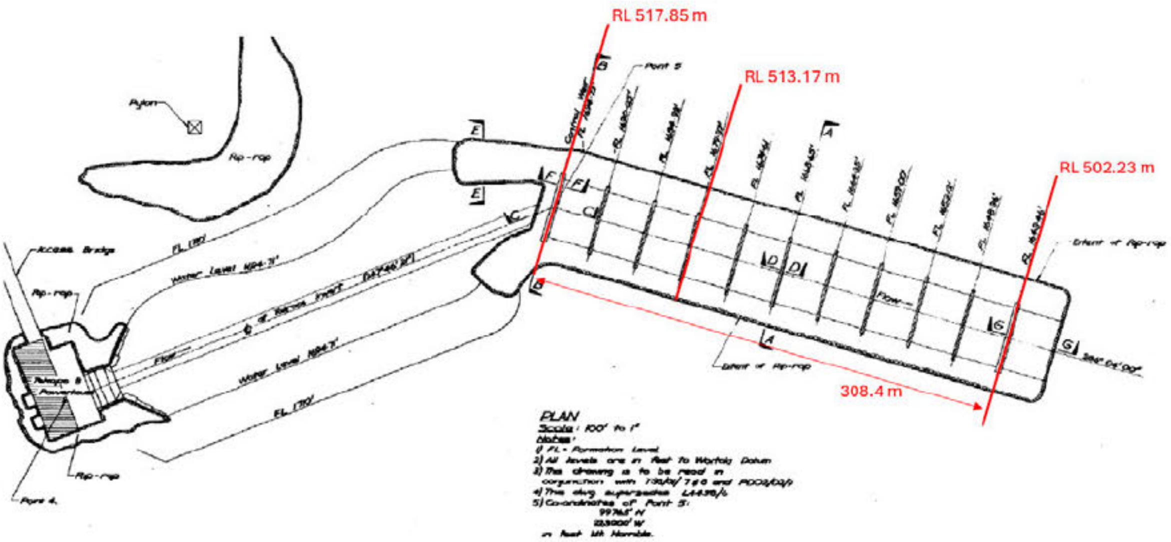

Figure 7 **Plan view showing the Tekapo B Power Station (left) and the tailrace weir (right) with reduced levels.**

Lake level records show the last time the lake level was below 518.0 m RL was immediately following the commissioning of the Tekapo B Power Station in August 1977. Figure 8 indicates that the power station operated below 518.0 m RL for approximately 609 days (or 20 months). Since that time the weir and chute have been submerged, although at low lake levels the structure remains visible below the surface (See Appendix B)

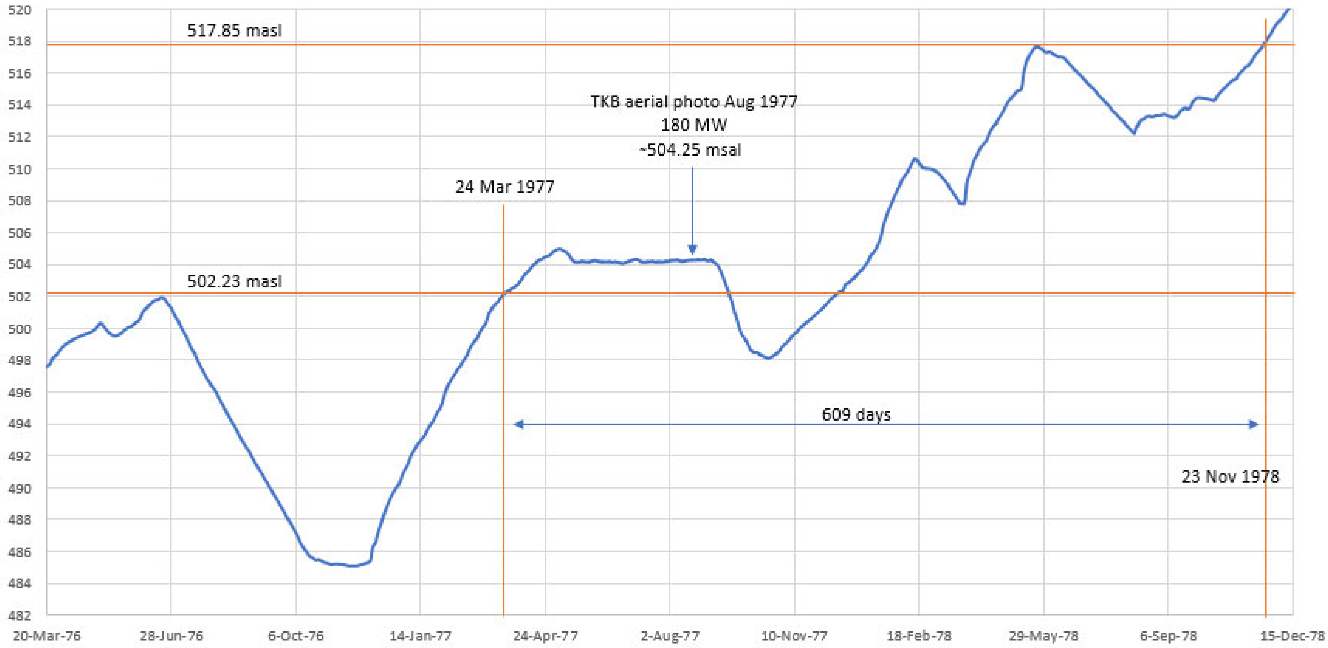

Figure 8 ***Lake level records from 1976 – 1978 showing 15 months following the August 1977 commissioning of Tekapo B power station where the lake was operated below 518.0 m RL, i.e. up until 23 Nov 1978.***

5.1.3 Damwatch Study and Summary of Findings

Drawing the lake level below 518.0 m RL and down to 513.0 m RL will expose the weir and the upper sills with discharge water from Tekapo B power station flowing over the weir and down the chute. Damwatch were commissioned by Meridian in August 2025 to complete a study of the weir and chute (Damwatch 2025 – see Appendix B): The study provides:

– A review of historical documentation for the design and construction of Tekapo B Power Station and other associated studies
– A hydraulic assessment of the weir and chute structure under the proposed operational conditions for accessing Lake Pūkaki below 518.0 m RL. This included:
  - Use of a1D HEC-RAS hydraulic model to assist in the interpretation of hydraulic behaviour
  - An assessment of the likely performance and structural stability of the weir and chute structures based on historical performance
  - An assessment of the structural stability of the weir and chute structures from exposure to wind-generated waves during lowered lake levels.

The full report is included in Appendix B. In summary, the key findings are:

– The chute structure comprises 11 concrete sills with sill 1 being the crest weir. Drawing the lake down to 513.0 m RL will expose the first four of the concrete sills in still-water conditions, with sill 4 located at approximately 513.17 m RL (i.e. no flow from Tekapo B).
– The first two bays (sill 1-2 and 2-3) have large rock material with a medium diameter d_{50} of 0.6 to 0.85 m. From sill 3 to 5 the rock material has a d_{50} of 0.3 to 0.45 m. Below sill 5 the median diameter d_{50} = 0.27m.
– Some scouring occurred during operation of the spillway in the 1970s downstream of sill 5. Rock was stockpiled on site at the time and repairs were made during station shutdowns.
– Field measurement of the hydraulic performance of the chute were made in the 1970s. Overall, it was concluded that the control weir and chute *“successfully maintained a well-defined waterway in spite of some movement of rock downstream”* (Malan and Hancock, 1979 – see Appendix B).
– Modelling completed by Damwatch indicates as the lake drops to 514.0 m RL and with Tekapo B operating, “ramp-type” (or chute) flow conditions (turbulent flow (USBR, 2007 – see Appendix B) would occur across sills 1-2, 2-3 and 3-4.

- Irrespective of the lake level, tailwater levels at Tekapo B Power Station will be unchanged due to the control by the crest weir (sill 1) and remain at around 518.0 m RL.
- Damwatch assessed the weir and chute against the following issues regarding the draw down of the lake below 518.0 m RL:
  - the stability of the rock chute under conditions of ramp-type flow
  - the effects of sediment deposition on flow behaviour down the rock chute
  - the effects of wind-generated waves on the lining of the rock chute
- Damwatch concluded that:
  - The rock material in the bays between sills 1 and 4 is very likely to remain stable if the level of Lake Pukaki is drawn down to 513.0 m RL with ramp-type flow down the chute.
  - It is possible that fine sediment may have deposited on the chute since the 1970s. This could change the characteristics of the chute flow. However, the highly turbulent nature of ramp-type flow will rapidly remove any fine sediment.
  - For a mean annual windstorm the significant wave height will be in the order of 1.5 m. Waves of this height would not break before reaching the rock chute and would impact on the structure as plunging type breaking waves. Damwatch concluded that the rock in the bays as low as sills 5-6 is likely to remain stable under the impact of these waves if the lake level is drawn down to 513.0 m RL. They also note there is no evidence of the rock chute being damaged by wind generated waves during operation in the 1970s.

In summary, Damwatch's report has identified and assessed the potential areas of risk associated with operation of the weir and chute at lake levels between 518.0 and 513.0 m RL. The study has not identified any areas of specific concern. It also notes that the crest weir maintains a water level of 518.0 m RL at Tekapo Power Station. Therefore, the power station is not directly impacted by the lower lake levels.

In addition:

- As describe in Section 3.1, the application is limited to a three-year period and modelling completed by Meridian indicates that any draw down events below 518.0 m RL are most likely to be short in duration and shallow (refer to Section 3).
- As part of PC1, the proposed lake level draw down is already permissible with SO approval. There has been no change in structures or conditions at Tekapo B Power Station since PC1 that would materially alter the effects considered for PC1.
- The operation of the weir and chute will be monitored during periods of lake lowering below 518.0 m RL. The weir and chute, along with the lake edges adjacent to the weir's abutments, will be visually monitored by Meridian. The frequency and nature of the monitoring will be agreed with Genesis Energy Limited.

5.2 Gabion retaining wall

A gabion retaining wall approximately 60 m long is situated on the western shoreline of Lake Pūkaki adjacent to SH80 Mount Cook Road (SH 80 RS 17 RP 12.64 – 12.7).

Figure 9 shows an aerial view of the site and a site photo, and the shoreline profile is shown in Figure 10.

Figure 9 Left: Aerial image of the Gabion retaining wall between SH 80 RS 17 RP 12.64 – 12.7 (Source: Google Maps). Right: Photo taken of the gabion retaining wall facing north.

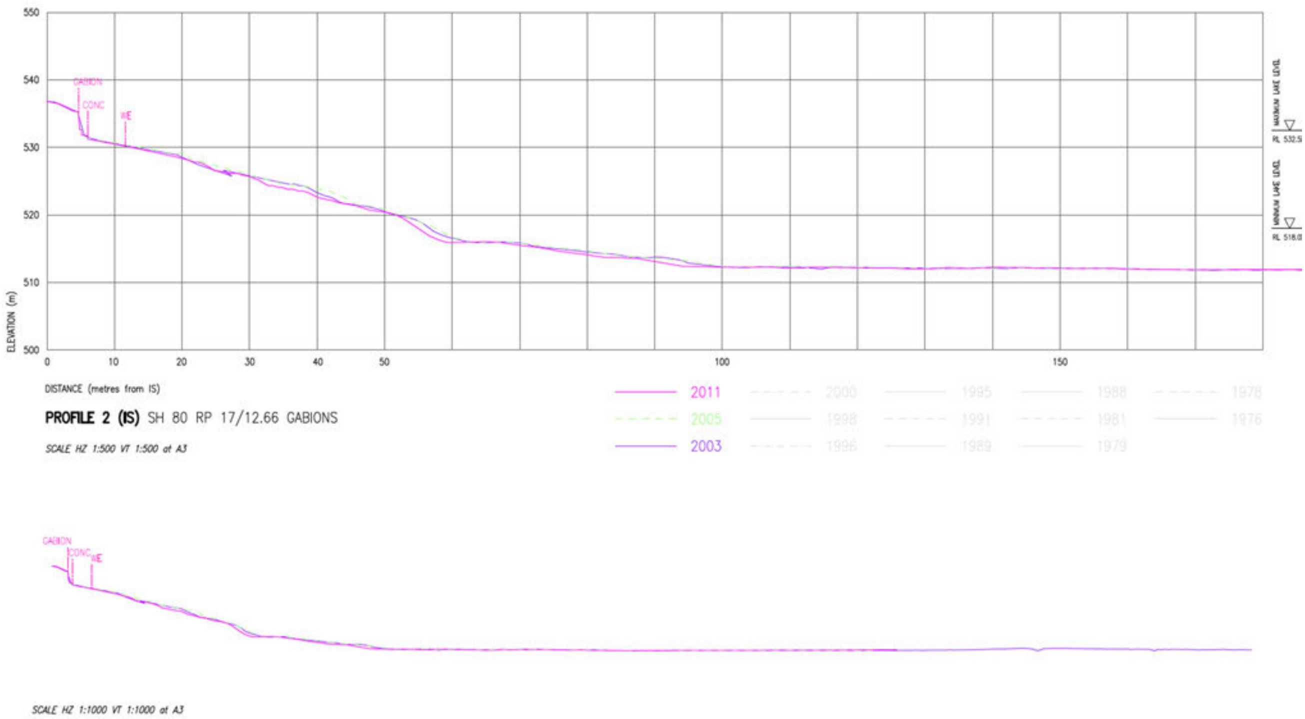

Figure 10 Lake shoreline survey profile surveyed from the gabion retaining wall (Source: Opus report).

As shown in the shoreline survey profile in Figure 10, the concrete base of the gabion retaining wall is situated at approximately 532.0 m RL, and the lakebed slopes down at a gradient of approximately 25%, levelling out at approximately 512.0 m RL. From the photos in

Figure 9, riprap protection is provided for the toe of the wall however the vertical/lateral extent of the protection is unknown.

The gabion wall is unlikely to be directly affected by a reduced lake level as it is located well above the existing operating range. Depending on the duration spent at lake levels below 518.0 m RL and the level reached, there may be some localised bed erosion if the lake level drops below the extent of existing riprap due to wave action. Due to the distance between the gabion retaining wall and the lake shoreline at these lower levels, and the expected short duration at levels below 518.0 m RL (most likely days or weeks with a likely maximum of up to four months), the potential for erosion will be sufficiently far from the wall (80m +) that it will not affect the retaining wall integrity.

Nonetheless, regular inspection of the rip rap erosion protection at the Mount Cook Road gabion wall is recommended during each period of water levels lower than the rip rap extent.

5.3 Boundary Stream Bridge

The Boundary Stream Bridge 170 is situated on the western shoreline of Lake Pūkaki adjacent SH80 Mount Cook Road, where Boundary Stream crosses under the SH80 (Figure 11). The elevation of the base of the bridge piers is within the operating range of Lake Pūkaki, meaning the piers are partially submerged at higher lake levels (Figure 11), and at lower lake levels there is no inundation of the stream channel or bridge piers (Figure 12).

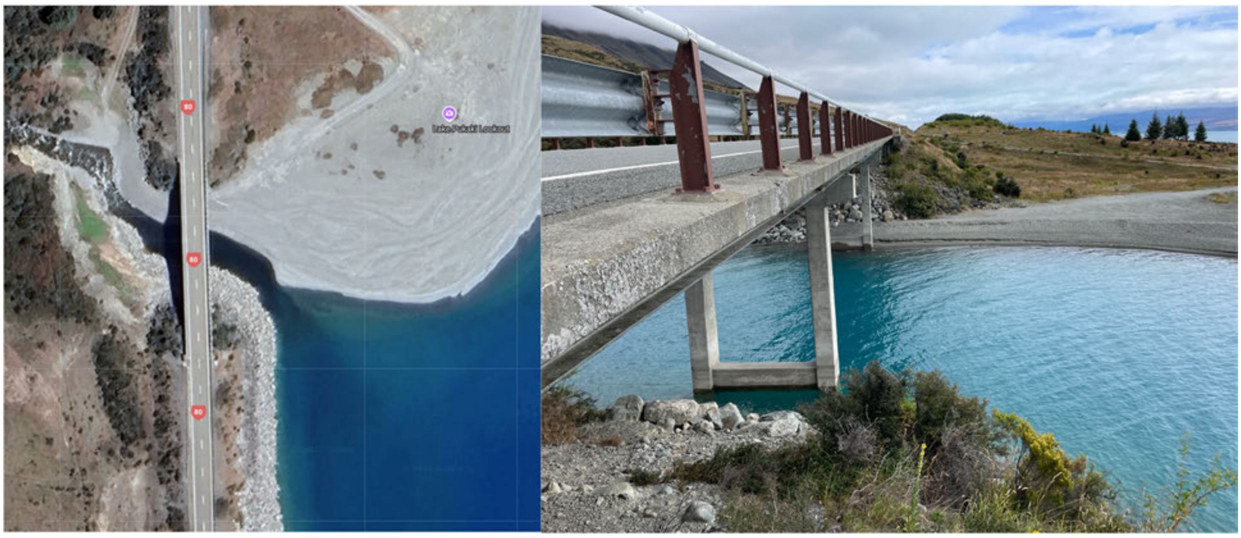

Figure 11 Left: Aerial view of Boundary Stream Bridge (SH 80 RS 17 RP 12.64 – 12.7, Source: Google Maps). Right: Site visit photo showing the partially submerged bridge abutments.

Figure 12 *View from the bridge at a lower lake level in July 2017 (Source: Google Maps).*

The bridge piers are therefore exposed to a range of conditions from typical stream flows and associated potential scour to lake inundation as part of the normal lake operating envelope above 518.0 m RL. Therefore, no additional risk of stream scour is anticipated from a lake level operated below 518.0 m RL. Figure 13 shows the lake shoreline profile in the vicinity of the bridge. An additional risk is a significant step change in lake bed profile between 518.0 m RL and 513.0 m RL which could result in downward and upstream erosion and degradation of the Boundary Stream during low lake level events in the vicinity of the bridge piers. However, the lake bed gradient appears consistent through this area from 522.0 m RL to below 513.0 m RL indicating that additional degradation of the stream bed is unlikely.

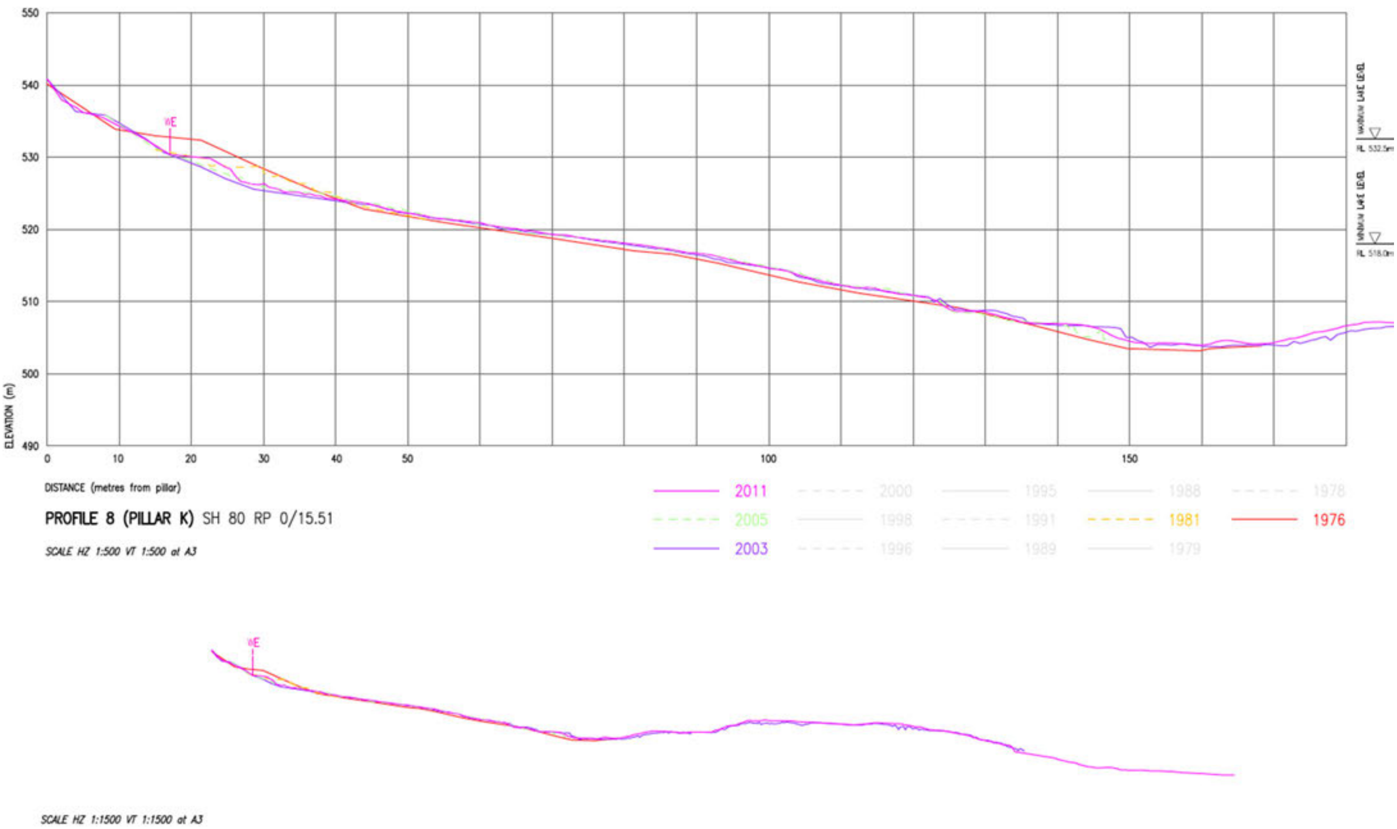

Figure 13 *Lake shoreline survey profile surveyed from the southern abutment of the Boundary Stream Bridge (SH 80 RS 17) (Source: Opus report).*

5.4 Catherine Fields Irrigation Intake

The 150 L/s Catherine Fields Irrigation Intake is located adjacent to SH8, immediately north of the Lake Pūkaki Spillway. This intake structure was constructed circa 2017 and is, therefore, the only structure assessed in this report that was designed and installed after 2012 (i.e. since Plan Change 1).

Figure 14 indicates a plan view of the intake and Figure 15 a long section indicating the anchor blocks and inlet screen within the normal operating range of the intake. Figure 16 provides a close-up view of the intake screen elevation.

A high-level intake options assessment was completed by GHD in September 2024 for this intake, which highlighted that the existing system is inoperable at lake levels below 518.42 m RL. Several temporary design options were developed for consideration, with the aim of addressing the issue of water abstraction at lower lake levels. Meridian has an agreement in place with the owners of the intake structure regarding the supply of water when lake levels fall below 518.0 m RL. It should be noted, however, that the resource consent to take water via this intake has a condition which references the level at which the abstraction must cease - therefore, even if Meridian provides access to water below 518.0 m RL, their resource consent does not enable them to take it.

Based on design drawings of the intake (refer to Figure 16), lowering the lake level below 518.0 m RL will expose the intake screen foundation to wave action, that may not have been anticipated during the original intake design. However, the concrete anchoring system of the screen appears the same as for the pipeline anchor blocks that are currently exposed to wave erosion, so it is unlikely that any more erosion damage will occur than would normally occur on the anchor blocks subjected to wave action above 518.42 m RL.

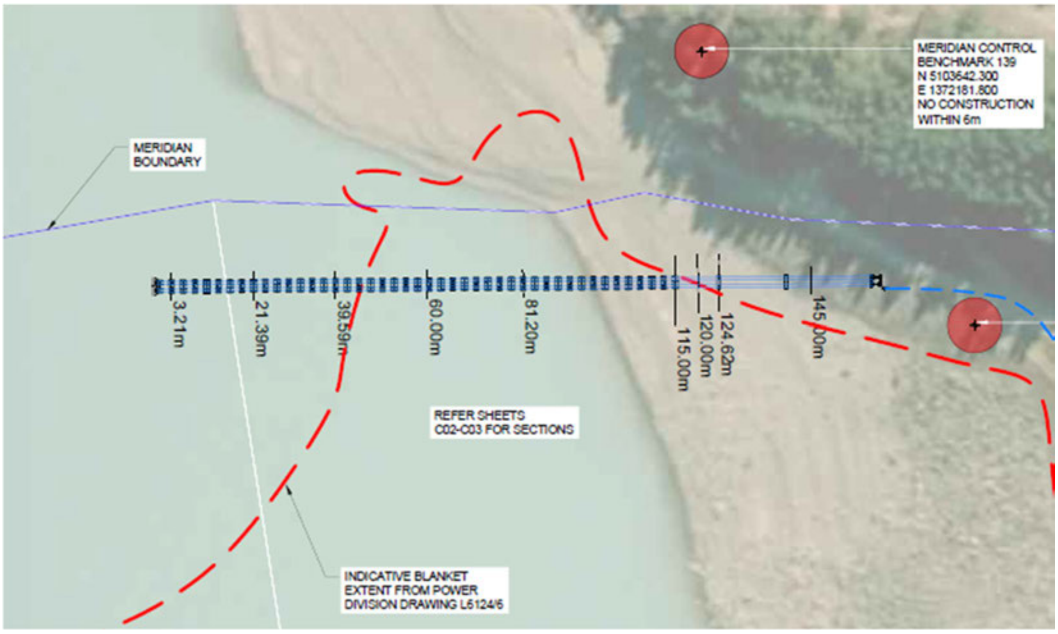

Figure 14 Catherine Fields Intake Plan

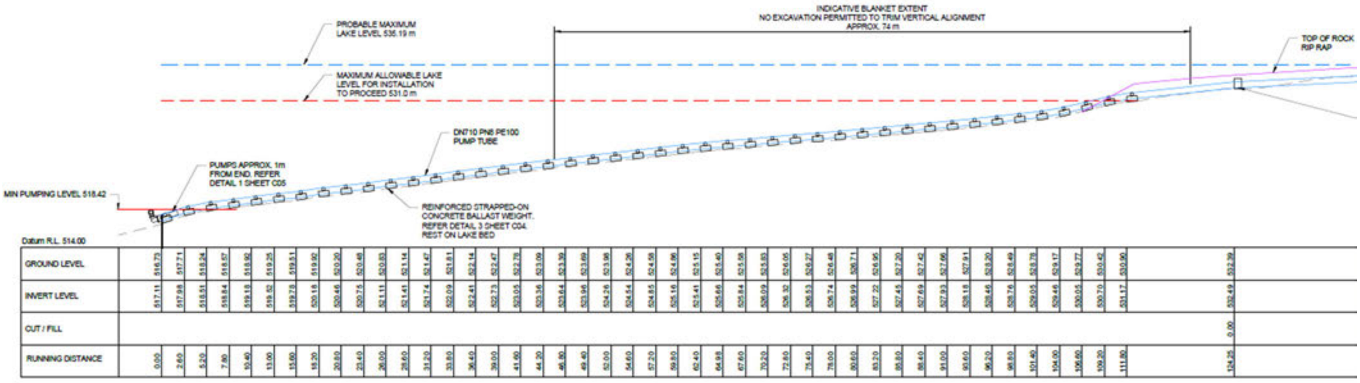

Figure 15 *Catherine Fields Intake Long Section*

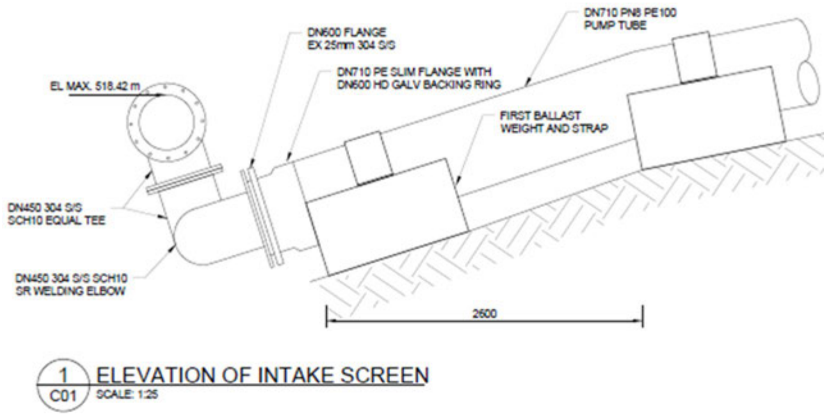

Figure 16 *Catherine Fields Intake Screen*

6. Construction Works Impacts on Existing Facilities

6.1 Alps to Ocean Cycle Trail and Te Araroa Trail

The A2O route is shown on Figure 17 along with the existing rock stockpiles and proposed work zones associated with the Pūkaki dam rip rap placement work. The Te Araroa Trail shares the same route as the A2O in the vicinity of the proposed works. Details of the construction methodology are provided in GHD 2025A. In summary:

- Rock is currently stockpiled at two locations to the south of SH8.
- Rip-rap placement will occur from the two Project Designated Stockpiles, as indicated on Figure 17. Works will occur on the left and right abutments and the upstream face of the main dam.
- Rip-rap rock will be transported via truck from the existing stockpiles to the project designated stockpiles for temporary storage prior to placement. This potentially impacts the A2O and TA route in several locations. In addition, creating a safe construction work zone will require the closure of the existing carpark to the west of the dam and the western most access route to the Pines freedom camping area.–These closures, will also potentially impact the A2O and TA route (see Figure 18 and 19).
- As discussed in Section 1, rock armouring will take a total of 12-18 weeks to complete but is expected to be undertaken over multiple stages over several years and works may be required to be completed beyond 2028.

- Truck movements are likely to be in the order of 20 to 30 per day[2] from each of the northern and southern stockpiles. Light vehicle movements associated with the construction works will also occur. Likely truck routes are shown on Figure 20.

The key areas where the A2O and Te Araroa Trail interact with the proposed works are circled on Figure 20. The proposed approach to mitigation is as follows:

- To the extent possible, the approach is to maintain the existing route with appropriate traffic management controls (as described below). The exception is Area 1 and 2. For Area 1, the A2O/TA route currently traverses through the middle of the western stockpile. In discussion with A2O/TA, the recommended approach is to re-route the trail for the duration of any works, approximately as shown on Figure 20. Two options for re-routing are shown – either can be adopted. For Area 2 the route currently goes through the middle of the eastern work zone and the temporary stockpile area. The recommended approach is to re-route the trail as shown on Figure 20. This re-route will also partly follow the existing access road and will interface with construction traffic and will be subject to the traffic management approach outlined in Section 6.1.1. The second part of the re-route will cross Meridian land on a temporary section of track running parallel to SH8 before re-joining the existing trail immediately before crossing the Pūkaki dam. This route will be closed to all traffic other than A2O, Te Araroa, and construction traffic (see Figure 18).
- For Areas 2 and 3, given the relatively low traffic movements (likely 3 to 4 heavy trucks per hours plus light vehicle movements), the proposed approach is through a Traffic Management Plan (TMP).

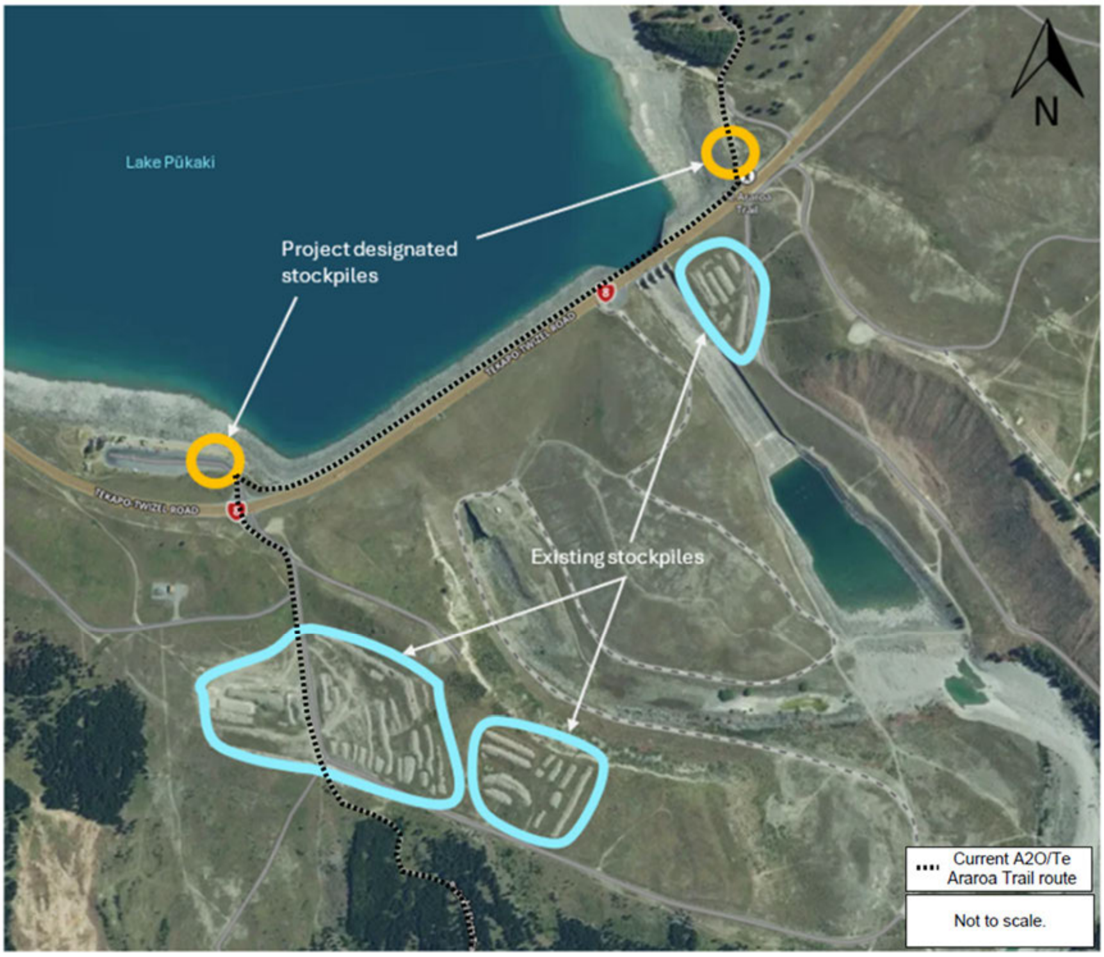

Figure 17 ***Rip-Rap Existing Stockpiles, proposed work areas and A2O Route***

[2] This includes outgoing and returning truck movements for on average 11 deliveries per day of rock to the project stockpiles.

Figure 18 ***Left abutment - security fencing, closed access, access ramp and proposed re-route***

Figure 19 ***Construction Traffic Routes and Proposed Diversions for the A2O Trail***

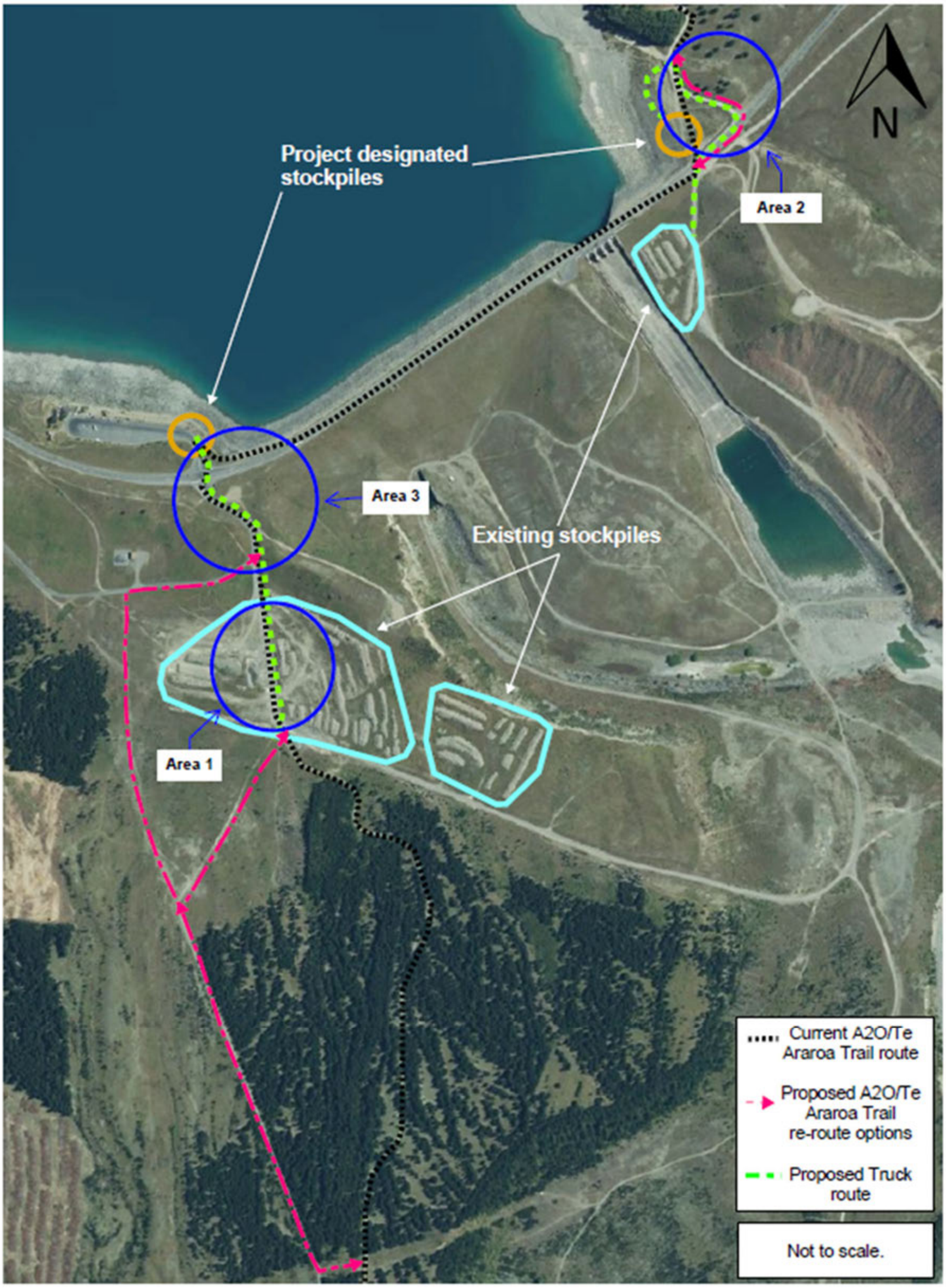

Figure 20 Construction Traffic Routes and Proposed Diversions for the A2O Trail

6.1.1 Traffic Management Plan

- Prior to the commencement of any construction or earthworks activity on the site, Meridian and its contractor will submit a TMP to NZTA (see further below) and A2O/Te Araroa for review to confirm that the TMP contains the information required. The TMP will be prepared by a Suitably Qualified and Experienced Practitioner (SQEP) and will be in accordance with industry best practice for temporary traffic management, and the requirements of the Road Controlling Authority.
- The objective of the TMP is to ensure that construction traffic is managed in a way that maintains the safety and efficiency of the surrounding transport network and minimises disruption to road and track users (including A2O and Te Araroa traffic).
- The TMP will include, but not be limited to, the following:
  a. The location and design of vehicle access points and haul routes.
  b. Anticipated construction traffic volumes and types of vehicles.
  c. Hours of operation for construction traffic.
  d. Measures to avoid, remedy or mitigate adverse effects on the A2O and Te Araroa trail and traffic safety and the efficiency of the road network, including signage, temporary traffic control, and parking restrictions if required.
  e. Provision for safe pedestrian and cyclist access through or past the site.
  f. Measures to prevent dust, debris, and mud being carried onto the public road network[3].
  g. Procedures for ongoing review and amendment of the TMP as necessary.
  h. Contact details for the site manager and the person responsible for traffic management.
  i. Reinstatement of the A2O/Te Ararao route after each period of construction
  j. All construction-related traffic shall be managed in accordance with the TMP for the duration of the works

6.2 SH8 NZTA

As shown on Figure 20, trucks will be crossing SH8 during the period of construction. Initial discussions with NZTA indicate that this will also be managed through the TMP.

NZTA will also require a Corridor Access Request to address remediation of any damage to the sealed highway. This will include highway surface inspections before and after a period of construction to quantify any highway surface damage that will require repair.

7. Conclusions

Based on the assessment described in this report, the conclusions are:

- Three of the four lake side structures assessed as part in this memorandum were constructed prior to the implementation of PC1 in 2012, with only the Catherine Fields Irrigation intake structure constructed later, circa 2017.
- It is highly unlikely that there will be any detrimental impacts on the Mount Cook Road gabion retaining wall and the Boundary Stream bridge structures.
- It is unlikely that that there will be any detrimental impacts of wave erosion on the Catherine Fields intake.
- The existing weir controls water levels at the Tekapo B Power Station at around 518.0 m RL meaning the power station can continue to operate when the lake is drawn down below 518.0 m RL.

[3] See GHD Erosion and Sediment Control Plan (ESCP) for further details on managing tracking of material onto the road network (GHD 2025B)

- Damwatch's report has identified and assessed the potential areas of risk associated with operation of the weir and chute at lake levels between 518.0 and 513.0 m RL. The study has not identified any areas of specific concern.
- During periods of rip-rap construction, effects associated with the A2O/TA trail and SH8 can be managed through:
  - Re-routing short sections of the A2O and Te Araroa trail
  - Implementation of a TPM
  - Implementation of a CAR

8. Recommendations

Based on the afore-mentioned conclusions, we recommend the following:

- That inspection of the rip rap erosion protection at the Mount Cook Road gabion wall is carried out during each period of water levels lower than the rip rap extent.
- Implementation of the TPM and CAR.
- In agreement with Genesis Energy, monitor the performance of the weir and chute during periods when lake levels are below 518.0 m RL

9. References

GHD 2025A Lake Pūkaki Reservoir Hydro Storage and Dam Resilience Works – Pūkaki Dam Rip-Rap Design and Construction Methodology

GHD 2025B Lake Pūkaki Reservoir Hydro Storage and Dam Resilience Works – Draft Erosion and Sediment Control Plan

Damwatch 2025 Hydraulic Review of Weir and Chute

Appendices

Appendix A

CVs

A GHD Principal

André Bresler

Technical Director and Project Director

[redacted]

Location

Christchurch

Experience

30 years

Qualifications/accreditations

- Bachelor of Engineering (Civil), 1992
- Short courses on: Membrane Processes, 2005, Pump & Pipeline Hydraulics, 2007
- National Certificate in Agriculture (Dairy), 2016

Key technical skills

- Project Management
- Engineer to Contract (NZS3910 & 3916)
- Pump and Pipeline Hydraulic Design

Memberships

- Chartered Member of Engineering New Zealand
- Water New Zealand

Relevant experience summary

André's experience encompasses project management and design, review and construction monitoring of bulk water, wastewater, stormwater and irrigation infrastructure projects. He provides design review and mentoring to the Three Waters design team, providing expert evidence, as well as fulfilling project management and client liaison or Client's Engineer roles. André has significant experience in design and design review of water and wastewater infrastructure and a good understanding of implementing large infrastructure projects, including provision of Engineer's Representative services for NZS3910 and 3916 construction contracts. He is also able to quickly assess project requirements from a practical and pragmatic approach.

Experience demonstrating capability in project management, design and design review

Tukituki Water Security Scheme Primary Distribution

Tukituki Water Supply Ltd | Hawkes Bay | 2025 to date
Design Manager

André is the design manager and design reviewer for a new approximately 20,000 ha irrigation and water supply scheme. Stage 1 of the scheme is comprised of developing the Concept Design of the primary canal and pipeline distribution of around 13 m^3/s of water, including river intakes with fish screening and sediment removal. One or more pump stations will also be required.

Part of André's responsibilities involves close collaboration with the client to develop options for the infrastructure alignment based on hydraulic requirements, topography, consented alignment corridors, natural features and landowner constraints and requirements.

Dunedin Resource Recovery Park

Dunedin City Council | Dunedin | 2023 – date
Design Manager

André's services include oversight of the project civils and 3-waters design team and review of project deliverables including the NZS3910 tender documentation.

This project involves a significant redevelopment of the existing 75 ha Green Island solid waste landfill site into a resource recovery park, Dunedin. The site requires the design of civil, roads, water, wastewater and stormwater services to support an Organics Receiving Building, a Materials Recovery Facility, a Bulk Waste Transfer Building, offices and other materials aprons and bunkers. Approximately 5,000 m^2 of roofed buildings is proposed.

Waitaki Street Stop Bank & Stormwater Treatment

Christchurch City Council | Christchurch | 2020 to date
Project Director

André's services included oversight of the project team and review of project deliverables including the NZS3910 tender documentation.

The proposed stopbank is the first section of long-term stopbank that will be constructed to replace the temporary stopbanks that were constructed after the

2010 and 2011 earthquake sequence to protect residents from river flooding, exacerbated by tidal effects. It is therefore a precedent setting design that will likely be used as all the temporary stopbanks are replaced over the coming years.

The proposed stormwater treatment facility is able to treat around 43 ha of residential runoff and consists of incoming stormwater pipelines and structures discharging into an ephemeral wetland system.

Whakatu West Stormwater Pump Station

Hastings District Council | 2023 – 2025

Project Director

Hastings District Council engaged GHD to undertake the detail design of a new 5.5 m^3/s stormwater pump station. The multi-disciplinary project involved the design of a pumped discharge manifold into an existing concrete stormwater pipeline, including non-return flap gate. The system also had to allow discharge of gravity flows when the pump station is not in use.

André was the project director and technical civil design reviewer for this project. He oversaw the multi-disciplinary design project including, civil, mechanical, geotechnical and electrical design aspects.

Ashburton Solar Farm

Confidential Client | Canterbury, NZ | 2022
Project Manager and Design Reviewer

André managed and reviewed this flooding and geotechnical risk assessment for a new solar farm.

The project scope included a desktop assessment of flooding and geotechnical risks to the site, plus river flood modelling to determine potential flood depths and flow velocities over the site. A report was developed to describe the findings of the assessment as part of the client's business decision.

Wakamoekau Community Water Storage Scheme

Wairarapa Water Limited | Masterton | 2020 – 2021
Project Manager & Design Reviewer

André's services included leading and managing the multi-disciplinary design team and acting as the main point of contact with the client. He also provided specific design review for the hydraulic design and river intake aspects of the scheme.

The project involved investigation and design of a new 20 million m^3 storage dam and distribution network, proposed for the Wairarapa (northwest of Masterton). If completed, the scheme will ultimately provide a reliable source of water for industrial and residential users.

As well as around 7,000 ha of irrigable land. An approximately 38 m high embankment dam, plus smaller saddle dam is proposed together with a river intake on the Waingawa River, which is the primary water source for the dam. The Feasibility Design phase of the project was completed, and the project stopped in June 2021.

Kurow Duntroon Irrigation Scheme Upgrade

North Otago | 2016 – 2022
Project Manager & Engineer's Representative

André led GHD's team that provided advice to the Client regarding the various procurement models available, including Traditional, Design Build or Early Contractor Involvement (ECI) type procurement.

Following development of the Specimen Design for a 6,000 ha irrigation scheme, GHD was appointed as Client's Engineer to assist the Client during the procurement process and the subsequent ECI design phase. GHD provided the Engineer to Contract and construction management and supervision support to the Client, with André in the role of Engineer's Representative for the NZS3916 Design Build Contract.

Waikato River Intake Board of Inquiry Evidence

Watercare Services | Auckland | 2020
Expert Witness

André provided specialist evidence at the Board of Inquiry consent application for a proposed river intake for a new 150 MLD water treatment plant adjacent to the Waikato River, near Tuakau. The consent application evidence has been prepared and will be presented later this year.

The river intake consists of multiple fish screens on a submerged intake manifold, anchored to the riverbed. The peak capacity of the new intake is around 3.2 m^3/s. André managed the preliminary intake design and attended the consenting team planning and consultation meetings.

Prebbleton Terminal WW Pump Station

Selwyn District Council | Selwyn District | 2017 – 2020
Project Manager, Design Reviewer & Engineer's Representative

The project was for a new 100 L/s wastewater pump station and involved the design of a wet well and architecturally designed control house with stringent odour and noise control specifications, given its location within a residential area. André's services included oversight of the preparation of tender documentation, and management of the tender evaluation process. André was the Engineer's Representative under a NZS3910 construction contract, with his team providing construction monitoring, surveillance and contract administration services.

Career history

2016 – present	GHD (NZ), Technical Director
2009 – 2014	URS (NZ), Principal Engineer
2000 – 2008	Technical Director (South Africa)
1993 – 1999	Civil Engineer (South Africa)

Appendix B

Damwatch Report

Hydraulic Review of Weir and Chute

28/10/2025

Prepared for:
Meridian Energy

E2567

Issue 3.1

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Project	Tekapo B Power Station Temporary Tailrace Weir and Rock Chute
Document Title	Hydraulic Review of Weir and Chute
Client Name	Meridian Energy
Client Contact	Brent Wilson
Client Project Reference	
Damwatch Project No. and Task	E2567

Document history and status

Issue No.	Issue Date	Description	Prepared by	Reviewed by	Approved by
1	2025-09-10	First issue in draft	GW	DCE	DCE
2	2025-10-16	Incorporate Meridian review comments and other updates	GW	DCE	DCE
3.1	2025-10-28	Incorporate Meridian final review comments, minor edits	GW	DCE	DCE

Current document approval

Prepared by

Reviewed by

Approved for issue Signature: [redacted]

Executive Summary

Overview and Background

This report presents the findings of a review of the original temporary tailrace weir and rock chute for Tekapo B Power Station. The weir and chute structure could potentially be exposed in the future if Lake Pūkaki is drawn down to allow use of contingent storage (or during dam safety incidents or emergencies, or construction-related work on Pūkaki outlet structures).

Prior to the Pūkaki High Dam being completed and Lake Pūkaki raised to its current operational level range, Tekapo B Power Station which discharges into Lake Pūkaki was operated with a temporary tailrace outlet. The temporary tailrace consisted of a weir to control water levels in the permanent upstream outlet channel and a 1 in 19.2 sloping rock-lined chute to drop water into Lake Pūkaki.

Design, Construction and Performance of Original Temporary Tailrace

The design and construction of the temporary tailrace is well documented with a range of historic documents available including:

- the original design report (MWD, 1979)
- the original construction specification (MWD, undated-a)
- the original operating instructions (MWD, undated-b)
- a technical note describing the construction and performance of the temporary tailrace published in the *Proceedings of the Institution of Civil Engineers (UK)* (Malan & Hancock, 1979)

The temporary tailrace was designed and constructed with the objectives of minimising construction costs and the need for skilled labour to construct it, and being easily repairable in the event of damage.

The weir component was formed with a concrete sill on the crest (sill 1) to maintain control of water levels in the upstream outlet channel. The nose of the weir and the downstream chute were lined with a layer of rock riprap material with an additional 10 concrete sills (sills 2-11) spanning the width of the structure to hold the rock lining in place.

Enough rock material meeting the median diameter d_{50} = 0.4 m design specification was available for lining the chute downstream of sill 4. Upstream of sill 4 larger sized rock material up to a maximum diameter of 2.4 m at the weir crest was used.

The rock lining for the chute component was perceived at the time of construction to be on the limits of stability at the design discharge of 122 m^3/s. In recognition of this, a stockpile of rock riprap material and gravel foundation material was required to be held on site to effect urgent repairs in the event of any damage occurring.

Two scour failures on the rock chute were experienced during operation of the temporary tailrace. One semicircular scour hole formed downstream of sill 5 after only 72 hours operation and remained stable over the next 8 weeks until winter generation at Tekapo B Power Station ceased. A larger scour hole formed downstream of sill 9 (where poor foundation conditions were encountered during construction) after 12 days operation at full station load and was rapidly repaired during a brief station shutdown.

Field measurements of the hydraulic performance and actual gradings for the rock lining of the chute were made (Thompson & Campbell, 1979) which provide valuable information. The flow rate and depth measurements demonstrated that a sub-critical flow regime existed down the rock chute for all flow conditions with a Froude number approaching 1 (marking the transition between sub-critical and supercritical flow regimes) at the highest flow rate. The measurements also enable the flow resistance to be quantified. Energy dissipation was evenly distributed over the whole channel, and the channel flow was highly aerated.

Overall, it was concluded that the concrete sills on the control weir and rock chute "*successfully maintained a well-defined waterway in spite of some movement of rock downstream*" and the formation of a couple of scour holes (Malan & Hancock, 1979).

Flow Regimes on Temporary Tailrace in Event of Contingent Storage Drawdown in Lake Pūkaki

A simple one-dimensional HEC-RAS computational hydraulic model was developed and used to predict water surface profiles over the length of the temporary tailrace for the following flow conditions representing a contingent storage drawdown situation in Lake Pūkaki:

- Tekapo B Power Station discharges of 65 m^3/s, 105 m^3/s and 130 m^3/s
- Drawdown levels in Lake Pūkaki of RL 520 m, RL 517 m and RL 514 m

At a Lake Pūkaki level of RL 520 m, the control weir at the head of the rock chute is nearly fully drowned for both discharge cases with only a small drop in water level occurring over the weir crest (predicted to be ≈ 4 cm for the 65 m^3/s flow case, ≈ 10 cm for the 105 m^3/s flow case and ≈ 11 cm for the 130 m^3/s flow case).

When the lake level drops to RL 517 m, the chute flow beyond the weir crest behaves as ramp-type flow (USBR, 2007) (Pagliara & Chiavaccini, 2006) under a high Froude number sub-critical flow regime before a sharp transition to the backwater profile corresponding to the constant lake level occurs (the flow behaviour at a lake level of RL 518 m would be very similar although the distance over which high Froude number sub-critical flow occurs would be lesser). The extent of the rock chute exposed to ramp-type flow conditions would extend for about 30-35 m and cover the bays between concrete sills 1-2 and 2-3 (refer to Figure 5.1).

When the lake level drops further to RL 514 m, the ramp-type flow down the chute extends further downstream than for a lake level of RL 517 m, to a distance of ≈ 91 m in the 65 m^3/s flow case and ≈ 97 m in the 130 m^3/s flow case. Thereafter, the constant level backwater profile of the lake dominates. The extent of the rock chute exposed to ramp-type flow conditions would cover the bays between concrete sills 1-2, 2-3 and 3-4 (refer to Figure 5.1).

The weir at the head of the rock chute controls water levels in the upstream outlet channel from the Tekapo B Power Station (now the permanent tailrace channel). Flow conditions will remain sub-critical in the outlet channel even when the level of Lake Pūkaki is drawn down.

Irrespective of the lake level when Lake Pukaki is drawn down below RL 518 m, tailwater levels at Tekapo B Power Station will be unchanged due to them being controlled by the crest level of the weir at the head of the rock chute (tailwater levels at Tekapo B Power Station under these conditions are a function of the power station discharge).

Hydraulic Assessment of Weir and Rock Chute with Use of Contingent Storage in Lake Pūkaki

The control weir and rock chute on the original Tekapo B Power Station temporary tailrace were assessed against the following aspects for a future situation when the contingent storage in Lake Pūkaki is utilised by drawing down the lake level:

- the stability of the rock chute under conditions of ramp-type flow
- the effects of sediment deposition on flow behaviour down the rock chute
- the effects of wind-generated waves on the lining of the rock chute

The first two bays of the rock chute were constructed using over-size rock material compared to what was specified with median boulder sizes of 0.85 m and 0.60 m for the bays between sills 1-2 and 2-3 respectively. Consequently, there should no uncertainty about the stability of the rock material in these bays if the level of Lake Pūkaki is drawn down to RL 515 m in the future and Tekapo B Power Station discharges up to a maximum of 105 m^3/s.

The median boulder size (d_{50}) in the next bay downstream (between sills 3-4) is much larger than the Isbash stability criterion ($d > 0.19$ m) for a flow of 111 m^3/s. Therefore, the rock material in this bay is very likely to remain stable if the level of Lake Pūkaki is drawn down to RL 513 m and chute flows up to 105 m^3/s are discharged.

Due to the fine sediment carried in suspension in the water column of Lake Pūkaki, it is possible that fine sediment deposition on the original rock chute may have occurred since use of the temporary tailrace ceased in 1979. If there is a thin layer of sediment deposition over the rock chute, this could initially reduce the relative roughness of the structure and hence the flow resistance, resulting in any ramp-type flow down the chute developing a slightly supercritical flow regime (which would be more prone to causing erosion damage to the structure). However, the highly turbulent nature of the ramp-type flow would rapidly erode any deposited fine sediment, allowing the normal slightly sub-critical flow regime to be restored.

The critical wind direction for wind generated waves affecting the rock chute on the temporary tailrace is from the northwest. Winds from this direction are predicted to be able to generate waves with a significant wave height of up to 2 m for a 1% AEP windstorm (NIWA, 2013). For a mean annual windstorm (42.9% AEP), the significant wave height would be more likely to be in the order of H_s = 1.5 m and the wave period in the range of T = 3-5 seconds.

Waves with a significant wave height of 1.5 m would not break before reaching the rock chute and would impact on the structure as plunging type breaking waves. The boulder sizes in the bays between sills 1-2, 2-3 and 3-4 ($d_{50} \geq 0.48$ m) are likely to be large enough to resist such wave impact.

Although of much smaller size, the boulders lining the bay between sills 4-5 and 5-6 are also likely to remain stable under the impact of plunging type breaking waves of significant height $H_s \leq 1.5$ m and period T = 3-5 seconds (representative of a mean annual wind speed condition) if the level of Lake Pūkaki is drawn down to RL 513 m.

There is no evidence of the rock chute on the temporary tailrace being damaged by wind generated waves when the level of Lake Pūkaki was below RL 518 m before 1979.

Contents

Appendices

List of abbreviations and symbols

Abbreviation / Symbol	Meaning
1D	One-dimensional
AEP	Annual exceedance probability
d_{15}	Particle size for which 15% of all particles by mass are smaller than
d_{50}	Median particle size (particle size for which 50% of all particles by mass are smaller than)
d_{75}	Particle size for which 75% of all particles by mass are smaller than
d_{85}	Particle size for which 85% of all particles by mass are smaller than
D/S	Downstream
Damwatch	Damwatch Engineering Limited
ECNZ	Electricity Corporation of New Zealand
Fr	Froude number
H_s	Significant wave height
LYTTHT1937 VD	Lyttelton Vertical Datum 1937
Meridian	Meridian Energy
MOW	Ministry of Works
NZED	New Zealand Electricity Department
RCM	Regional climate model
RL	Reduced level
T	Wave height
VD	Vertical Datum

1 Introduction

1.1 Purpose of Report

The purpose of this report is to present the findings of a review of the original temporary tailrace weir and rock chute for Tekapo B Power Station.

1.2 Background

The tailrace weir and rock chute formed a temporary structure which was used in the late 1970's to allow Tekapo B Power Station to generate power before the level of Lake Pūkaki was raised with the completion of the Pūkaki High Dam (see Figure 1.1). The weir and chute structure discharged into Lake Pūkaki.

Source: Roger Bennett Collection (provided by Meridian Energy)

Figure 1.1: Tekapo B Power Station and Canal with temporary tailrace weir and rock chute in operation at bottom right corner of photograph (August 1977)

The Pūkaki High Dam was commissioned in early 1979 and the lake filled to allow operation in the consented operating range of RL 518.0 m to RL 532.5 m. Currently the temporary tailrace structure is permanently submerged although, at low lake levels, the structure remains visible below the surface (see Figures 1.2 and 1.3).

Source: Brent Wilson, Meridian Energy

Figure 1.2: Tekapo B Power Station temporary weir and rock chute structure on 20 August 2024 - Lake Pūkaki level at RL 520.65 m

Figure 1.3: Aerial view from Google Earth of Tekapo B Power Station tailrace channel showing outline of submerged weir and rock chute structure (marked by red box)

Meridian Energy (Meridian) have applied for fast-track consenting approval to be able, in the next three years, to draw on contingent storage in Lake Pūkaki if there is a shortfall in the national electricity supply. In practice this would mean being able to draw the level of Lake Pūkaki down below the current consented minimum operating level of RL 518.0 m. Correlation of historical low winter inflows to Lake Pūkaki with operating limits on the Ohau A Reservoir System (Pūkaki Canal, Ohau Canal And Ohau A Canal) (Opus, 2013) at low canal levels suggests that the minimum level that Lake Pūkaki could practically be drawn down to is about RL 514 m. However, under rare circumstances during extremely poor inflow sequences (sustained <5th percentile), a low lake level of RL 513 m could be realised (pers. comm., Brent Wilson, Meridian Energy).

If the contingent storage in Lake Pūkaki is required to be used in the next three years during late winter when lake levels are historically low, then the original temporary tailrace weir and rock chute may come into partial operation (the weir starts to become effective when lake levels are drawn down below RL 520 m). This review of the original temporary structure aims to assess the likely hydraulic performance and structural stability of the structure to sustain any future operation under such conditions.

The Tekapo Scheme including Tekapo B Power Station is owned by Genesis Energy.

1.3 Scope of Review

The scope of the review included the following:

- Review of historical documentation contained in the Tekapo B Power Station Data Book (ECNZ, undated) including.
  - The original design report for the structure (MWD, 1979).
  - The construction specification for the structure (MWD, undated-a).
  - MOW Operating Instruction No. 307 Flow Control and Monitoring of Tailrace Extension (MWD, undated-b).
  - Other relevant MOW file correspondence.
- Review of a 1979 Journal of Hydraulic Research paper by Thompson and Campbell "Hydraulics of Large Channel Paved with Boulders" (Thompson & Campbell, 1979).
- Review of 1979 Proceedings of Institution of Civil Engineers technical note by Malan and Hancock "Construction and Performance of the Rock-Lined Chute at Tekapo B Power Station" (Malan & Hancock, 1979).
- Hydraulic assessment of the tailrace chute under conditions of contingent storage drawdown in Lake Pūkaki including:
  - Interpretation of the hydraulic operating regime of the weir and chute structure under conditions of contingent storage drawdown in Lake Pūkaki.
  - Construction of a simple 1D HEC-RAS computational hydraulic model of the weir and chute structure to assist the interpretation of the hydraulic behaviour under conditions of contingent storage drawdown in Lake Pūkaki.
  - Assessment of the likely performance and structural stability of the weir and chute structure under conditions of contingent storage drawdown in Lake Pūkaki based on evidence from its historical performance.

- Assessment of the structural stability of the weir and chute structure from exposure to wind-generated waves under conditions of contingent storage drawdown by drawing on previous wave studies carried out for Lake Pūkaki (NIWA, 2013).
- Preparation of a technical report on the findings of the review

1.4 Level Datum

All levels referenced in this report are expressed in terms of the Lyttelton Vertical Datum 1937 (LYTTHT1937)[1]. This a local mean sea level datum used to reference structure and water levels throughout the Waitaki Hydro System and the Tekapo Scheme.

Original construction levels for the Tekapo Scheme including the temporary tailrace weir and chute structure at Tekapo B Power Station are defined in terms of a local construction datum using imperial units of feet and inches. Structure levels in terms of this local construction datum and imperial units are translated to metric units and the LYTTHT1937 Vertical Datum by the following equation:

$$H\,m = 0.3048\,(\,h\,ft + 4.27)$$

[1] https://www.linz.govt.nz/guidance/geodetic-system/coordinate-systems-used-new-zealand/vertical-datums/local-mean-sea-level-datums-lvds

2 Description of Tailrace Weir and Rock Chute

2.1 Reference Drawings for Structure

Table 2.1 lists key reference drawings for the Tekapo B Power Station temporary tailrace weir and rock chute structure. These drawings are included in Appendix A.

Table 2.1: Key reference drawings for Tekapo B Power Station tailrace weir and chute structure

MOW Drawing No.	Drawing Title	Comment
L4438/6	Temporary tailrace plan and longitudinal section	Original design drawing superseded by as built drawing T38/01/6
L4438/7	Temporary tailrace details	Original design drawing superseded by as built drawings T38/01/7 & 8
T38/01/6	Temporary tailrace plan and longitudinal section	As-built drawing
T38/01/7	Temporary tailrace details – Sheet 1	As-built drawing
T38/01/8	Temporary tailrace details – Sheet 2	As-built drawing
PD 02/02/9	Lake Pūkaki bathymetric survey at Tekapo B temporary tailrace entry	Survey drawing

2.2 Construction of Structure

The weir and chute structure is comprised of three linked components (refer drawings T38/01/6-8 in Appendix A). Figure 2.1 shows an oblique aerial view of the structure in operation at a discharge of 100 m^3/s.

The three components of the weir and chute structure are detailed in Table 2.2.

The outlet channel is a compound trapezoidal-shaped earth-lined channel from the Tekapo B Powerhouse outlet to the weir at the head of the rock-lined chute. It has a straight alignment which turns sharply to the right at an angle of about 38 degrees at the weir component (see drawing T38/01/6). The channel has a constant base width of 12.2 m, a uniform side-slope on the right side and a variable side-slope on the left side, and a constant invert level of RL 514.58 m. The outlet channel now forms the permanent tailrace channel for Tekapo B Power Station.

The weir at the head of the chute is a broad-crested weir with a trapezoidal-shaped long-section. The weir crest is formed by a concrete sill, 3.05 m wide and 3.26 m deep (see drawing T38/01/8). The crest (and concrete sill) length is 45.7 m. The 2:1 (horizontal to vertical) side-slopes of the weir crest are rock-lined on a free-draining gravel base (see drawing T38/01/7). The sloping nose of the weir is also rock-lined with gravel and sand filter layers underneath (see drawing T38/01/7).

Source: Thompson & Campbell (1979)

Figure 2.1: Oblique aerial view of Tekapo B Power Station temporary tailrace channel in operation discharging 100 m^3/s into Lale Pūkaki (date and time of photograph unknown but lake level inferred from as-built drawings to be below level of sill 11 at RL 502.23 m)

Table 2.2: Components of Tekapo B Power Station temporary tailrace channel

Tailrace Component	Chainages	Cross-Section Shape
Outlet channel	67.97 – 332.38	Non-uniform compound trapezoidal
Weir	332.38 – 343.96	Uniform trapezoidal
Rock-lined chute	343.96 – 708.05	Uniform trapezoidal

The chute downstream of the weir features concrete sills (beams) spaced every 30.5 m over the length of the structure. The spaces between the sills are lined with rock boulders overlying gravel and sand filter layers. The overall length of the chute is 364 m. It has a slope of 19.3:1 (horizontal to vertical except over the last 61 m at the downstream end where it increases to 8:1 (see drawing T38/01/6). The base width of the chute is 39.6 m and the side-slopes 2.07:1 (horizontal to vertical).

2.3 Detailed Geometry of Structure

The detailed geometry of the temporary tailrace channel is summarised in Table 2.3. The three different components of the tailrace channel are marked in the table by different colour shading. The dimensions and levels for the weir and chute components were sourced from as-built drawings T38/01/6-8. The dimensions and levels for the upstream outlet channel component were sourced from the original design drawings L4438/6-7 so will be indicative only of as-built dimensions and levels.

The uniform trapezoidal cross-section shape of the weir and chute components is defined by paired level and width values at the base and top.

The compound trapezoidal cross-section shape of the outlet channel is defined by a set of four paired level and width values as shown on Figure 2.2.

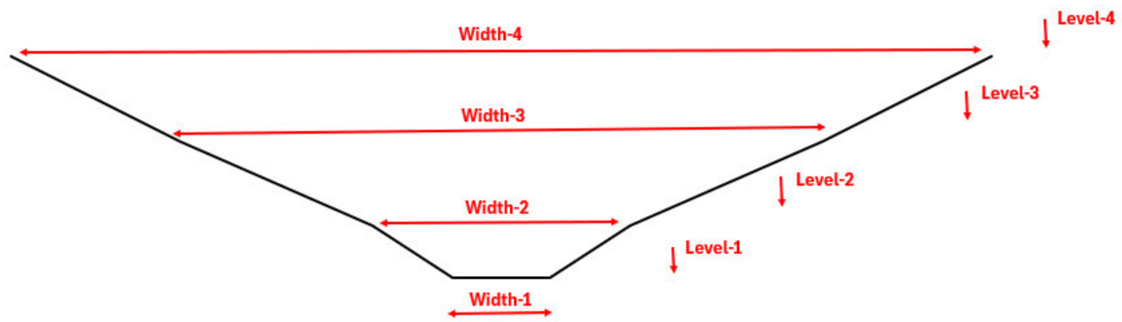


Figure 2.2: Definition of cross-section geometry of compound trapezoidal shaped upstream outlet channel component of Tekapo B Power Station temporary tailrace channel

The temporary tailrace channel geometry data in Table 2.3 were utilised for the development of a simple one-dimensional (1D) HEC-RAS computational hydraulic model of the temporary tailrace channel as described in Section 4.

Table 2.3: Detailed geometry of Tekapo B Power Station temporary tailrace channel

Tailrace Component	Drawing Ref	Cross-section shape	Distance (m)	Level-1 (RL m)	Width-1 (m)	Level-2 (RL m)	Width-2 (m)	Level-3 (RL m)	Width-3 (m)	Level-4 (RL m)	Width-4 (m)
outlet channel	L4438/6	Compound trapezoidal	67.97	514.58	12.19	516.41	32.31	519.46	80.47	522.51	123.14
	L4438/6	Compound trapezoidal	113.69	514.58	12.19	516.41	32.31	519.46	80.47	522.51	123.14
	L4438/6	Compound trapezoidal	159.41	514.58	12.19	516.41	32.31	519.46	80.47	522.51	123.14
	L4438/6	Compound trapezoidal	205.13	514.58	12.19	516.41	32.31	519.46	80.47	522.51	123.14
	L4438/6	Compound trapezoidal	220.37	514.58	12.19	516.41	32.31	519.46	80.47	522.51	123.14
	L4438/6	Compound trapezoidal	250.85	514.58	12.19	516.41	32.92	519.46	73.76	522.51	114.00
	L4438/6	Compound trapezoidal	296.57	514.58	22.56	516.41	32.92	519.46	57.30	522.51	89.00
start of weir	L4438/6	Compound trapezoidal	332.38	514.58	45.72	519.46	57.91	522.51	79.25		
weir crest - sill 1	T/38/01/7	Trapezoidal	340.92	517.85	45.72	520.90	57.91				
weir crest - sill 1	T/38/01/7	Trapezoidal	342.44	517.85	45.72	520.90	57.91				
weir crest - sill 1	T/38/01/7	Trapezoidal	343.96	517.85	45.72	520.90	57.91				
rock chute - sill 2	T/38/01/7	Trapezoidal	372.77	516.43	39.62	519.78	53.04				
rock chute - sill 3	T/38/01/7	Trapezoidal	403.25	514.87	39.62	518.22	53.04				
rock chute - sill 4	T/38/01/7	Trapezoidal	433.73	513.14	39.62	516.50	53.04				
rock chute - sill 5	T/38/01/7	Trapezoidal	464.21	511.72	39.62	515.08	53.04				
rock chute - sill 6	T/38/01/7	Trapezoidal	494.69	509.91	39.62	513.26	53.04				
rock chute - sill 7	T/38/01/7	Trapezoidal	525.17	508.60	39.62	511.95	53.04				
rock chute - sill 8	T/38/01/7	Trapezoidal	555.65	506.96	39.62	510.32	53.04				
rock chute - sill 9	T/38/01/7	Trapezoidal	586.13	505.17	39.62	508.53	53.04				
rock chute - sill 10	T/38/01/7	Trapezoidal	616.61	503.90	39.62	507.26	53.04				
rock chute - sill 11	T/38/01/7	Trapezoidal	647.09	502.23	39.62	505.58	53.04				
rock chute	T/38/01/7	Trapezoidal	708.05	494.61	39.62	497.96	53.04				

3 Review of Historical Documentation for Structure

3.1 List of Historical Documentation

The following historical documents were reviewed as part of this review:

- The original design report for the structure (MWD, 1979).
- The original construction specification for the structure (MWD, undated-a).
- The original operating instructions for the structure (MWD, undated-b).
- Other file correspondence from the Tekapo B Power Station Data Book (ECNZ, undated).
- A technical note published in the *Proceedings of the Institution of Civil Engineers (UK)* (Malan & Hancock, 1979).
- A technical paper published in the *Journal of Hydraulic Research* (Thompson & Campbell, 1979).

3.2 Design Report (MWD, 1979)

The following notes provide a summary of the key points of the original design report for the temporary tailrace channel (MWD, 1979). This report focused solely on the weir and chute components.

- The introduction to the report provides the background to the need for a temporary tailrace channel facility at Tekapo B Power Station. Unusually dry weather and electricity demand during the preceding year "*resulted in Lake Pūkaki being some 16 m below the minimum tailwater level for the Tekapo B Power Station at commissioning*". Previously, in anticipation of this occurring, it had been decided to construct a temporary tailrace facility incorporating a weir to ensure satisfactory tailwater levels and a channel to convey the powerhouse discharge down to Lake Pūkaki. A July 1976 report investigated a range of channel options (pipes, flumes, a mild-sloped channel and a steep-sloped channel), with the latter option being selected.
- The design parameters for the temporary tailrace facility were:
  - a minimum tailwater level of RL 518 m was required to be maintained at the Tekapo B powerhouse outlet under the minimum flow condition of half the output from one power station unit (i.e. 30 m^3/s)
  - the maximum design discharge was 122 m^3/s
  - a minimum rock size for the riprap material of 0.6 m was advised to the Upper Waitaki Power Project
- Ground topography dictated the location of the weir at the head of the chute. The weir was designed as a broad-crested weir with a concrete crest to protect the crest level. The main body of the weir was designed to be formed from impermeable materials (field permeability 1×10^{-7} m/s). This impermeable material extended upstream as a blanket to make the seepage path under the weir structure as long as possible. Side-slopes were flat and protected with riprap material. Graded filter materials were specified between natural ground, the impermeable weir core and the riprap material to prevent the occurrence of internal erosion of materials.
- The chute was designed with a width of 40 m and a slope of 0.05 (20:1 horizontal to vertical). The design assumed that enough minimum diameter 0.6 m riprap material was available. The specification (MWD, undated-a) including grading envelopes for the riprap, gravel filter and sand sub-base was attached in an appendix although not incorporated in the Tekapo B Power Station Data Book (ECNZ, undated).

- Various design formulae for the stability of rock-lined channel were considered: Isbash, Oliver, Shield's entrainment function, and Hartung and Scheuerlein. The latter expression was discarded as it assumed tightly wedged stones on steep slopes where self-aeration of the flow is significant (note Figure 2.1 shows highly aerated flow conditions at a discharge of 100 m^3/s). The other three design formulae were applied with most confidence being placed in Isbash's equation which predicts the maximum average flow velocity under which a boulder will remain stable. The use of these design formulae required a hydraulic resistance relationship to be adopted. Two resistance relationships were considered (Strickler and Straub) which both relate the Manning's *n* channel roughness coefficient for a natural channel to the d_{75} size of the bed material to the power of one sixth. i.e.

$$n = constant\ x\ {d_{75}}^{1/6}$$

- For a boulder diameter of 0.6 m, flow velocities down the chute were estimated to be in the range of 3.8-4.7 m/s (with Froude numbers Fr in the range 1.4-1.9) for the design flow of 122 m^3/s compared to Isbash's flow velocity of 3.8-5.3 m/s for a stable channel. Similarly, for a boulder diameter of 0.4 m, flow velocities down the chute were estimated to be in the range of 3.8-5.0 m/s (with Froude numbers Fr in the range 1.4-2.1) for the same design flow compared to Isbash's flow velocity of 3.8-4.3 m/s for a stable channel. These predictions implied a slightly supercritical flow regime down the chute with the rock lining being on the margins of stability.
- Because of the marginal stability of the rock-lined channel, concrete sills were placed 30 m apart flush with the channel bed to limit bed movement. It was anticipated that, if failure of the rock lining was to occur, it would occur rapidly with the sills limiting the extent of bed erosion. The power station outflow could then be shut off to allow repairs to be made. Repairs would be facilitated by a stockpile of suitable riprap material.
- The channel was extended 30 m further into the lake to a level of approximately RL 494.5 m to provide protection to the lakebed where it was expected that the supercritical flow would cause a hydraulic jump to form.
- The compatibility criteria for the filter layers between the riprap and sub-base materials were based on well-established design guidelines.
- The actual median diameter of the rock riprap material was noted to be 0.4 m (Thompson and Campbell (1979) indicate this was true downstream of sill 4 – refer to Table 5.1).
- The report summary notes that the tailrace weir performed satisfactorily. This is discussed further below in reviews of Malan and Hancock (1979) and Thompson and Campbell (1979).

3.3 Construction Specification (MWD, undated-a)

The original construction specification (MWD, undated-a) covered the riprap, gravel, sand and core and blanket materials as well as the compatibility requirements between materials.

The riprap material was required to be:

- Reasonably well graded with a maximum size of 0.61 m (a grading was attached but this is missing from the Tekapo B Power Station Data Book (ECNZ, undated))
- Composed of hard, dense and durable rock fragments with a preference for rounded particles to be avoided

- Placed by dumping and then graded off to provide an even distribution of larger rock fragments with smaller fragments filling the voids, thereby resulting in a uniform and closely packed riprap layer

The gravel material was required to be:

- Well graded from sands to cobbles of 0.150 m diameter, but without excess fines (a grading was attached but this is missing from the Tekapo B Power Station Data Book (ECNZ, undated))
- Placed in layers not exceeding 0.305 m in depth and compacted to a relative density of at least 65%

The moisture content of the gravel material and the placement methods were required to minimise segregation into parts and breakages of particles.

The sand material was required to be:

- A uniformly graded material conforming to the grading envelope attached to the specification (missing from the copy of the specification in the Tekapo B Power Station Data Book (ECNZ, undated))
- Placed and compacted in a moist condition with care taken to avoid clogging and contamination of the sand layer by passing construction traffic

The core and blanket materials were required to:

- Have a field permeability of less than 1 x 10^{-7} m/s
- Be compacted by rollers to at least 95% of its Proctor density and within ± 2.5% of its optimum moisture content
- not be allowed to dry out

The following compatibility criteria between adjacent materials were specified:

- The ratio d_{15} of coarser material / d_{85} base material was to be not greater than 5
- The ratio d_{15} of coarser material / d_{15} base material was to be not greater than 40

3.4 Operating Instructions (MWD, undated-b)

The original operation instructions (MWD, undated-b) specified how the temporary tailrace facility was required to be operated:

- The instructions were applicable when the level of Lake Pūkaki fell below RL 518.37 m.
- While the structure was being operated, the flow over the weir and down the chute was required to be observed at least three times daily. These observations were required to check for any signs of erosion, unusual turbulence, asymmetric flow behaviour, bypassing of the formed channel or any other abnormal conditions.
- A procedure was to be in place to enable the rapid shutoff or reduction in Tekapo B Power Station discharge if adverse flow conditions were observed in the weir and chute section of the temporary tailrace outlet.
- Tekapo B Power Station outflows were to be limited to the values specified in the table below:

Lake Pūkaki Level (RL m)	Maximum Allowable Tekapo B Power Station Outflow (m^3/s)
502.2	0
502.5	24
502.8	49
503.1	73
503.3	97
503.7	122
> 503.7	122

- Construction plant was to be positioned so that stockpiled riprap material could be dumped in the tailrace chute without delay to repair any observed damage.
- Any stockpiled material that was utilised for damage repairs was to be replaced so that the following approximate volumes of material were maintained:
  - riprap material 1500 m^3
  - gravel material 750 m^3

3.5 Other MOW File Correspondence

Other MOW file correspondence about the Tekapo B Power Station temporary tailrace outlet is included in the Tekapo B Power Station Data Book (ECNZ, undated). Key points from this correspondence are summarised below:

- The New Zealand Electricity Department (NZED) was considering the need for temporary tailrace outlet at Tekapo B Power Station as early as June 1976 as it undertook a detailed study of the filling of Lake Pūkaki and its subsequent management.
- NZED formally requested MOW to construct a temporary tailrace outlet to permit operation of Tekapo B Power Station at a Lake Pūkaki level of RL 501.17 m in November 1976. Subsequently NZED approved two design drawings (refer to drawings L4436/6 & 7 in Appendix A).
- A further letter from NZED to MOW in January 1977 noted that it appeared possible the level of Lake Pūkaki might not reach the bottom end of the temporary tailrace outlet by the commissioning date of May 1997 for Tekapo B Power Station. The letter requested MOW to "*plan resources and materials and to construct the tailrace chute for operation of Tekapo B with Pūkaki at RL 486 m*". The letter further requested the design and construction of the extended tailrace chute to permit future re-use. i.e. allow the level of Lake Pūkaki to be lowered again after it had been raised. Of the two possible chute designs, the letter expressed a preference for the two-slope version and agreed that a permanent concrete structure was not possible.
- A letter from MOW to NZED in February 1978 noted that "*so far, the tailrace has performed satisfactorily as designed and constructed, under conditions of local rising and falling lake levels*". It was expected that that the tailrace would continue to perform as in the past after complete submergence of the structure. In respect of the request for confirmation of the practicality of using the Tekapo B Power Station tailrace outlet on a falling lake level after the outlet had been flooded, the letter noted that wind-generated wave attack is "*in the opposite direction from the*

usual tailwater attack on the bed". Detrimental erosion of the structure could occur if a storm event coincided with the level of Lake Pūkaki partway up the chute. The letter recommended that the level of Lake Pūkaki should be held for prolonged periods only above RL 518 m and below RL 502.2 m.

3.6 Historic Lake Level and Flow Records

Figure 3.1 shows the historic lake level record for the period over which Lake Pūkaki was being filled. The level of RL 502.23 m coincides with the bottom end of the chute component of the temporary tailrace outlet where the bed slope increases sharply to 8:1 (horizontal to vertical). The level of Lake Pūkaki remained above this level for 609 days except for a brief period from September to November 1977 when the lake level dropped back down to a minimum of about RL 498 m.

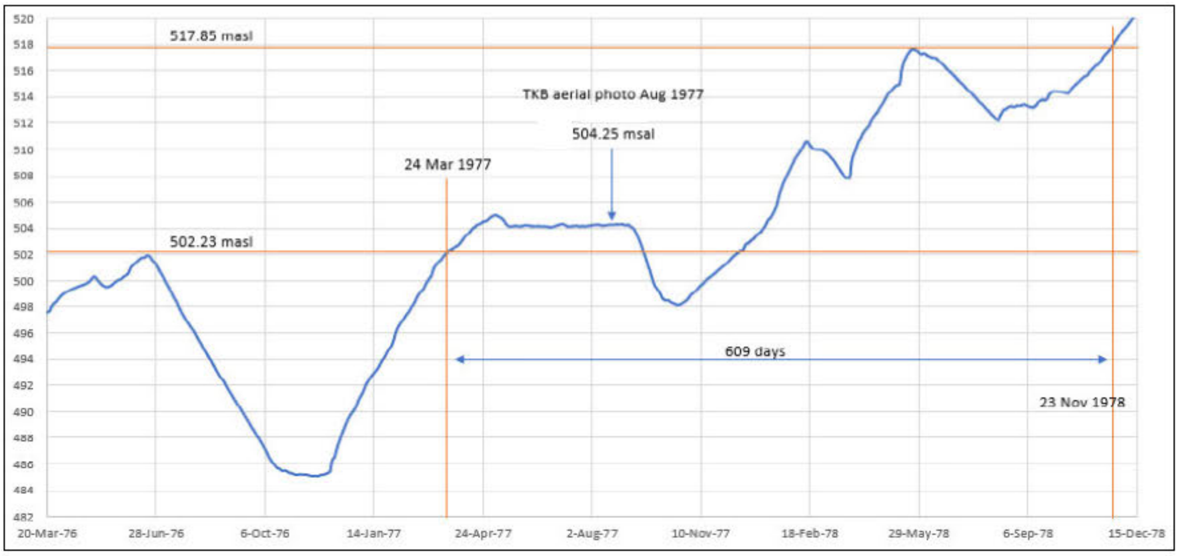


Figure 3.1: Historic lake level record for Lake Pūkaki from March 1976 to December 1978

Tekapo B Power Station was commissioned in June 1977. Figure 3.2 shows the power station outflow record from June 1977 (when the power station was commissioned) to December 1978 when the tailrace channel was fully flooded with lake levels rising above RL 518 m permanently. Except for a period from October 1977 to January 1978, Tekapo B Power Station was operated at full load for a significant proportion of the time (noting that at full load, the maximum station flow achieved was about 105 m^3/s).

The lake level record is also included on the graph in Figure 3.2.

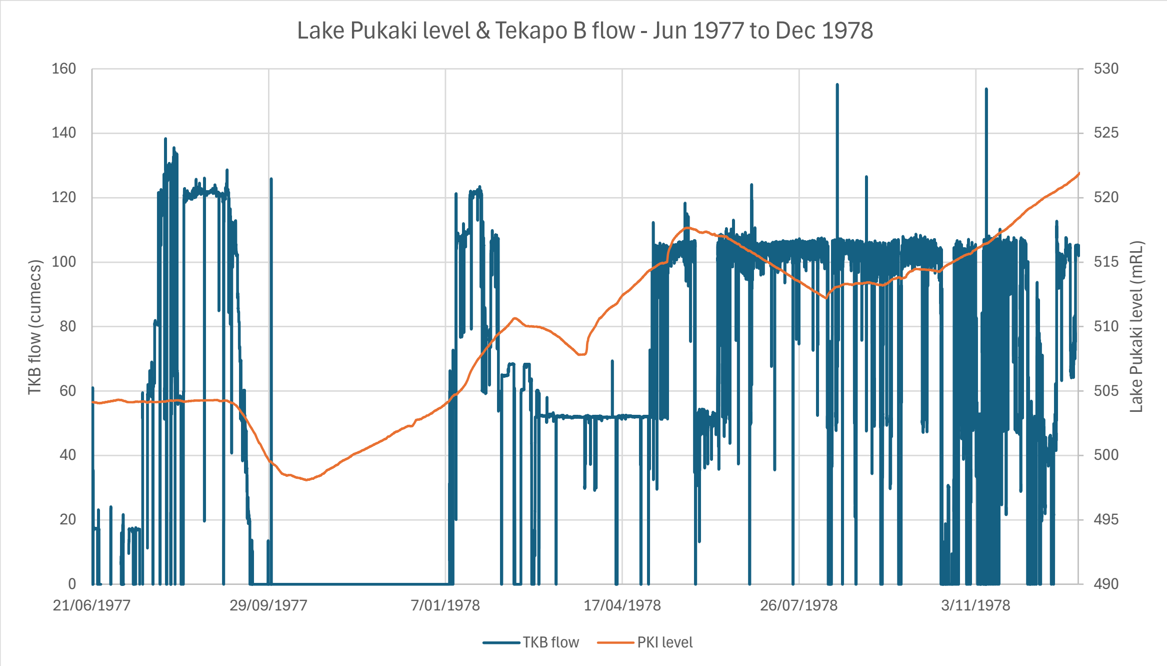


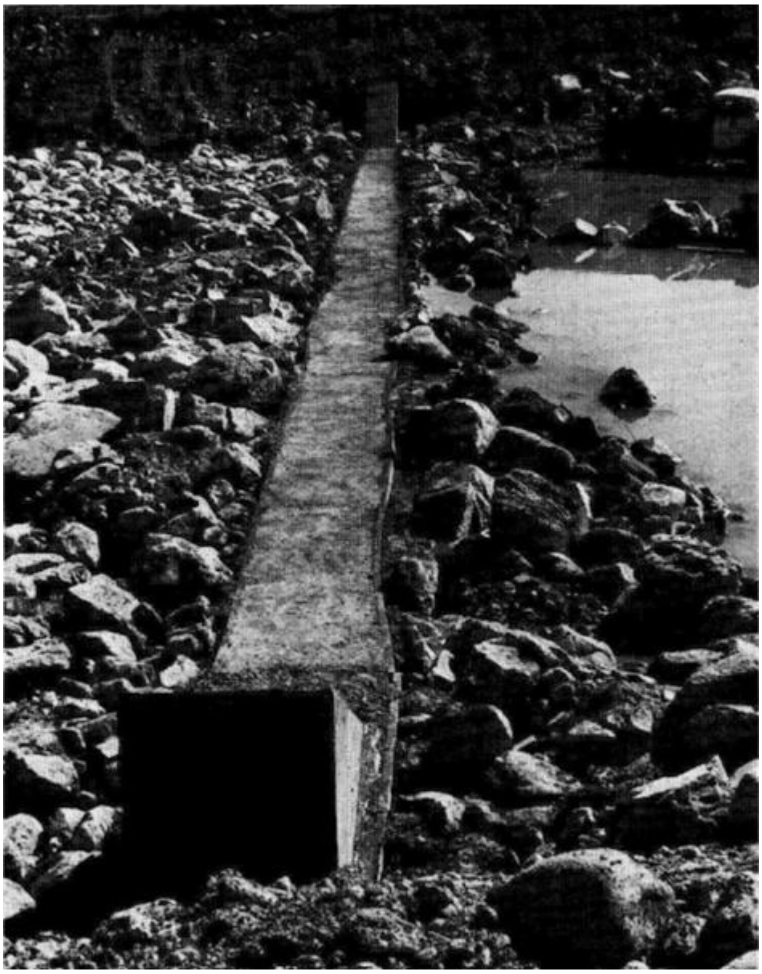
Figure 3.2: Tekapo B Power Station outflow and Lake Pūkaki level from June 1977 to December 1978

3.7 Malan and Hancock (1979)

The construction and performance of the rock-lined chute is described in a detailed technical note published in the *Proceedings of the Institution of Civil Engineers (UK)* (Malan & Hancock, 1979). Key points from this paper about the construction of the chute are as follows:

- The chute was designed as a temporary structure with construction objectives of minimising costs and the need for skilled construction labour. Due to the inexpensive nature of the structure, it had a "*correspondingly high risk of failure*" and was considered "*an appropriate design for a short-life structure*".
- The original design of the chute called for 13 concrete sills spaced 30.5 m apart over the length of the structure. The sills were numbered from the control weir at the top of the chute (the concrete sill on the weir crest was numbered as 1).
- It was noted that the "*performance under load indicates that the rock lining is close to instability and also indicates an extreme case of the threshold for sediment transport*" (the companion paper by Thompson and Campbell (1979) is referenced in this respect).
- The chute cut through an alluvial fan resting on a 4 m thick layer of blue-grey clayey silt. This silt layer intercepted the chute invert diagonally between the left side of concrete sill 10 and the right side of concrete sill 11.
- Groundwater levels were high in the chute invert below concrete sill 8 with springs emerging downstream. These flows, combined with drainage from the powerhouse area, caused severe gully erosion in the fine sand foundation material.
- Because of the poor foundation materials and the rapid rise in lake level at the time of construction, it was decided to omit concrete sills 12 and 13 and start the steeper 8:1 slope from sill 11. Sill 11 was also widened from the standard 1.12 m to 1.42 m with the top of the sill formed to a 14:1 slope (this being the average of the upstream 20:1 slope and the downstream 8: 1 slope).

- The three lower sills (9, 10 and 11) were constructed in very wet ground. This is where a subsequent scour failure occurred. The wet ground conditions required a different construction technique for the concrete sills. Due to the rapid liquefaction of the sands from groundwater flows, a slurry replacement method was used with low-slump concrete discharged into an excavated trench. This continued until the liquified sand in the trench was fully replaced by concrete. This method seemed reasonably successful in producing a continuous concrete sill.
- After the concrete sills were formed, earthworks in the bays between sills were carried out. Placement of the sand filter, gravel and rock riprap layers occurred in sequence, working progressively up from the lower end of the structure to the upper end. After the chute invert had been completed, the side-slopes of the structure were formed.
- Enough rock riprap material meeting the design specification was available for placement along the chute invert downstream of concrete sill 5. Upstream of sill 5, larger sized rock material was used up to a maximum of 2.4 m at the control weir (Table 5.1 summarises rock size data from Thompson and Campbell (1979) indicating larger sized material only upstream of sill 4). The side-slopes were covered in whatever size boulders could be sourced with the material generally being larger than the specified 0.76 m. Figure 3.3 shows a view of a completed concrete sill with the size of rock riprap material clearly evident.



Source: Malan and Hancock (1979)

Figure 3.3: View of a completed concrete sill across the width of the chute component of the Tekapo B Power Station temporary tailrace channel with the rock lining upstream and downstream visible (top width of the sill is 1.12 m)

The temporary chute structure was commissioned with a sequence of steadily increasing constant flows (Malan & Hancock, 1979):

- Stage 1 (19 June 1977) – 4 hours at 31 m^3/s
- Stage 2 (19 July 1977) – 3.5 hours at 61 m^3/s
- Stage 3 (26 July 1977) – 72 hours at 81 m^3/s
- Stage 4 (29 July 1977) – continuous at 122 m^3/s

For the first two stages, the flows were observed continuously. For the second two stages, the flows were observed at 2-hour intervals. Observations were focused on picking up signs of bed erosion, high turbulence, asymmetric flow behaviour, breakout from the formed channel and any other abnormal flow conditions. A stockpile of rock riprap material was on hand to facilitate immediate repairs if required. Visual inspection of the structure for any signs of bed movement was carried out after Stages 1-3 and after an appreciable period of continuous operation in Stage 4.

A semi-circular scour hole measuring 14 m in diameter and 3 m deep formed immediately downstream of sill 5 after 72 hours operation at 122 m^3/s in Stage 4. This scour hole remined stable over the next 8 weeks until winter generation at Tekapo B Power Station ceased.

After twelve days operation at full load (122 m^3/s), a major scour hole measuring 24.4 m in diameter and 4.9 m deep immediately downstream of sill 9 was found after a routine inspection during a shutdown of Tekapo B Power Station for minor unit repairs. This scour hole was filled in with gravel material and riprap material, care being taken to ensure that the gravel material penetrated under the exposed base of the concrete sill. This repair performed satisfactorily over the remaining period of operation of the temporary chute structure.

Before either of the scour holes formed, it was generally observed that some movement of the riprap lining occurred downstream of concrete sill 5. Lining material tended to be eroded in the lee of each sill and then piled up upstream of the next sill downstream, effectively causing a slight flattening of the bed in the bay between the two sills. This effect occurred to a much lesser extent upstream of sill 5 where the size of the rock lining material was larger.

Malan and Hancock (1979) concluded that the concrete sills "*successfully maintained a well-defined waterway in spite of some movement of rock downstream*" with the formation of a couple of scour holes, only one of which required repair. They do not say whether the first scour hole downstream of sill 5 self-healed with the movement of rock material from upstream bays although the size of the hole to be filled suggests this was unlikely to have occurred). The construction of the temporary tailrace outlet facility allowed Tekapo B Power Station to be commissioned on time.

3.8 Thompson and Campbell (1979)

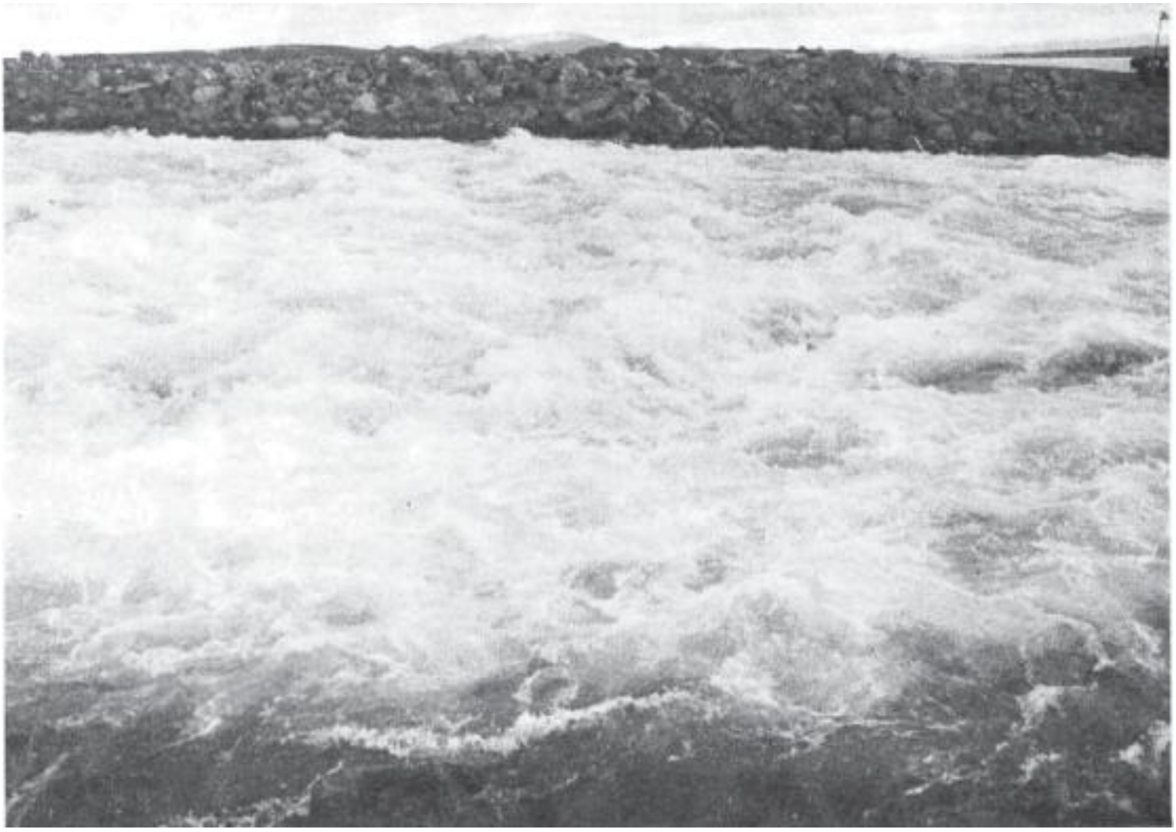
3.8.1 Overview

Thompson and Campbell (1979) presented the results of an analysis of some field measurements of flow depths down the rock-lined chute in a *Journal of Hydraulic Research* paper. The depth observations were made at each of the concrete sills spaced 30.5 m (100 ft) apart along the length of the chute.

The motivation for the field measurements and subsequent analysis was that:

> "*It was feared that the flow might be supercritical with a violent dissipation zone where the channel met the lake, leading to channel failure from the downstream end. Alternatively it was feared that subcritical flow with large depth and consequently large shear stresses would cause failure of the weaker parts*."

The paper noted that the flow proved to be sub-critical over the length of the chute with flow depths in the range of 0.5-1.5 m. Energy dissipation appeared to be evenly distributed over the whole channel as seen in Figure 2.1 with the flow aerated on the surface (as indicated by the white water). Figure 3.4 provides a close-up view of the aerated flow between concrete sills 4 and 5 while Figure 3.5 shows a view of the channel bed at the same location.



Source: Thompson and Campbell (1979)

Figure 3.4: Close-up view of aerated flow between concrete sills 4 and 5 on the rock-lined chute component of the Tekapo B Power Station temporary tailrace channel

Source: Thompson and Campbell (1979)

Figure 3.5: View of rock-lined bed between concrete sills 4 and 5 on the chute component of the Tekapo B Power Station temporary tailrace channel

3.8.2 Depth Measurements and Flow Resistance

Table 3.1 summarises the field measurements made by Thompson and Campbell (1979) over a range of flows between 60 m^3/s and 140 m^3/s. The depth values in Table 3.1 represent the average measured flow depths at both ends of sills 1 to 5 corrected for a bias due to the flow depth over each sill being observed from photographs to be slightly higher than the flow depth over the upstream bed.

The analysis of the field data in Table 3.1 assumed the following constant parameter values:

- channel width W = 41 m
- channel slope S = 0.052
- median rock size d_{50} = 0.4 m

Table 3.1: Summary of field data from the rock-lined chute component of the Tekapo B Power Station temporary tailrace channel measured by Thompson and Campbell (1979)

Flow Q (m^3/s)	Depth R (m)	Relative Roughness R/d_{50}	Darcy-Weisbach Friction Factor F	Manning's Channel Roughness Coefficient n	Flow per Unit Width q (m^2/s)	Average Flow Velocity V (m/s)	Froude Number Fr
60	0.79	1.98	0.94	0.105	1.46	1.85	0.67
82	0.88	2.2	0.69	0.092	2.00	2.27	0.77
111	1.00	2.5	0.56	0.084	2.71	2.71	0.87
125	1.04	2.6	0.49	0.080	3.05	2.93	0.92
140	1.09	2.73	0.45	0.077	3.42	3.13	0.96

The flow resistance of the rock-lined chute was analysed by evaluating Darcy-Weisbach friction factor *f* values using the equation

$$f = \frac{8\, g\, R^3\, S\, W^2}{Q^2}$$

where *g* is the gravitational acceleration, *R* is the flow depth, *S* is the channel slope, *W* is the channel width and *Q* is the flow rate (i.e. discharge).

The Darcy-Weisbach friction factor *f* values can be translated into more conventional Manning's *n* channel roughness values using the equation (Henderson, 1966)

$$n = \frac{f}{\sqrt{8g}}\, R^{\frac{1}{6}}$$

The Darcy-Weisbach friction factor *f* and Manning's *n* channel roughness factor are measures of the flow resistance of a channel due to bed friction. These parameters are function of the relative roughness of the channel surface R/d_{50} where d_{50} is the median particle size of the rock lining for the channel (the relative roughness is the ratio of the flow depth to the median particle size).

Figure 3.6 shows the Manning's *n* roughness coefficient values derived from Thompson and Campbell's (1979) field data plotted as a function of relative roughness R/d_{50}. The Manning's *n* values can be seen to decrease as flow depth, *R* and hence flow rate, *Q* increase:

- For a flow of 60 m^3/s and a relative roughness R/d_{50} value of 1.98 (see Table 3.1), the Manning's channel roughness value for the chute is just over 0.10; and
- For a flow of 125 m^3/s and a relative roughness R/d_{50} value of 2.6 (see Table 3.1), the Manning's channel roughness value for the chute is 0.080.

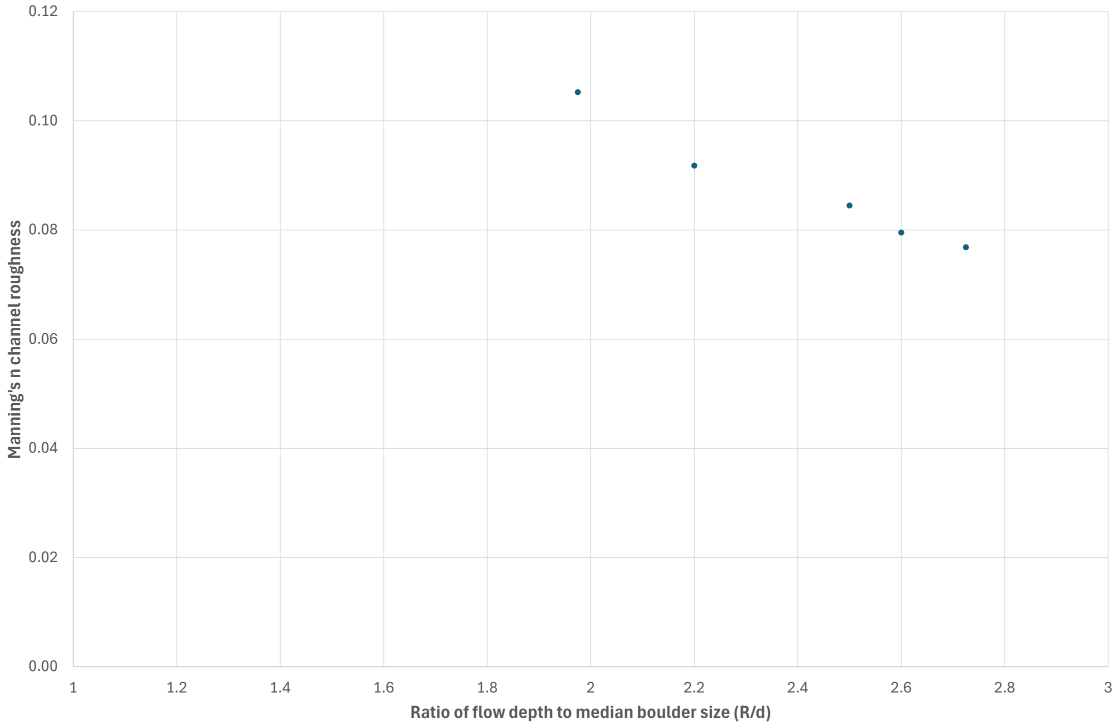


Figure 3.6: Mannings n channel roughness as a function of relative roughness R/d_{50} from Thompson and Campbell's (1979) field measurements

The Froude number *Fr* indicates whether channel flow is sub-critical (*Fr* < 1) or supercritical (*Fr* > 1). Figure 3.7 shows the Froude number *Fr* values derived from Thompson and Campbell's (1979) field data plotted as a function of flow depth R. The data indicate that:

- at the lowest flow for which data were available (60 m^3/s), the Froude number *Fr* was about 0.67; and
- for the higher flows, the Froude number *Fr* was approaching a value of 1 (close to trans-critical flow conditions).

For flow conditions in most steep gravel-bed rivers in New Zealand, the Froude number *Fr* typically has a value less than about 0.7 (Hicks & Mason, 1991).

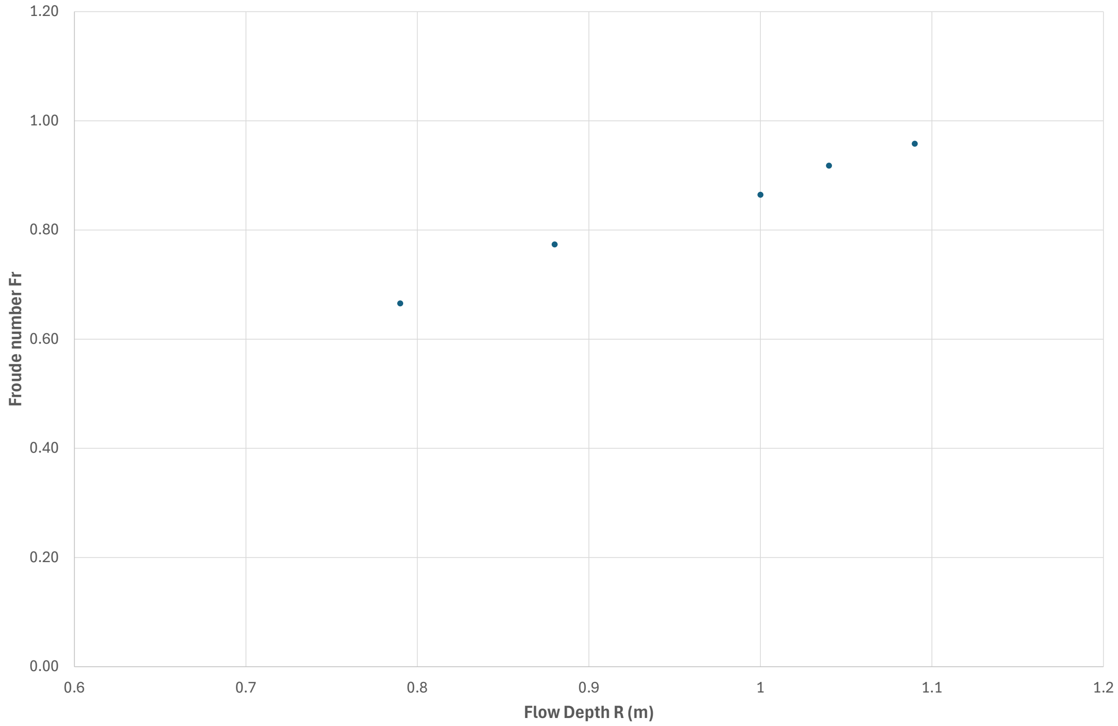
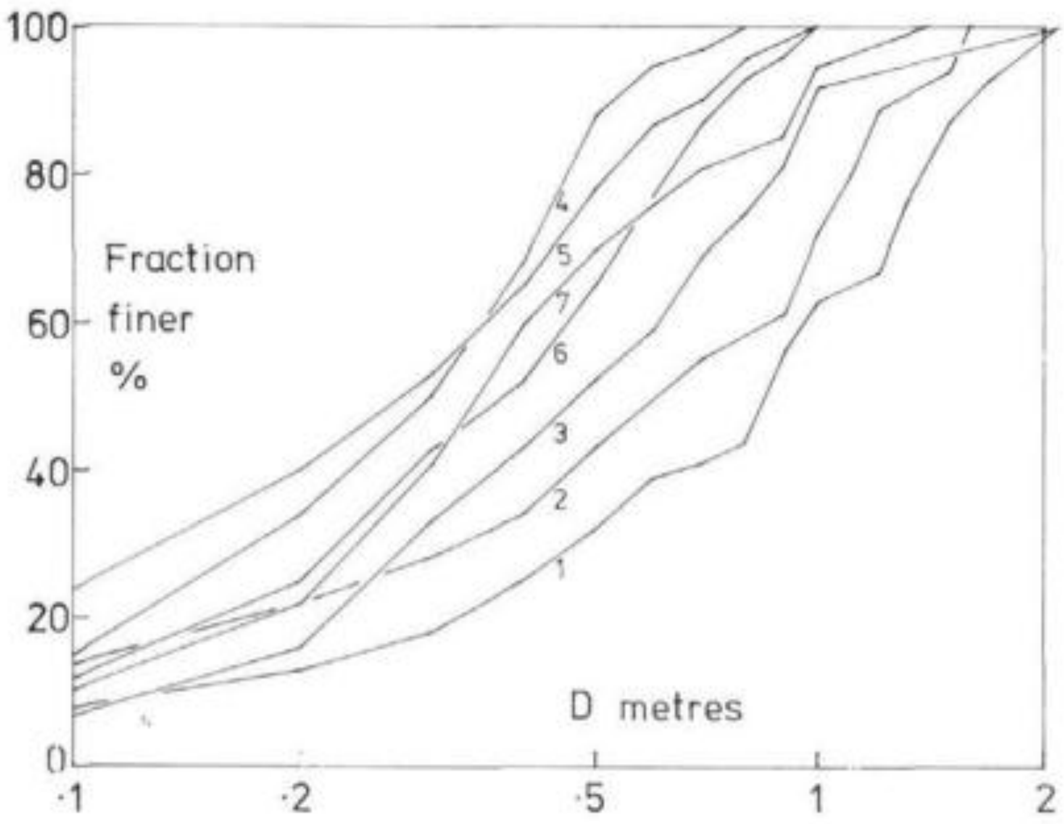


Figure 3.7: Froude number of chute flow as a function of flow depth from Thompson and Campbell's (1979) field measurements

3.8.3 Size of Rock Lining Material

Thompson and Campbell (1979) provide other useful measurements of the actual size grading of the rock lining of the chute in the absence of the required grading from the original construction specification (MWD, undated-a). Figure 3.8 shows the envelope of particle size gradings obtained from transect measurements of the rock lining material using the Wolman method. Boulder size distributions were obtained for the rock lining of the chute between sills 1 and 8 (the concrete sills were numbered from the top of the chute).



Source: Thompson and Campbell (1979)

Figure 3.8: Measured boulder size distributions between concrete sills 1 and 8 on the rock-lined chute component of the Tekapo B Power Station temporary tailrace channel

The grading curves in Figure 3.8 indicate a spread of boulder sizes. The median boulder size d_{50} varied from a low value of 0.27 m in the bay between sills 5 and 6 (grading curve number 5) to a high value of 0.85 m in the bay between sills 1 and 2 (grading curve number 1). The overall average d_{50} was estimated to be 0.40 m based on all measurements. Note d_{15}, d_{50} and d_{85} boulder sizes are tabulated in Table 5.1.

The ratio d_{85}/d_{50} which is used as typical measure of the size of the largest boulders relative to the median boulder size was generally in the range of 1.6 to 2.4 with an average value of 2.0.

Of relevance to this review is that the boulder size distribution in the first two bays between sills 1 and 2 and 2 and 3 was generally coarser (d_{50} values of 0.85 m and 0.60 m respectively) than further down the chute.

3.8.4 Rock Lining Stability

Thompson and Campbell (1979) compared pairs of vertical photographs of the rock lining of the chute taken just prior to the first flow and after one month of flow passing down the chute when the paving had stabilised. Key observations from these photographs were:

- About half of the area of the chute bed was occupied by groups of large boulders, identifiable in both before and after photographs, and some of these groups had in places moved up to 3 m downstream.

- No significant modification of the rock lining had occurred in the first bay between concrete sills 1 and 2 where the lining was comprised of larger boulders (d_{50} = 0.85 m).
- Between concrete sills 2 and 8, large areas of the stabilised paving consisted of a tight mosaic of boulders measuring between 0.2 m and 0.6 m across. It was inferred that the original paving had been modified by the flow to arrange the mosaic (like the bed armouring process in a gravel-bed river). The downward drift of paving material to form this armoured mosaic surface layer resulted in a depressed surface in the lee of each concrete sill.
- A local failure of the rock lining had occurred at sill 9 where a large scout hole formed underneath the sill. This scour hole required repair.

The stability of the rock lining was explored using the Isbash method referenced in the original design report (refer to Section 3.2). Isbash's equation defines the smallest boulder size *d* that will not move when dropped through flowing water onto a stable bed of similar-sized material:

$$d = \frac{0.25\, Q^2}{g\, W^2\, R^2}$$

Application of this equation to the Tekapo B Power Station temporary tailrace chute structure for the largest measured power station discharge in Table 3.1 of 140 m^3/s indicated a boulder size $d > 0.25$ m was required for stability. It was noted that 30% of the chute area was paved with boulders smaller than this size although nowhere was the median boulder size smaller than this value.

Application of the same equation to the measured power station discharges of 125 m^3/s and 111 m^3/s (see Table 3.1) indicates boulder sizes of $d > 0.22$ m and $d > 0.19$ m respectively for channel stability.

4 Simple Computational Hydraulic Model of Tailrace Weir and Chute

4.1 Development of Model

A simple one-dimensional (1D) HEC-RAS (USACE, 2002) computational hydraulic model of the temporary tailrace facility for Tekapo B Power Station was constructed based on the geometry summarised in Table 2.3. The purpose of the model was to demonstrate the hydraulic behaviour of the original tailrace facility in a future situation if the contingent storage in Lake Pūkaki is utilised with the lake level drawn down below RL 518 m.

The HEC-RAS model incorporated the three components of the temporary tailrace facility – the outlet channel (now the permanent tailrace channel for Tekapo B Power Station), the control weir and the rock-lined chute. As noted in Section 2.3:

- The geometry of the control weir and rock-lined chute was based on as-built dimensions and levels (drawings T38/01/6-8 in Appendix A); and
- The geometry of the upstream outlet channel was based on the design dimensions and levels (drawings L4438/6&7 in Appendix A).

The use of design dimensions and levels for the upstream outlet channel rather than as-built dimensions is not critical as the as-built channel geometry on drawing T38/01/6 appears to be fairly similar to the design channel geometry on drawing L4438/6, and the hydraulic behaviour of this part of the model will not be affected significantly. The as-built and design channel invert levels and widths are the same, the right bank side-slopes are the same and only the left bank side-slopes are slightly different (the left bank side-slope varies slightly along the length of the structure in the as-built channel whereas it was designed as an approximately constant side-slope in the design channel).

The HEC-RAS model was used to predict water surface profiles along the length of the temporary tailrace facility for selected boundary conditions representing a contingent storage drawdown situation in Lake Pūkaki.

4.2 Boundary Conditions

If the contingent storage available in Lake Pūkaki is required to be utilised in the future to bolster a shortfall in the national electricity supply, the minimum anticipated level that Lake Pūkaki could be drawn down to is about RL 514 m. However, under rare circumstances in extremely poor inflow sequences (sustained <5th percentile, a low lake level of RL 513 m could be realised (pers. comm., Brent Wilson, Meridian Energy)[2].

Three equally spaced levels for Lake Pūkaki between RL 520 and RL 514 were selected as downstream boundary conditions for a series of HEC-RAS model simulations. At a lake level of RL 520 m, the

[2] As noted in Section 1.2, this is based on a correlation of historical low winter inflows to Lake Pukaki with operating limits on the Ohau A Reservoir System (Pukaki, Ohau and Ohau A Canals) (Opus, 2013).

control weir is just starting to control water levels in the upstream outlet channel. A lake level of RL 514 m is likely to be the lowest that Lake Pūkaki could be drawn down to if the lake is utilised for contingent storage as noted above.

Tekapo B Power Station has two power generation units and is capable of discharging a maximum flow of 130 m^3/s through both units (Opus, 2014). However, as the net head on the units increases when the level of Lake Pūkaki is drawn down below RL 518 m, the maximum station flow required to maintain a constant output of 160 MW reduces to 105 m^3/s.

An upstream flow boundary condition was applied to the HEC-RAS model using flows of 65 m^3/s, 105 m^3/s and 130 m^3/s.

Table 4.1 summarises the selected boundary conditions which were applied to the HEC-RAS model.

Table 4.1: Summary of boundary conditions for HEC-RAS model simulations

Simulation No.	Upstream Boundary Condition Flow (m^3/s)	Downstream Boundary Condition Lake Pūkaki Level (RL m)
1	65	520
2	65	517
3	65	514
4	105	520
5	105	517
6	105	514
7	130	520
8	130	517
9	130	514

4.3 Other Input Parameters

The other key input parameter required for the HEC-RAS model simulations was the definition of the Manning's n channel roughness coefficient. Table 4.2 summarises the Manning's n values selected for the different components of the temporary tailrace facility under conditions of contingent storage drawdown in Lake Pūkaki.

Table 4.2: Summary of selected Manning's n channel roughness values for HEC-RAS model of Tekapo B Power Station temporary tailrace facility

Tailrace Component	As-Built Channel Chainage (m)	Manning's n Value		
		Q = 65 m^3/s	Q = 105 m^3/s	Q = 130 m^3/s
Outlet channel	69.97 – 332.39	0.035	0.035	0.035
Control weir	332.39 -343.97	0.100	0.085	0.080
Rock-lined chute	343.97 – location of flow regime transition	0.100	0.085	0.080
	location of flow regime transition – 708.05	0.050	0.050	0.050

The Manning's n value of 0.035 selected for the outlet channel is a typical value for a natural gravel-bed river channel. However, the model is unlikely to be sensitive to this value as the water levels in the channel are controlled by the crest level of the control weir at the head of the rock-lined chute.

From the control weir down to where the chute flow meets the lake level (and where there is a sharp change in flow regime change), the Manning's n value is based on the values shown in Figure 3.6 derived from Thompson and Campbell's (1979) field measurements. The Manning's n value selected therefore depends on the magnitude of the flow.

Where the backwater effect of the lake dominates the flow profile on the rock-lined chute, the selected Manning's n value of 0.050 is a nominal value only as it has negligible influence on the flow profile.

4.4 Results of Model Simulations

Figures 4.1-4.3 show the predicted backwater profiles for the following scenarios respectively:

- a tailrace flow of 65 m^3/s and Lake Pūkaki levels of RL 514 m, RL 517 m and RL 520 m;
- a tailrace flow of 105 m^3/s and Lake Pūkaki levels of RL 514 m, RL 517 m and RL 520 m; and
- a tailrace flow of 130 m^3/s and Lake Pūkaki levels of RL 514 m, RL 517 m and RL 520 m.

At a Lake Pūkaki level of RL 520 m, the control weir at the head of the rock chute is nearly fully drowned for both flow cases with only a small drop in water level occurring over the weir crest (predicted to be ≈ 4 cm for the 65 m^3/s flow case, ≈ 10 cm for the 105 m^3/s flow case and ≈ 11 cm for the 130 m^3/s flow case).

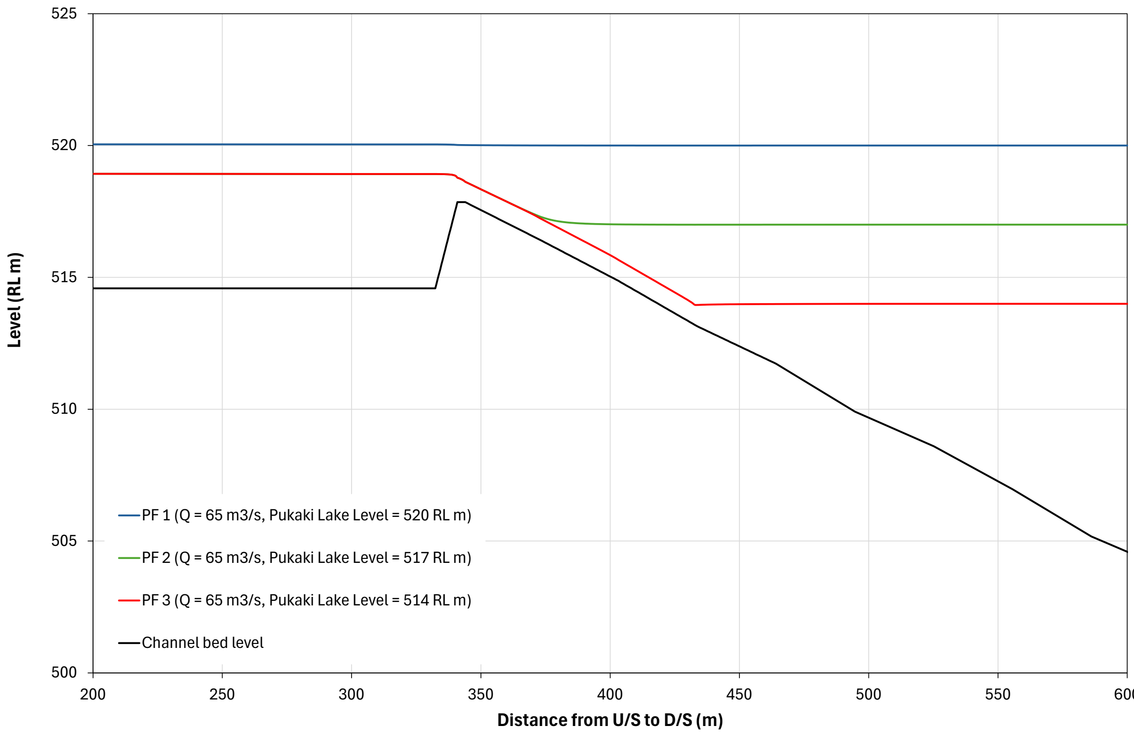


Figure 4.1: Predicted backwater profiles for Tekapo B Power Station temporary tailrace channel flow of 65 m^3/s and Lake Pūkaki levels of 514 m, 517 m and 520 m

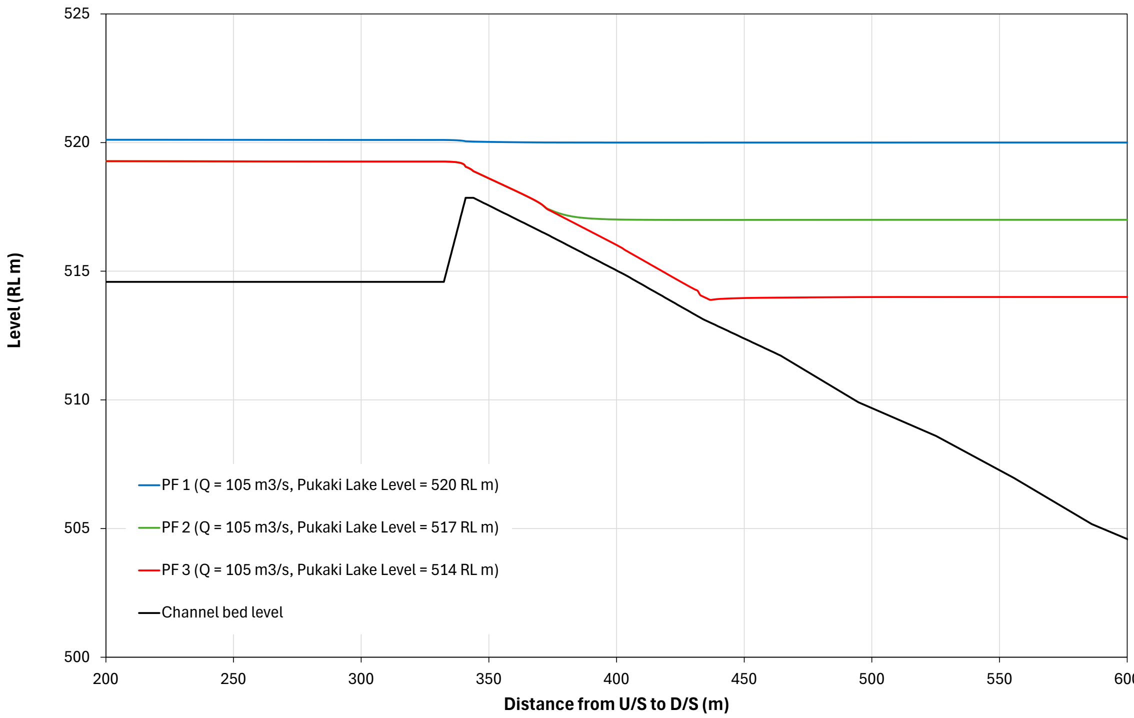


Figure 4.2: Predicted backwater profiles for Tekapo B Power Station temporary tailrace channel flow of 105 m^3/s and Lake Pūkaki levels of 514 m, 517 m and 520 m

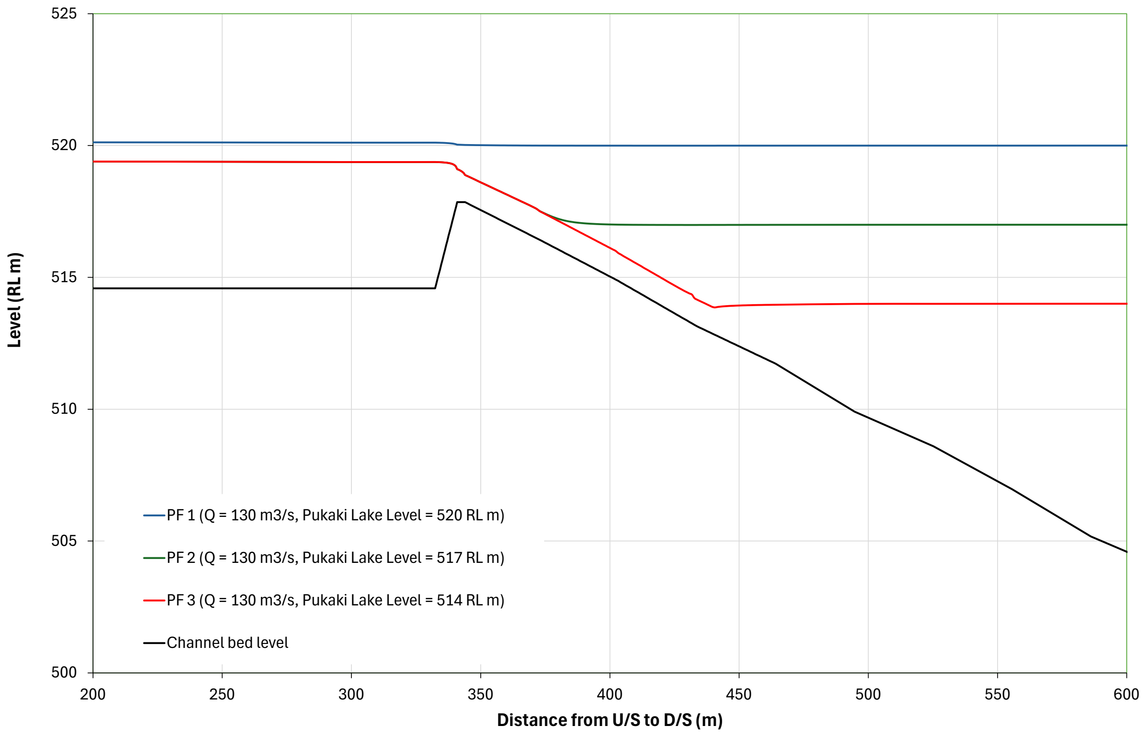


Figure 4.3: Predicted backwater profiles for Tekapo B Power Station temporary tailrace channel flow of 130 m^3/s and Lake Pūkaki levels of 514 m, 517 m and 520 m

When the lake level drops to RL 517 m, the chute flow beyond the weir crest behaves as ramp-type flow (USBR, 2007) (Pagliara & Chiavaccini, 2006) as shown in Figures 2.1 and 3.4 before a sharp transition to the backwater profile corresponding to the constant lake level occurs. The flow behaviour at a lake level of RL 518 m would be very similar although the distance over which the ramp-type flow occurs would be lesser.

When the lake level drops further to RL 514 m, the ramp-type flow down the chute extends further downstream than for a lake level of RL 517 m, a distance of ≈ 91 m in the 65 m^3/s flow case and ≈ 97 m in the 130 m^3/s flow case. Thereafter, the constant level backwater profile of the lake dominates.

If in a very extreme situation the lake level dropped even further to RL 513 m, the backwater profile would be very similar to that for a lake level of RL 514 m shown in Figures 4.1, 4.2 and 4.3.

Table 4.3 summarises predicted flow depths and velocities for the ramp-type flow for the three flow cases immediately before the transition to the constant level backwater profile of the lake.

Table 4.3: HEC-RAS model predicted flow depths and velocities down rock-lined chute immediately before constant level backwater profile from Lake Pūkaki dominates flow profile

Flow Parameter	65 m^3/s Flow Case		105 m^3/s Flow Case		130 m^3/s Flow Case	
	Lake Pūkaki Level RL 517 m	Lake Pūkaki Level RL 514 m	Lake Pūkaki Level RL 517 m	Lake Pūkaki Level RL 514 m	Lake Pūkaki Level RL 517 m	Lake Pūkaki Level RL 514 m
Flow depth R (m)	0.83	0.80	1.01	0.97	1.05	1.01
Flow velocity (m/s)	1.82	1.98	2.49	2.61	2.64	3.09
Froude number	0.65	0.72	0.81	0.87	0.84	1

Figure 4.4 compares the predicted flow depths from the HEC-RAS model in Table 4.3 against the flow depth / discharge rating curve derived from Thompson and Campbell's (1979) measured field data in Table 3.1. As the HEC-RAS model uses Manning's n channel roughness values derived from Thompson and Campbell's (1979) measured data, we would expect to see good agreement between the HEC-RAS model predicted flow depths and the rating curve. Figure 4.3 confirms this to be true although:

- the HEC-RAS model predicted flow depth at a chute flow of 105 m^3/s for a Lake Pūkaki level of RL 517 m is slightly higher than the rating curve indicates; and
- the HEC-RAS model predicted flow depth at a chute flow of 130 m^3/s for a Lake Pūkaki level of RL 514 m is slightly lower than the rating curve indicates.

In the latter case, Figure 4.3 indicates that the HEC-RAS model predicts a Froude number value of 1 (right at the transition between sub-critical and supercritical flow regimes). The algorithm within HEC-RAS may have had some difficulty solving the relevant flow equations for this flow case with the flow conditions predicted to be at the transition between the two flow regimes (Thompson and Campbell's (1979) measured data indicate a Froude number value of about 0.9 for this flow case with a flow depth of 1-1.05 m, i.e. slightly sub-critical – refer to Figure 3.7).

Irrespective of the lake level when Lake Pukaki is drawn down below RL 518 m, tailwater levels at Tekapo B Power Station will be unchanged as they are controlled by the crest level of the weir at the head of the temporary tailrace chute (tailwater levels at Tekapo B Power Station are a function of the power station discharge). This is illustrated by Figures 4.1, 4.2 and 4.2 where the backwater profiles in the permanent tailrace outlet channel upstream of the control weir are identical for lake levels of RL 514 m and 517 m.

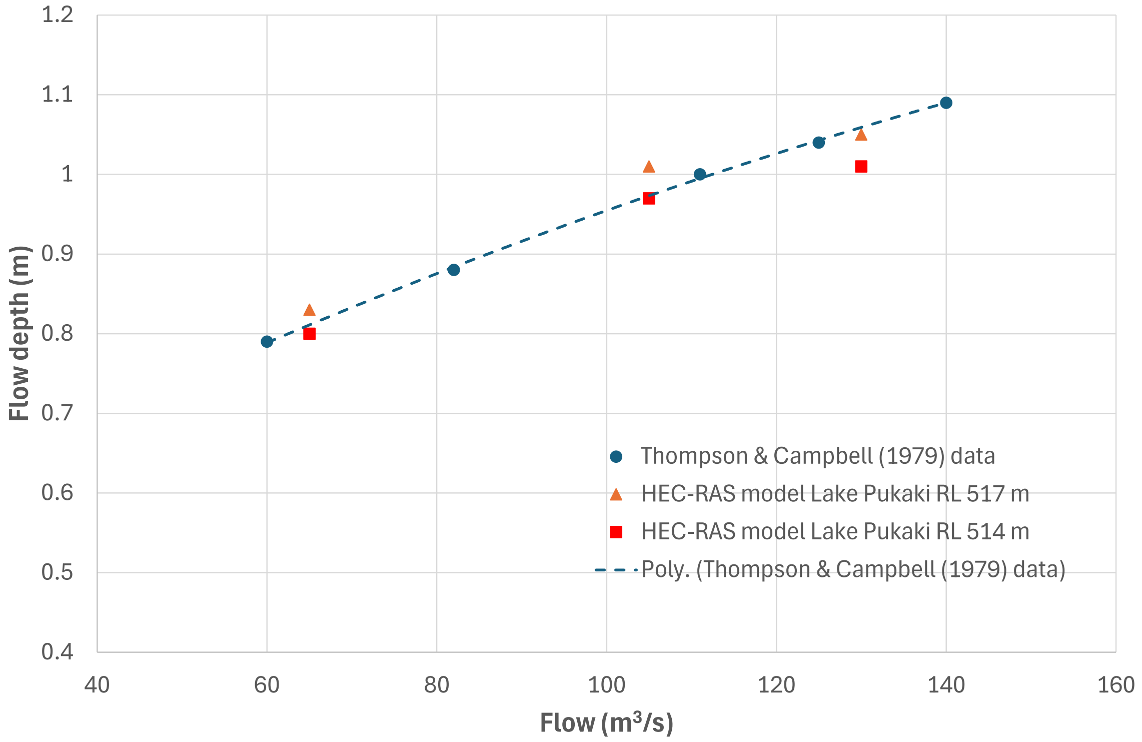


Figure 4.4: Comparison of predicted flow depths from HEC-RAS model against flow depth / discharge rating curve derived from Thompson & Campbell's (1979) measured data

For all flow conditions likely to be experienced if the contingent storage in Lake Pūkaki is drawn on in the future, channel flow down the exposed part of the rock-lined chute is very likely to be sub-critical but approaching the transition to a supercritical regime (i.e. with Froude numbers *Fr* in the range of 0.7-0.95).

With aerated flow conditions as seen in Figures 2.1 and 3.4, the tendency would therefore be for the chute (or ramp-type) flow to exhibit a high Froude number (Fr→1) sub-critical flow regime.

5 Hydraulic Assessment of Rock Chute with Use of Contingent Storage in Lake Pūkaki

5.1 Overview

In the following section, the control weir and rock-lined chute on the original Tekapo B Power Station temporary tailrace are assessed against the following aspects for when the contingent storage in Lake Pūkaki is utilised in the future by drawing down the lake level:

- the stability of the rock chute under conditions of channel flow
- the effects of sediment deposition on flow behaviour down the rock chute
- the effects of wind-generated waves on the lining of the rock chute

Note that the weir structure, on the basis of its configuration, is part of the first part of the chute and is not considered separately.

5.2 Stability of Rock Chute under Ramp-Type Flow Conditions

Figure 5.1 shows an extract of the as-built long-section of the control weir and rock chute on the original Tekapo B Power Station temporary tailrace. On this image, the sill numbers (numbered from the control weir crest) have been marked along with distances measured from the downstream edge of the sill on the control weir crest to the centreline of sills 1-7.

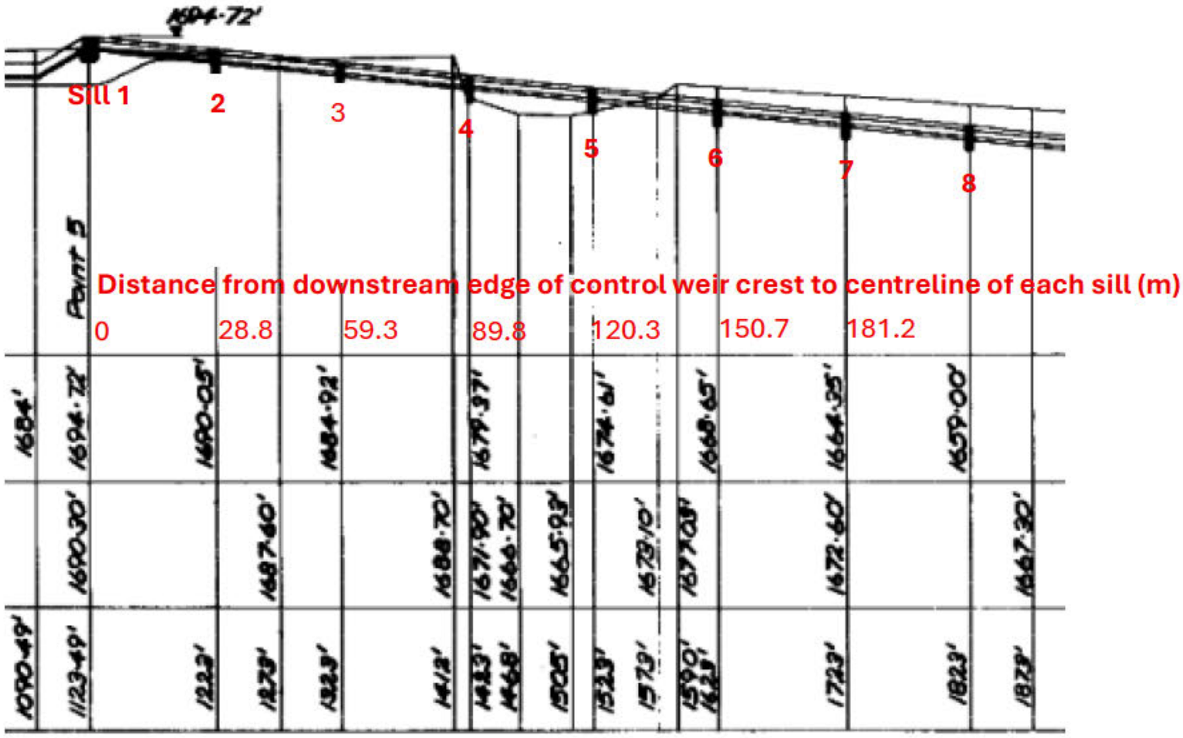


Figure 5.1: Extract from MOW drawing T 38/01/6 showing long section of top end of rock-lined chute on Tekapo B Power Station temporary tailrace (note original dimensions and levels in imperial units)

Based on the predictions of the HEC-RAS model presented in Section 4.4:

- the bays between sills 1-2 and 2-3 would be exposed to ramp-type flow conditions (USBR, 2007) (Pagliara & Chiavaccini, 2006) if the level of Lake Pūkaki was drawn down to RL 517 m; and
- the bays between sills 1-2, 2-3 and 3-4 would be exposed to ramp-type flow conditions if the level of Lake Pūkaki was drawn down to RL 513 m.

Table 5.1 summarises boulder sizes in each bay from the control weir crest down to sill 6 based on tabulated and graphical information sourced from Thompson and Campbell (1979) (see Figure 3.8). The green shaded rows in the table indicate those bays exposed if the level of Lake Pūkaki is drawn down to RL 517 m while the red shaded rows indicate the additional bays exposed if the level of Lake Pūkaki is drawn down to RL 513 m.

Table 5.1: Boulder sizes in each bay of rock chute on original Tekapo B Power Station temporary tailrace from Thompson & Campbell (1979)

HEC-RAS Model Distance (m)	Distance from D/S Edge of Control Weir Sill (m)	Bay Between Sills	Lake Level Range (RL m)	Boulder Size (m)		
				d_{15}	median d_{50}	d_{80}
343.96 – 372.77	0.0 – 28.8	1-2	516.43 – 517.85	0.23	0.85	1.44
372.77 - 403.25	28.8 – 59.3	2-3	514.87 – 516.43	0.16	0.60	1.15
403.25 – 433.73	59.3 – 89.8	3-4	513.14 – 514.87	0.18	0.48	0.93
433.73 – 464.21	89.8 – 120.3	4-5	511.72 – 513.14	0.12	0.30	0.48
464.21 – 494.69	120.3 – 150.7	5-6	509.91 – 511.72	< 0.1	0.27	0.57

As noted in Sections 3.7 and 3.8.3, the first two bays of the rock chute were constructed using over-size rock material compared to what was specified (refer to Section 3.3). This is confirmed by the boulder size (d_{15}, d_{50} and d_{84}) data given in Table 5.1. This indicates that there should no uncertainty about the stability of the rock lining if the level of Lake Pūkaki is drawn down to RL 515 m in the future and Tekapo B Power Station discharges up to a maximum of 105 m^3/s.

The median boulder size (d_{50}) in the next bay downstream (between sills 3-4) is much larger than the Isbash stability criterion for a flow of 111 m^3/s (d > 0.19 m) discussed in Section 3.8.4. Therefore, the rock lining of the chute is very likely to remain stable if the level of Lake Pūkaki is drawn down to RL 513 m.

Overall, the tailrace chute should remain stable under Tekapo B Power Station discharges up to 105 m^3/s if the level of Lake Pūkaki is drawn down to RL 513 m.

5.3 Effects of Sediment Deposition on Flow Behaviour in Rock Chute

Due to the fine sediment carried in suspension in the water column of Lake Pūkaki, it is possible that fine sediment deposition on the original rock chute on the Tekapo B Power Station temporary tailrace may have occurred since use of the temporary tailrace ceased in 1979. Without access to recent bathymetric survey data for the rock chute, it is not possible to quantify the extent and depth of sediment deposition. However, in view of the clear visibility of the outline of the chute structure in the aerial photograph in Figure 1.3, any sediment deposition is not likely to have completely buried the structure.

If there is a thin layer of sediment deposition over the rock chute, this could initially reduce the relative roughness of the structure and hence the flow resistance (expressed in terms of the Manning's n channel roughness coefficient). This would result in any ramp-type flow down the chute with a lowered lake level (due to contingent storage drawdown in Lake Pūkaki) developing a slightly supercritical flow regime which would be more prone to causing erosion damage to the structure). However, the highly turbulent nature of the ramp-type flow would rapidly erode any deposited fine sediment causing the relative roughness of the structure to revert to its 1979 condition and the flow resistance characteristics measured by Thompson and Campbell (1979) to be restored. The slightly sub-critical flow regime of the ramp-type flow would also be restored.

If low lake level conditions were to persist for several weeks due to contingent storage drawdown, the hydraulic behaviour of the rock chute would be similar to that which was observed by Thompson and Campbell (1979) when the structure formed part of the temporary tailrace facility for Tekapo B Power Station.

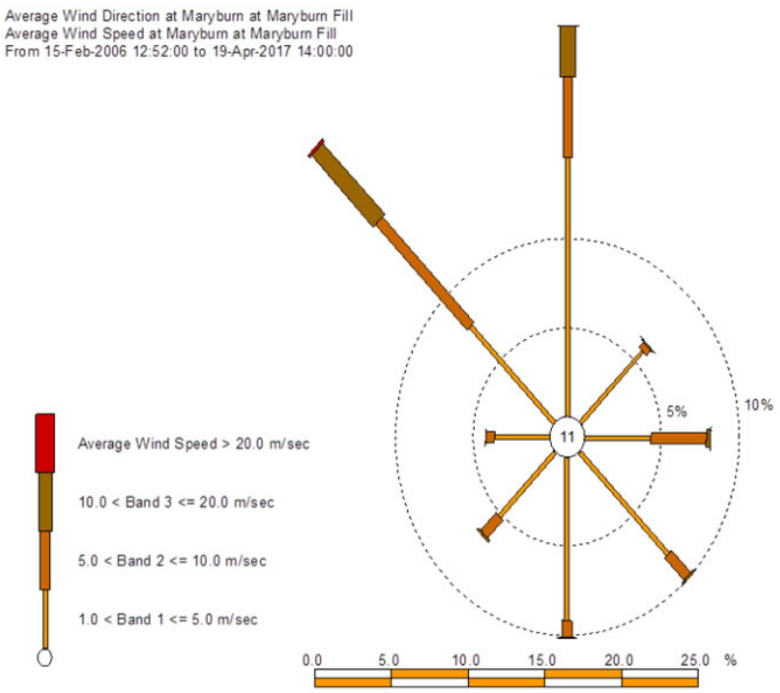
5.4 Effects of Wind Generated Waves

5.4.1 Critical Wind Direction for Tekapo B Power Station Tailrace

If the rock chute component of the original Tekapo B Power Station temporary tailrace is exposed in the future due to contingent storage drawdown of Lake Pūkaki, the rock lining will potentially be exposed to a structural loading exerted by wind-generated waves. This loading was recognised with the original design of the structure (refer to Section 3.5) although the stability of the rock lining under wave attack was not specifically assessed.

Figure 5.2 shows a wind rose for the nearest climate station Maryburn at Maryburn Fill based on a February 2006 to April 2017 wind record (Damwatch, 2018). This site is considered the most representative for the wind climate affecting Lake Pūkaki from a review of available current and historical wind records (NIWA, 2013).

The wind rose in Figure 5.2 indicates the predominant wind directions affecting Lake Pūkaki are from the northwest and from the north.



Source: Damwatch (2018)

Figure 5.2: Wind rose for Maryburn at Maryburn Fill climate station based on February 2006 to April 20017 record

Figure 5.3 shows an extract of the 1:50,000 scale topographic map covering the southern part of Lake Pūkaki (the grid lines on the map indicate the north-south and east-west directions). The orientation of the eastern lakeshore at the site of Tekapo B Power Station means that the original temporary tailrace outlet will largely be sheltered from waves generated by northerly winds except for waves that are diffracted around the headland to the north. However, any diffracted waves will be smaller than the corresponding wave heights in open water for waves generated by wind coming from this direction.

The critical wind direction for wind generated waves affecting the rock chute on the original Tekapo B Power Station temporary tailrace (if the chute structure is exposed due to contingent storage drawdown of Lake Pūkaki) is from the northwest. This direction is marked by the red arrow in Figure 5.3. The waves generated by waves from this direction are likely to be higher than from a northerly direction as the 1% annual exceedance probability (AEP) wind speed frequency of ≈ 34 m/s is higher than the corresponding frequency for northerly winds (≈ 28 m/s) (Damwatch, 2018).

The straight-line fetch length for north-westerly winds at the location of the original Tekapo B Power Station temporary tailrace is approximately 9 km.

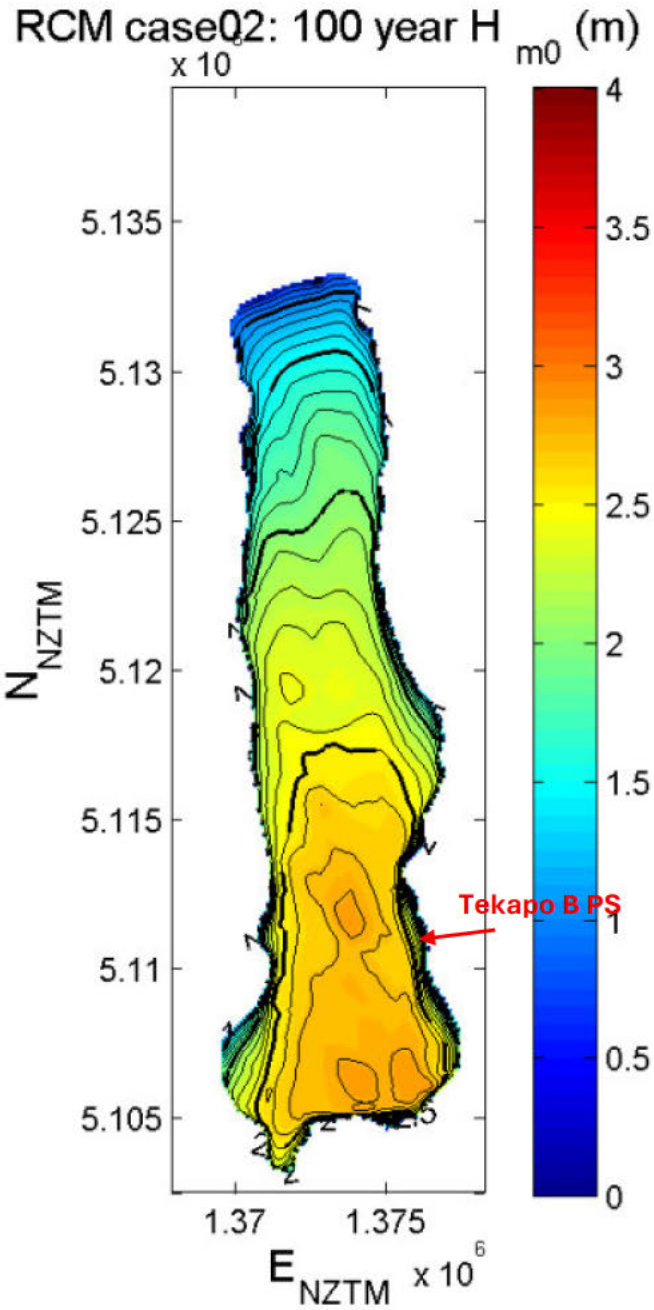
Source: https://www.topomap.co.nz/NZTopoMap

Figure 5.3: Extract of 1:50,000 scale topographic map of Lake Pūkaki and Tekapo B Power Station (labelled 'Pūkaki Power Station' along eastern lakeshore in top right of image)

5.4.2 Estimates of Significant Wave Heights at Location of Tekapo B Power Station Temporary Tailrace from NIWA (2013)

NIWA (2013) developed a SWAN spectral wave model for Lake Pūkaki and applied the measured Maryburm and Maryburn Fill climate station wind record as well as a synthetic wind record generated by a regional climate model (RCM) to evaluate the wave climate for the lake. At the time, only a 7-year record was available for the Maryburn at Maryburn Fill climate station which is why the 30-year synthetic wind record was also utilised. The wave climate for the lake was evaluated for two lake levels – RL 532.5 m and RL 513 m. The key model outputs were contour plots of significant wave heights for selected annual exceedance (AEP) values. The predicted wave climate for a lake level at RL 513 m is more relevant for this assessment.

Figure 5.4 shows a typical contour map of estimated 1% AEP significant wave heights[3] in Lake Pūkaki for a lake level of RL 513 m based on the RCM 30-year synthetic wind record. Similar contour maps of significant wave height were produced for other more frequent AEP values.



Source: (NIWA (2013)

Figure 5.4: Estimated values of significant wave height with an AEP of 1% for Lake Pūkaki at a level of RL 513 m from an RCM 30-year synthetic wind record applied to the lake

[3] The significant wave height H_s is defined as the average height of the highest one-third of all waves in a wave record (USACE, 2008). It is the most common parameter for defining a 'sea state' and can readily be related to various other wave height estimates assuming that a random wave field fits a Rayleigh probabilistic distribution.

The red arrow in Figure 5.4 marks the approximate location of the Tekapo B Power Station along the eastern shoreline of Lake Tekapo.

Table 5.2 summarises the significant wave height estimates for various frequencies at the approximate location of the Tekapo B Power Station inferred from the different contour maps presented in NIWA (2013). Note that these are fairly coarse estimates of significant wave height as the wave climate model was based on a 100 m lake grid resolution and the clarity of the wave height contours in the vicinity if the Tekapo B Power Station is quite poor. The estimates represent deep water estimates of wave height which do not account for the effects of wave shoaling and breaking caused by the shallow depths of water along the shoreline. An assessment of the effects of the shoreline bathymetry in this context is provided in Section 5.4.3.

Table 5.2: Summary of significant wave height estimates at location of Tekapo B Power Station

AEP (%)	Significant Wave Height (m)	
	based on 7-year Maryburn at Maryburn Fill wind record	based on RCM 30-year synthetic wind record
1		2.3
2	2.1	2.2
5	2.0	2.1

The significant wave height estimates given by the two wave records are reasonably similar which provides confidence in the values obtained from the longer RCM 30-year synthetic wind record.

However, the significant wave height estimates represent the effects of a 1% AEP windstorm affecting Lake Pūkaki. In the context of a relatively short exposure period for the Tekapo B Power Station temporary tailrace due to contingent storage drawdown in Lake Pūkaki, these 1% AEP significant wave height estimates are extremely conservative. Although the NIWA (2013) report also gives 5% AEP significant wave height estimates (see Table 5.2), these estimates are also very conservative. For this reason, alternative estimates of significant wave height for a mean annual windstorm event (42.9% AEP) were obtained using the hindcasting approach outlined in ICE (1996).

5.4.3 Estimates of Significant Wave Heights at Location of Tekapo B Power Station Temporary Tailrace Using ICE (1996) Approach

Table 5.3 summarises the results of a frequency analysis of the February 2006 to April 2017 Maryburn at Maryburn Fill wind record for north-westerly winds from Damwatch (2018). The frequency analysis was carried our using three frequency distributions: Gumbel, GEV and Log Pearson 3. The frequency analysis results for the GEV and Log Pearson 3 distributions are very similar and were used to hindcast significant wave height estimates using the ICE (1996) approach.

Table 5.3: Frequency analysis results for north-westerly winds from Maryburn and Marburn Fill wind record (February 2006 to April 2017) (Damwatch, 2018)

Annual Exceedance Probability AEP (%)	Wind Speed (m/s)		
	Gumbel	GEV	Log Pearson 3
1	36.72	33.86	34.16
2	34.92	33.06	33.18
5	32.52	31.76	31.73
10	30.67	30.55	30.48
20	28.74	29.05	28.99
42.9	26.36	26.82	26.82

Table 5.4 summarises significant wave heights for wind speeds ranging from 10-34 m/s assuming a straight-line fetch length for north-westerly winds of 9 km.

Table 5.4: Summary of significant wave height estimates for north-westerly winds with straight-line fetch length of 9 km using ICE (1996) approach

Wind Speed (m/s)	10	15	20	25	26.8	30	34
AEP (%)					42.9		1
Significant Wave Height (m)	0.54	0.81	1.08	1.35	1.44	1.62	1.83

The significant wave height estimates in Table 5.4 are slightly lower than those derived from the SWAN model in Table 5.2. However, they are sufficiently similar to enable them to be used for analysis of the stability of the Tekapo B Power Station temporary tailrace under wave loading conditions.

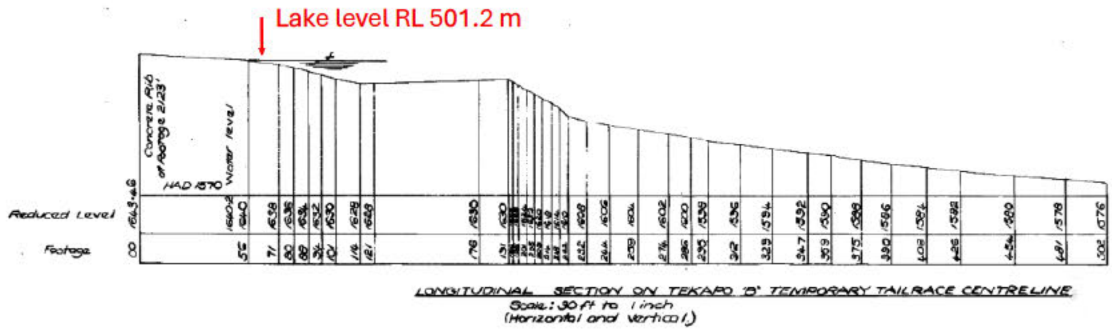
For the purposes of the latter analysis, we have assumed a nominal significant wave height of H_s = 1.5 m based on a mean annual wind speed estimate of 26.8 m/s. For a wave height of this magnitude, we would expect the wave period to be in the range of $3 < T < 5$ seconds.

5.4.4 Effects of Offshore Bathymetry on Wave Heights Approaching Tekapo B Power Station Temporary Tailrace Structure

MOW drawing PD 02/02/9 (refer to Appendix A) shows a contour plan and long-section of the near-shore bathymetry of Lake Pūkaki beyond the end the rock chute structure for the Tekapo B Power Station temporary tailrace. Figure 5.5 shows the long-section along the axis of the chute centreline sourced from this drawing.

The lake level at the time of the bathymetric survey is marked on the image in Figure 5.5. This level is nearly 13 m below the minimum level of RL 514 m considered for contingent storage drawdown in this assessment. Therefore, the water depths at this minimum drawdown level of RL 514 m are large enough to preclude the possibility of wave breaking occurring before the lower end of the chute structure and possibly large enough for waves to impact at the lower end of the 1 in 19.3 sloping ramp-type flow section of the structure without breaking.

Consequently, it is possible for 2 m high waves to impact on the rock chute structure in the bays between sills 1-2 and 2-3 (RL 517 m minimum drawdown level), and between sills 3-4 and 4-5 (RL 514 m minimum drawdown level). However, the duration of exposure would be limited so that the actual annual probability of this occurring would be very low.



Source: MOW drawing PD 02/02/9

Figure 5.5: Longitudinal section of shoreline bathymetry beyond end of Tekapo B Power Station temporary tailrace (profile aligned with centreline of structure)

5.4.5 Stability of Rock Lining on Chute under Wave Loading Conditions

As noted in Section 5.2, the boulder sizes in the bays between sills 1-2, 2-3 and 3-4 are fairly large (d_{50} > 0.50 m generally – refer to Table 5.1) which should be sufficient to sustain impact by waves with a significant wave height of up to H_s = 1.5 m. However, the boulder sizes in the bay between sills 4-5 and sills 5-6 are much smaller (d_{50} = 0.30 m and 0.27 m respectively) and may be prone to instability under wave impact if this part of the structure was exposed due to Lake Pūkaki being drawn down towards RL 513 m.

Although there is no evidence of the rock chute on the temporary tailrace being damaged by wind generated waves prior to the level of Lake Pūkaki being raised above RL 518 m in 1979, the potential for damage under wave impact loading if the lake level is drawn down towards RL 514 m is examined further in this section.

Waves with an offshore significant wave height of 1.5 m and a period of 3-5 seconds are predicted to be plunging type breaking waves when they impact on the temporary tailrace chute (USACE, 2003a) (see Figure 5.7).

At a minimum level of RL 514 m in Lake Pūkaki, the depth of water in the bay between sills 4 and 5 is in the range of 0.6-2.3 m. The water depth in the lower end of the bay is therefore likely to be deep enough for waves with a significant wave height of 1.5 m approaching obliquely to reach the temporary tailrace chute without breaking beforehand. They will therefore break within the bay between sills 4 and 5 at a lake level of RL 514 m and impinge on the rock lining material.

At a minimum level of RL 513 m in Lake Pūkaki, the depth of water in the bay between sills 4 and 5 is less than the breaking depth. Waves with an offshore significant wave height of 1.5 m and a period of

3-5 seconds will therefore break within the bay between sills 5 and 6 at a lake level of RL 513 m and impinge on the rock lining material.

Source: USACE (2003)

Figure 5.7: Example of plunging type breaking wave which could potentially impact on Tekapo B Power Station temporary tailrace chute in a north-westerly windstorm event affecting Lake Pūkaki

The stability under plunging wave attack of the rock material in the bay between sills 4 and 5 and sills 5 and 6 was assessed using two equations developed for evaluating the stability of rubble mound breakwaters (USACE, 2003b):

- the Hudson equation (Table VI-5-22 of ASCE (2023b)); and
- the van der Meer equation (Table VI-5-23 of ASCE (2023b)).

The Hudson equation relates the stability of rock material forming an armour layer on a rubble mound breakwater with a median size d_{50} to the incident wave height, the submerged density of the rock boulders, the slope of the structure and a stability coefficient determined from small-scale model tests in a laboratory.

The van der Meer equation relates the stability of rock material forming an armour layer on a rubble mound breakwater with a median size d_{50} to the incident wave height, the wave steepness, the submerged density of the rock boulders, the damage level on the structure (usually specified as 2-3%), the number of waves to reach an equilibrium damage level (usually less than or equal to 7,500). the permeability of the structure and the slope of the structure.

Both equations are intended for conditions where the incident waves are irregular and approach the structure head-on.

Application of the Hudson equation assuming a stability coefficient of K_d = 3.5 (USACE, 2003b) for rough angular rock on a 3:1 (horizontal: vertical) slope with a surface area damage level of 0-5% (i.e.

relative eroded area) and a submerged density of 1.65 indicates that the median rock size for structural stability for an incident significant wave height of H_s = 1.5 m is d_{50} = 0.42 m. As the actual slope of the Tekapo B Power Station temporary tailrace chute was much flatter than this, the stability of the rock riprap material in the bay between sills 4 and 5 and between sills 5 and 6 was examined further with the van der Meer equation.

Application of the van der Meer equation assuming the same incident wave conditions and submerged rock density, a notional permeability for the structure of 0.4 (based on a double layer thickness of riprap material on a filter layer and a bedding layer and a wave count of 7,500 waves to reach an equilibrium damage level (USACE, 2003b) indicates that the median rock size for structural stability is:

- d_{50} = 0.20 m for a wave period of 3 seconds and a structure slope of 10:1 (horizontal: vertical)
- d_{50} = 0.23 m for a wave period of 4 seconds and a structure slope of 10:1 (horizontal: vertical)
- d_{50} = 0.26 m for a wave period of 5 seconds and a structure slope of 10:1 (horizontal: vertical)

As the actual slope of the Tekapo B Power Station temporary tailrace chute is nearly twice as flat as the assumed minimum structure of 10:1 (horizontal: vertical) for application of the van der Meer equation, the rock riprap material lining the bay between sills 4 and 5 (median size d_{50} = 0.30 m, d_{15} = 0.12 m and d_{85} = 0.48 m from Table 5.1) is likely to remain stable under the impact of plunging breaking waves of significant height $H_s \leq$ 1.5 m if the level of Lake Pūkaki is drawn down to RL 514 m. The same conclusion applies to the rock riprap material lining the bay between sills 5 and 6 (median size d_{50} = 0.27 m, d_{15} < 0.10 m and d_{85} = 0.57 m from Table 5.1) if the level of Lake Pūkaki is drawn down to RL 513 m.

This conclusion about the stability of the rock riprap material lining the bay between sills 4 and 5 and the bay between sills 5 and 6 would remain valid if the impinging waves approach the tailrace chute structure obliquely rather than head-on.

6 Conclusions

This report presents the findings of a review of the original temporary tailrace weir and rock chute for Tekapo B Power Station. The weir and chute structure could potentially be exposed in the future if Lake Pūkaki is drawn down to allow use of contingent storage in the event of a shortfall in the national electricity supply.

Key conclusions from the review are as follows:

- Although the original control weir and rock chute structure was designed and constructed as an inexpensive temporary facility and perceived to have a high risk of failure, the structure performed satisfactorily over its lifetime with only minor damage being sustained despite the rock lining in some parts being close to the threshold of stability.
- The rock lining material on the chute is much larger in the first three bays between sills 1-2, 2-3 and 3-4 than the original construction specification required.
- The weir at the head of the rock chute controls water levels in the upstream outlet channel from the Tekapo B Power Station (now the permanent tailrace channel). Flow conditions will remain sub-critical in the outlet channel even when the level of Lake Pūkaki is drawn down.
- Irrespective of the lake level when Lake Pukaki is drawn down below RL 518 m, tailwater levels at Tekapo B Power Station will be unchanged due to them being controlled by the crest level of the weir at the head of the rock chute (tailwater levels at Tekapo B Power Station under these conditions are a function of the power station discharge).
- When the level of Lake Pūkaki is drawn down below RL 520 m, highly aerated ramp-type flow develops at the upper end of the rock chute. Flow and depth measurements on the original structure demonstrated that the flow conditions were sub-critical although with a Froude number approaching 1 (the transition criterion between the sub-critical and super-critical flow regimes).
- There is a sharp transition between the ramp-type flow on the chute and the backwater profile corresponding to the horizontal lake surface.
- Under ramp-type flow conditions down the rock chute, the rock lining in the bays between sills 1-2, 2-3 and 3-4 should remain stable if the level of Lake Pūkaki is drawn down to RL 513 m and power station flows up to 105 m^3/s are discharged.
- Deposition of fine sediment over the rock chute structure since use of the temporary tailrace ceased in 1979 could initially cause the flow resistance to be slightly lower when ramp-type flow conditions develop with Lake Pūkaki being drawn down below RL 520 m. However, the highly turbulent nature of the flow would rapidly erode any deposited sediment with the flow conditions reverting to a sub-critical regime.
- The rock chute could be exposed to north-westerly wind-generated waves impacting on the structure without breaking before reaching the structure. The impacting waves are predicted to be plunging type breaking waves.
- The boulder sizes in the bays between sills 1-2, 2-3 and 3-4 are likely to be large enough to resist such wave attack.
- Although of much smaller size, the boulders lining the bay between sills 4-5 and 5-6 are also likely to remain stable under the impact of plunging type breaking waves of significant height $H_s \leq 1.5$ m (representative of a mean annual wind speed condition) if the level of Lake Pūkaki is drawn down to RL 513 m.

- There is no evidence of the rock chute on the temporary tailrace being damaged by wind generated waves when the level of Lake Pūkaki was below RL 518 m before 1979.

7 References

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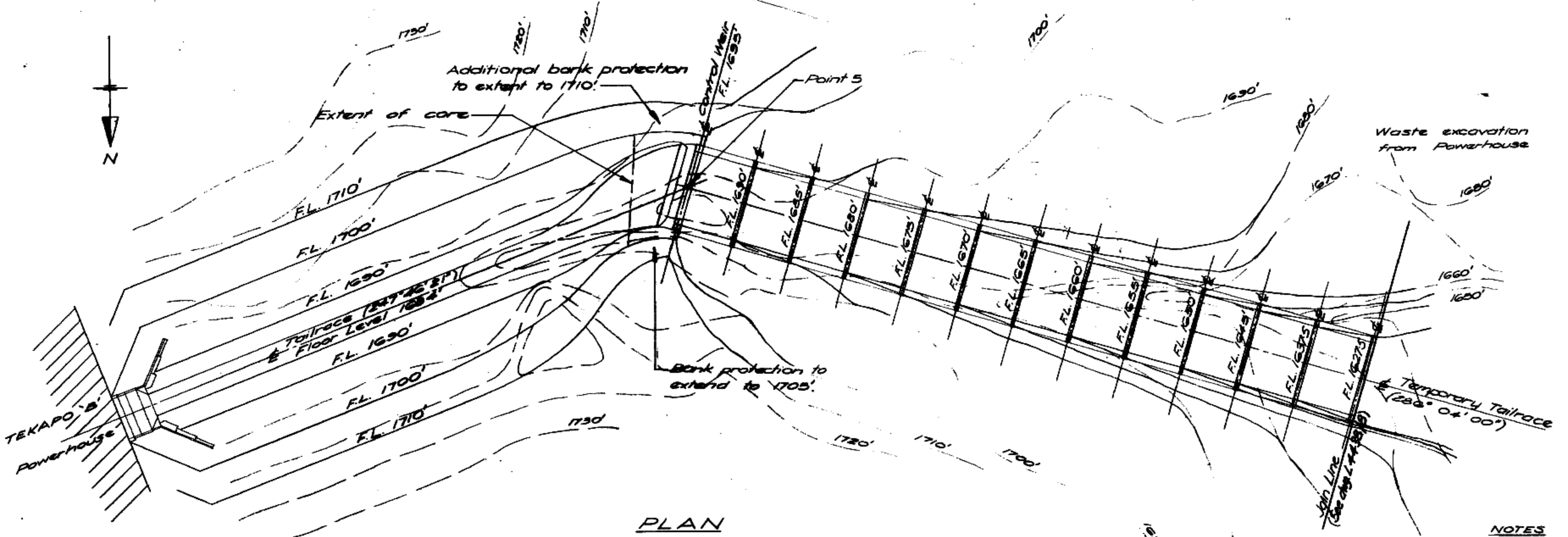
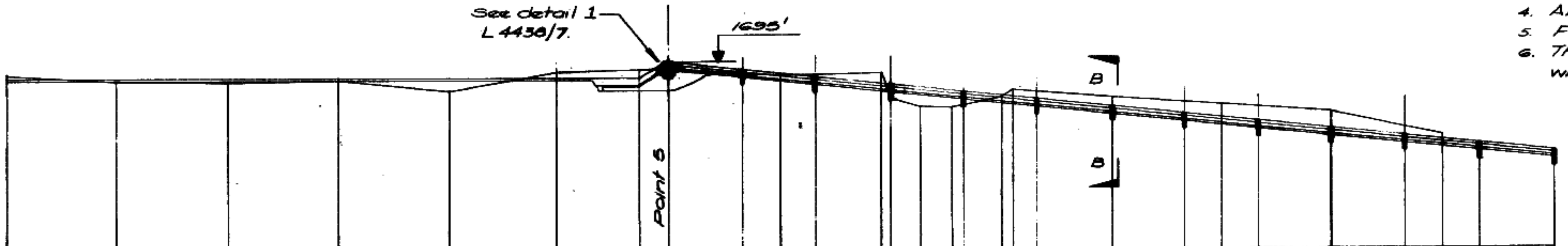
Appendix A Reference Drawings



NOTES

1. As the design office has no positive information on the Haul Road it has been omitted from these drawings. It will be left up to site to move all material necessary to attain formation levels shown on this drawing.
2. Existing contours shown dotted.
3. F.L. = Formation Level.
4. All levels are to Waitaki Datum.
5. Formation Levels have a tolerance of ±2".
6. This drawing is to be read in conjunction with drawing L4438/7, & L4438/8.



H.A.D. 1667'

FORMATION LEVEL (See note 5) (FT)	1684						1684	1695	1690		1685		1680			1675		1670		1665	1660		1655	1650	1645		1637.5	1627.5
EXISTING GROUND LEVEL (FT)	1687.00	1683.50	1682.50	1683.00	1676.60	1688.00		1690.30		1687.60		1688.70	1671.30	1666.70	1665.93		1673.10	1677.03		1672.60		1667.30		1663.30		1646.50		
DISTANCE (FT)	225	375	523	673	823	973	1090.49	1123.49	1223	1273	1323	1412	1423	1468	1505	1523	1573	1590	1623	1723	1823	1873	1923	2023	2123	2173	2223	2323

LONGITUDINAL SECTION

Scale: Horiz. 100' to 1 inch
Vert. 50' to 1 inch

REFERENCE DRAWINGS

T 38/01/2 Plan & Long Section, Tailrace
T 38/01/3, 4 & 5 Cross Sections

ORIGINAL DESIGN DWG. as built details on T.38/01/6, 7 & 8.

This drawing complies with the requirements of N.Z. Electricity Department. Snr. Design Engineer Date 17-11-76 N.Z. Elect. Dept.

100'-0" to 1 inch 100 0 100 200' 300' 400' 500'
GRAPHIC SCALES:

X As Built			8/87
AMENDMENTS	BY	APPD	DATE

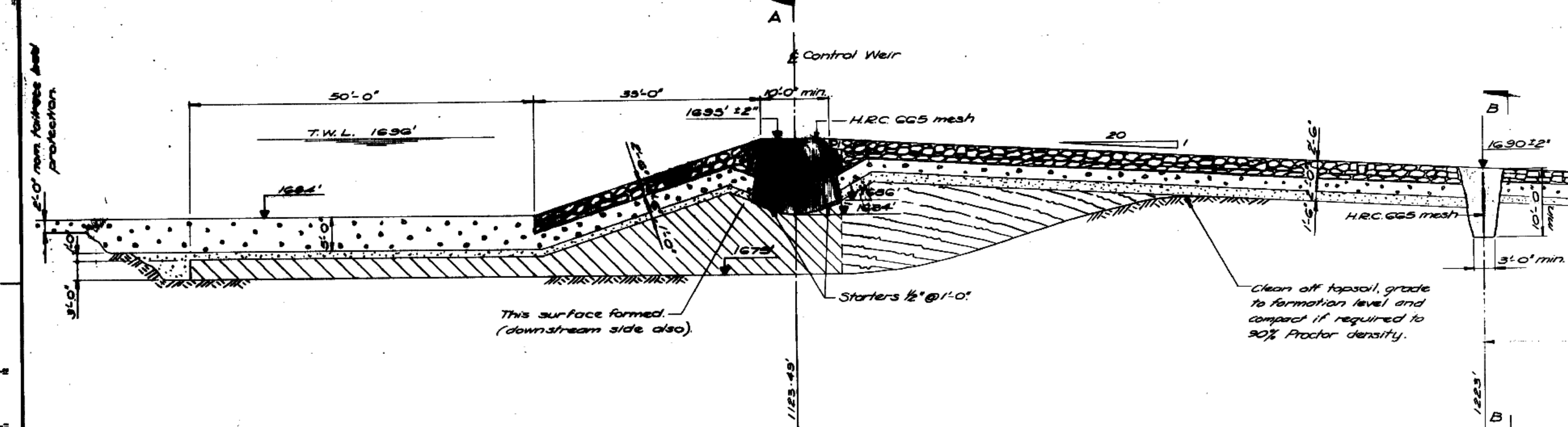
	BY	CHECKED BY	DATE
Drawn			10/11/76
Computed			11/76

RECOMMENDED:
APPROVED
Ministry of Works and Development
POWER
J. J. CHESTERMAN CHIEF ENGINEER
N.C. McLEOD Commissioner

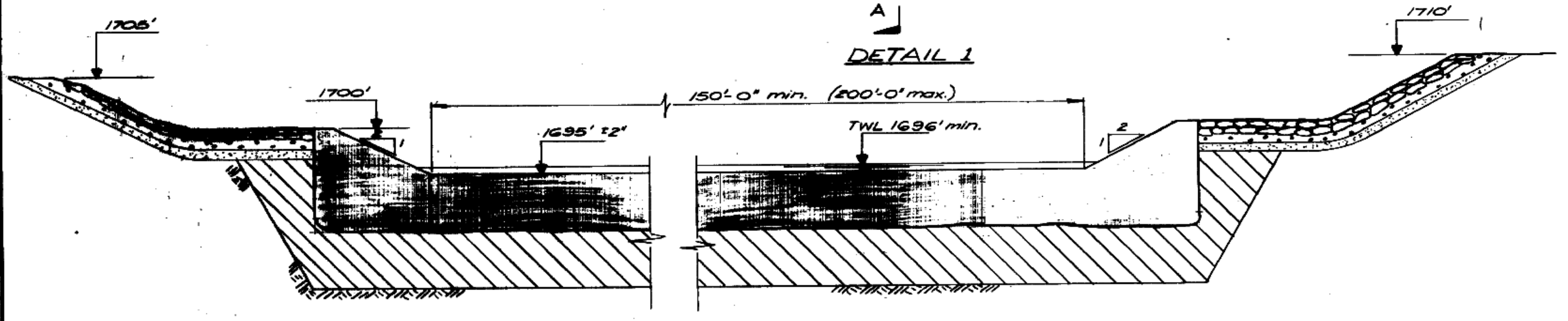
TEKAPO 'B' POWER STATION
TEMPORARY TAILRACE
PLAN AND LONGITUDINAL SECTION

ORIGINAL SCALES 100'-0" to 1 inch or as shown.
FILE: 98/2/77/42 FOLDER: P77/3j SPEC: H.D.1147
ISSUE XX
SHEET No.
POWER DESIGN
L 4438/6
IN SHEETS

MICROFILMED—NO Issue X

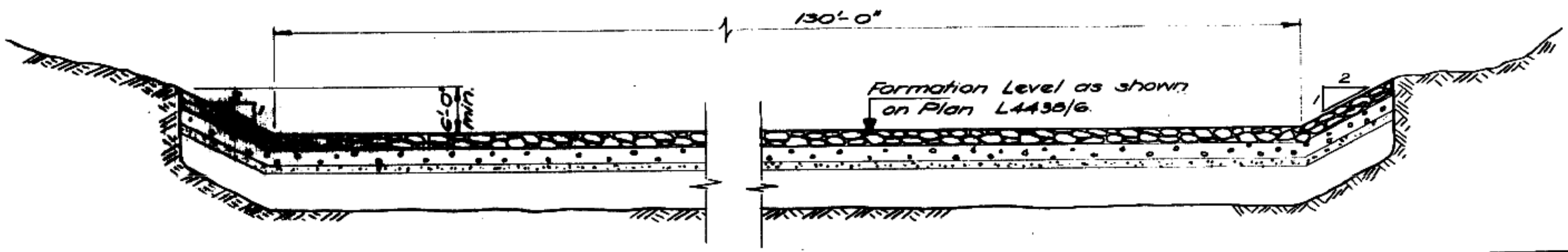


DETAIL 1



A - A

Scale: 10'-0" to an inch.



B-B

TYPICAL SECTION

Scale: 10'-0" to an inch.

LEGEND

- Riprap
- Gravels
- Sand Filter
- Impermeable Core
- Compacted natural material
- Concrete

T38/261

Notes:

2. Min lap for mesh 1'-0".
3. Reinforcement to pass through all construction joints.
4. Stockpile 2,000 cub.yds riprap and 1,000 cub.yds. gravel for repair and emergency work.
5. Dump random rocks on downstream side of final weir at distance 2323'.

ORIGINAL DESIGN DWG.
as built details on
T38/01/6,7 & 8

8'-0" to an inch
GRAPHIC SCALES: 10' 0' 10' 20' 30' 40'

	BY	CHECKED BY	DATE
Drawn			12/11/76
Computed			11/76

RECOMMENDED: 17/11/76

APPROVED 18/11/76

X As Built — 8/81

AMENDMENTS | BY | APP'D | DATE

Ministry of Works and Development

POWER

J. L. CHESTERMAN-CHIEF ENGINEER

N.C. McLEOD Commissioner

TEKAPO 'B' POWER STATION

TEMPORARY TAILRACE DETAILS

ORIGINAL SCALES: 8'-0" to an inch or as shown

FILE: 92/12/77/48 | FOLDER: P77/3j | SPEC: H.D. 1147

POWER DESIGN

L 4438/7

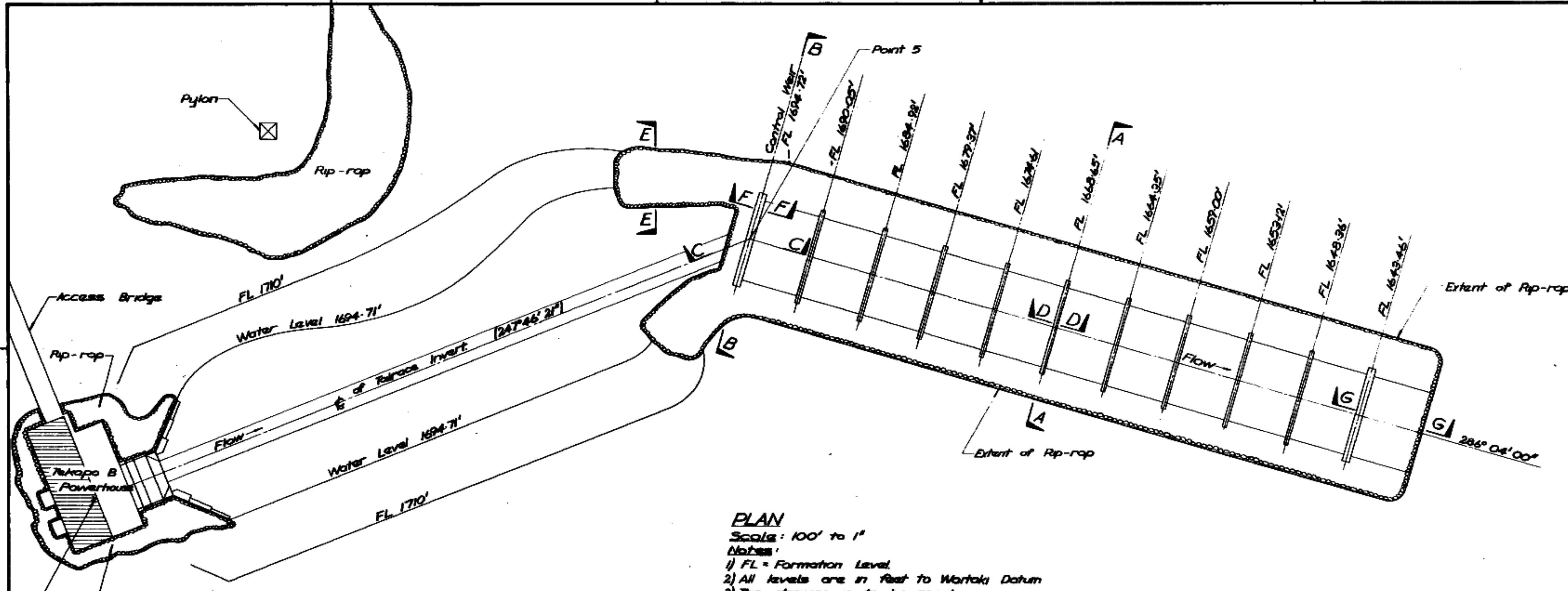
ISSUE: XX

SHEET No.

IN SHEETS

482-A1 July 1972

ORIGINAL SIZE mm INCHES

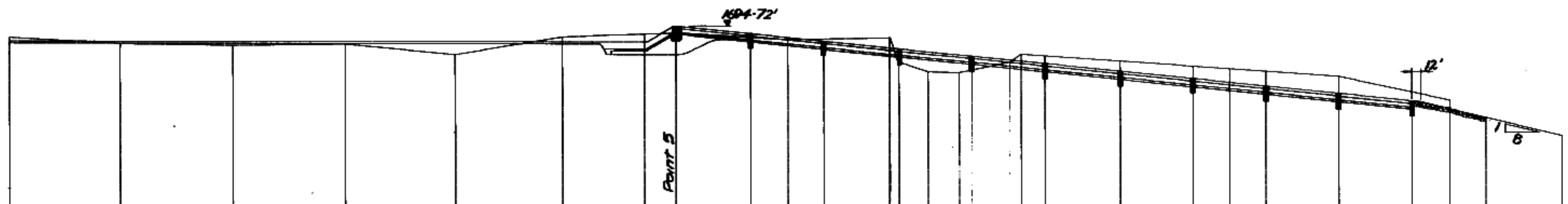


PLAN

Scale: 100' to 1"

Notes:

1) FL = Formation Level.
2) All levels are in feet to Waitaki Datum
3) This drawing is to be read in conjunction with T38/01/7 & 8 and PD02/02/9
4) This dwg. supersedes L4438/6
5) Co-ordinates of Point 5:
   99765' N
   223000' W
   in feet Mt. Horrible.



Distance (0.00' is at Pt 4)	Formation Level	Pre-construction Ground Level
223'	1684'	1687.00'
373'		1683.50'
523'		1682.50'
673'		1683.00'
823'		1676.60'
973'		1688.00'
1090.49'	1684'	
1123.49'	1694.72'	1690.30'
1222'	1690.05'	
1273'		1687.60'
1323'	1684.92'	
1412'		1688.70'
1423'	1679.37'	1671.90'
1468'		1666.70'
1505'		1665.93'
1523'	1674.61'	
1573'		1673.10'
1590' 1625'	1668.65'	1677.03'
1723'	1664.35'	1672.60'
1823'	1659.00'	
1873'		1667.30'
1923'	1653.12'	
2023'	1648.96'	1663.90'
2123'	1643.46'	
2173'		1645.50'
2223'		
2323'		

LONGITUDINAL SECTION

Scale: Horizontal = 100' to 1"
Vertical = 50' to 1"

New Zealand Electricity
FOLDER: T25/3F1

CERTIFIED
AS BUILT
DRAWING
ENGINEER 26/1/77 DATE
REVISED
SEE ALSO T38/01/7-8

	BY	APP'D	DATE
☒ AS BUILT - Supersedes L4438/6	CGB		
AMENDMENTS	BY	APP'D	DATE

	BY	CHECKED BY	DATE
DRAWN	C. Baxter		10-77
SUP'D			10-77
Recommended:			22/11/77

SMJ Smith
Project Engineer

Approved 23-11-77
Resident Engineer

Ministry of Works New Zealand

POWER
J J Chesterman
Chief Engineer

N C McLeod Commissioner

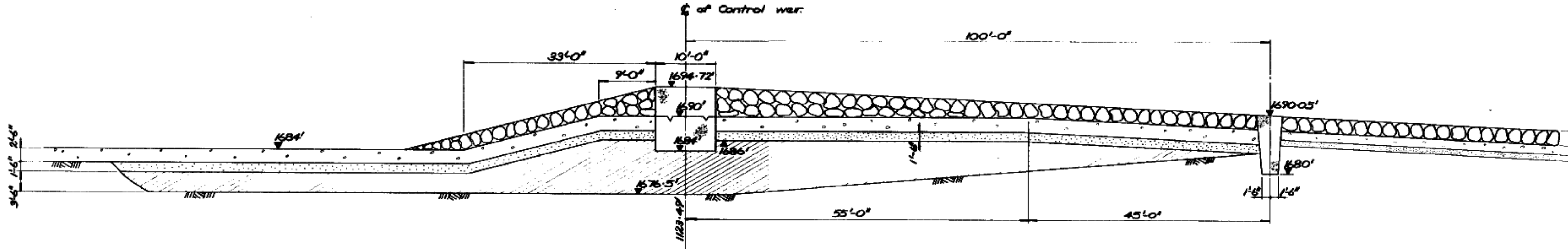
TEKAPO B POWER PROJECT

TEMPORARY TAILRACE
PLAN AND LONGITUDINAL SECTION

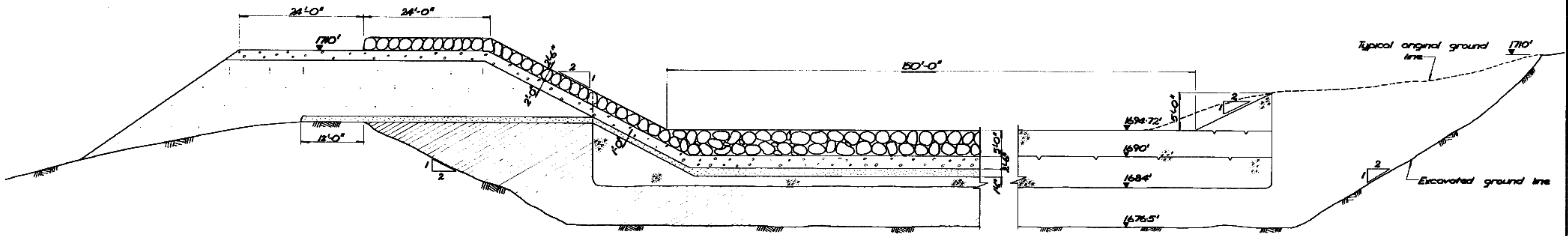
ORIGINAL SCALES AS SHOWN
FILE:
TWIZEL
T38/01/6

ISSUE X
SHEET No.
IN SHEETS

a b c d e f



SECTION C-C



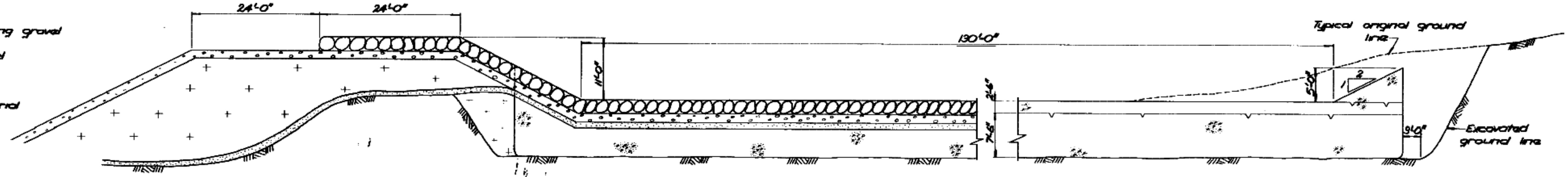
HALF SECTION SHOWING EARTHWORKS

HALF SECTION SHOWING CONSTRUCTION JOINTS

SECTION B-B

Legend

- Rip-rap
- Free draining gravel
- Silty gravel
- Sand
- Core material
- Concrete



HALF SECTION SHOWING EARTHWORKS

HALF SECTION SHOWING CONSTRUCTION JOINTS

SECTION A-A

Note:
1) This dwg. to be read in conjunction with T38/01/6 & 8
2) This dwg supersedes L4438/7

CERTIFIED **AS BUILT** DRAWING
ENGINEER 23/11/77 DATE
NO CHANGE
REVISED
SEE ALSO T.38/01/6 & 8

10' 5' 0 10' 20' 30' 40' 50' 10' to 1"

New Zealand Electricity
FOLDER: T25/3F1

	BY	CHECKED	DATE
DRAWN	C Baxter		10·77
SUP'D			10·77

S.M.J Smith Project Engineer

APPROVED 23-11-77 Resident Engineer

Recommended 22/11/77

Ministry of Works and Development
POWER
J J Chesterman Chief Engineer
N.C. McLEOD Commissioner

TEKAPO B POWER PROJECT

TEMPORARY TAILRACE DETAILS - SHEET 1

ORIGINAL SCALES 10' to 1"

FILE

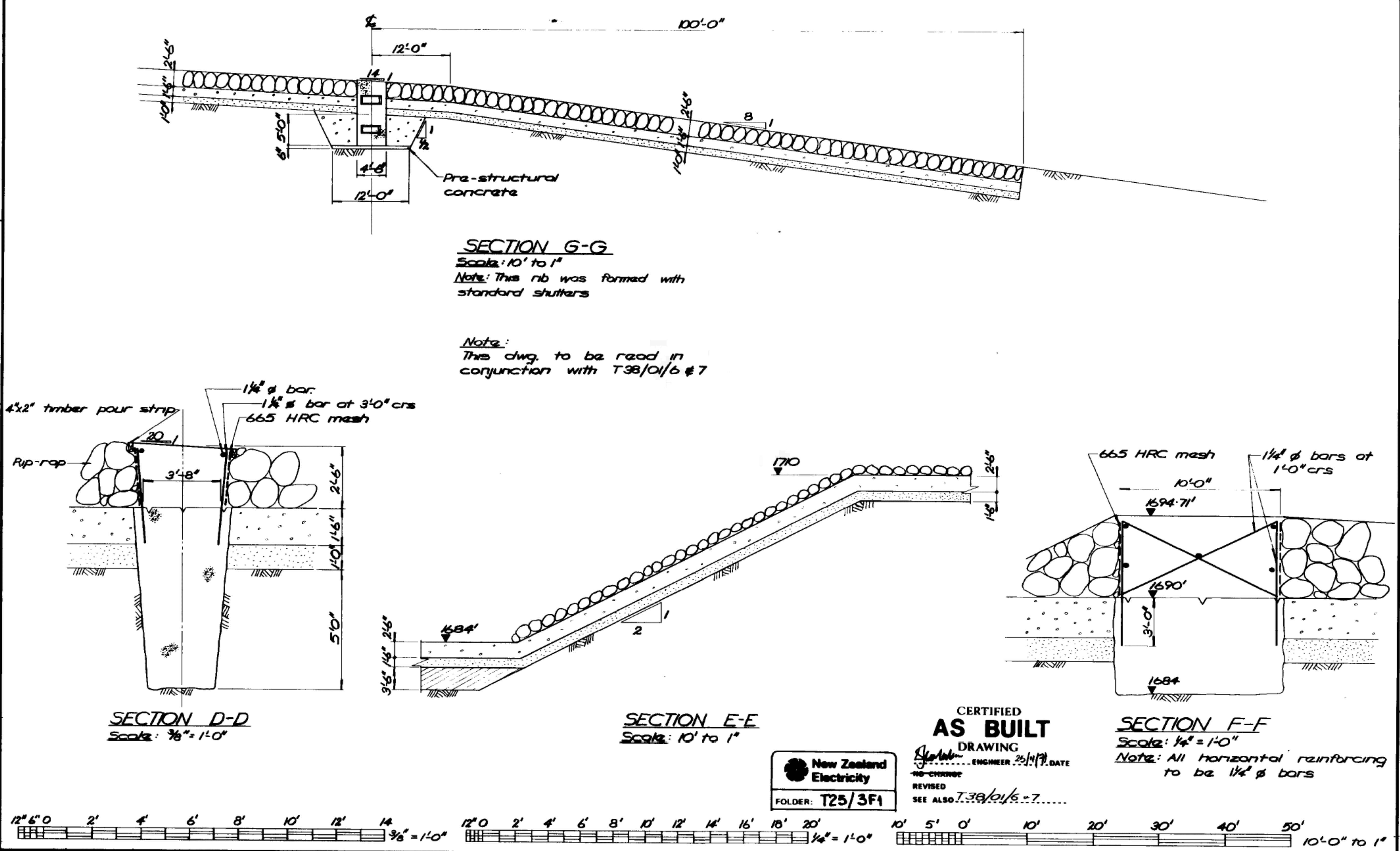
TWIZEL

JOB	CODE	SHEET	REVISION
T38/01/7			X

AMENDMENTS	BY	APPD.	DATE
X AS BUILT - Supersedes L4438/7	CGB		

LOCAL OR TRACING No.

1:1 1:2 1:5



Supersedes L4438/7			
X / AS BUILT	CGB		
AMENDMENTS	BY	APP'D	DATE

	BY	CHECKED	DATE
DRAWN	C. Baxter		10-77
SUPD			10-77

Recommended: 22/11/77

SMJ Smith
Project Engineer

APPROVED
23-11-77
Resident Eng

Ministry of Works and Development
POWER
J J Chesterman
Chief Engineer
N.C. McLeod Commissioner

TEKAPO B POWER PROJECT

TEMPORARY TAILRACE
DETAILS - SHEET 2

ORIGINAL SCALES: AS SHOWN

FILE

TWIZEL

T38/01/8

SHEET,

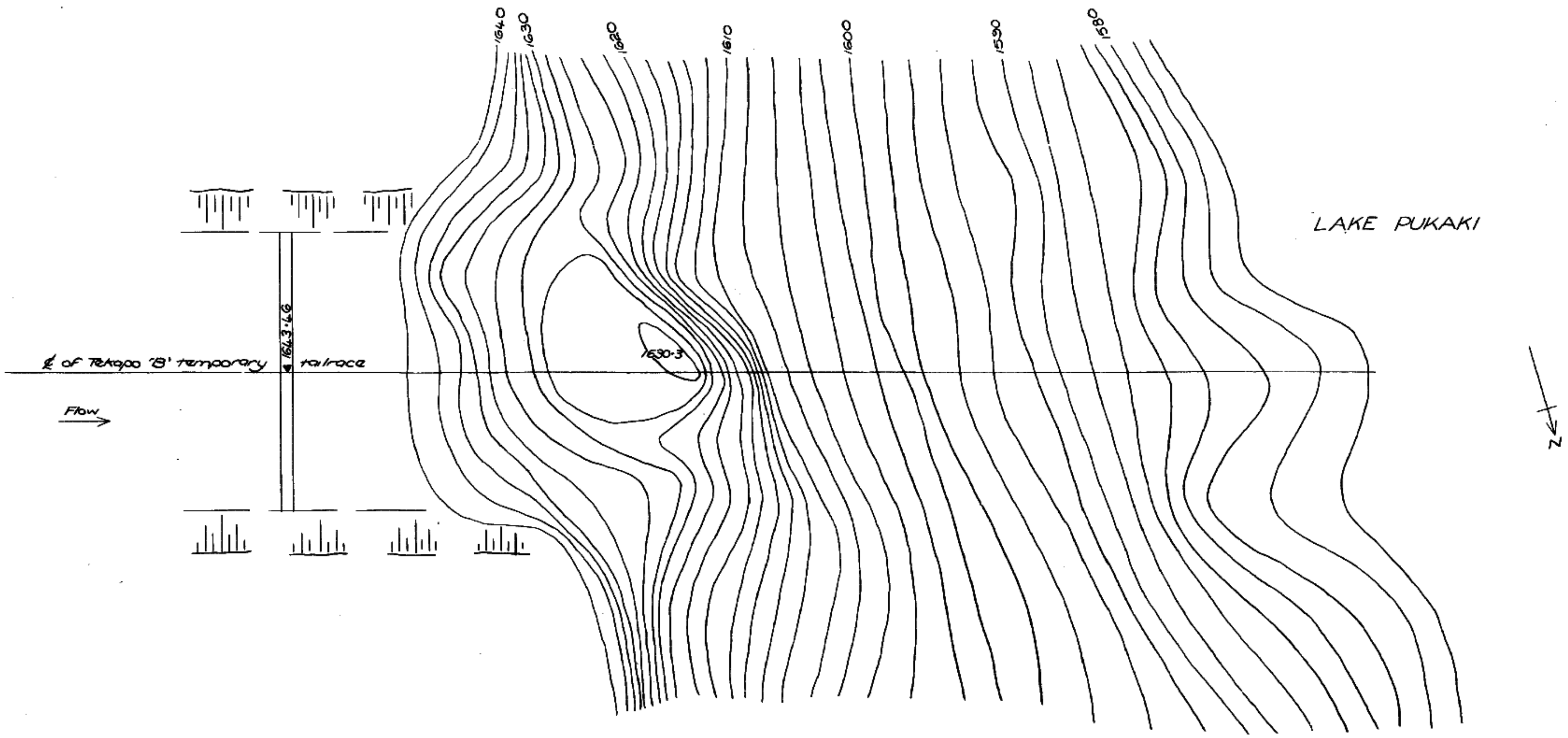
REVISION X

A2 LOCAL OR TRACING No.

976

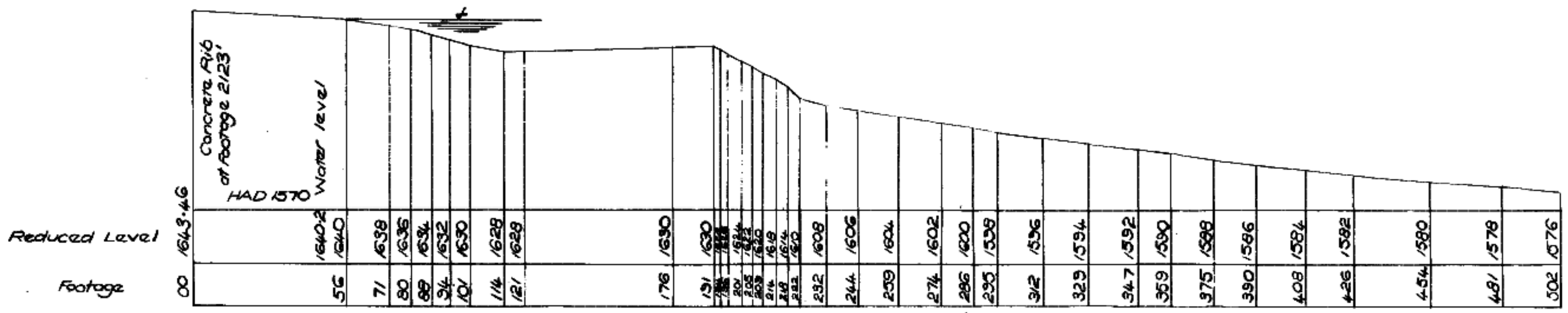
1:1 1:2 1:5 1:25

Issue X



Surveyed - 22nd September 1977
Lake level - 1640·2 Feet Waitaki Datum

Reference drawing
T38/01/6 — Tekapo 'B' Temporary Tailrace (NZE Folder T25/3F1)



LONGITUDINAL SECTION ON TEKAPO 'B' TEMPORARY TAILRACE CENTRELINE
Scale: 30 ft to 1 inch
(Horizontal and Vertical.)

30' 20' 10' 0 30' 60' 90' 120' 150'
30 Ft to 1 inch

CERTIFIED
AS BUILT
DRAWING
ENGINEER DATE
NO CHANGE
SEE ALSO T38/01/6

AMENDMENTS	BY	APP'D	DATE
X/ As built	NIC		

	BY	CHECKED BY	DATE
Drawn	C.G.Brown		7/10/77
Sup'v'd			10.77
Recommended:			12/10/77

APPROVED 12-10-77
Resident Engineer

S.M.J. Smith
Project Engineer

Ministry of Works New Zealand
POWER
J. J. Chesterman
Chief Engineer
N.C. McLeod Commissioner

LAKE PUKAKI DAM

LAKE PUKAKI BATHYMETRIC SURVEY
AT TEKAPO 'B' TEMPORARY TAILRACE ENTRY

ORIGINAL SCALES: 30 ft to 1 inch
FILE:
TWIZEL
PD 02/02/9
ISSUE: X
SHEET No.
IN SHEETS