Technical Advice – Water Engineering - by Mark Groves

Date	30.03.2026
To	Trinity White, Environmental Manager – South Island Renewables, Genesis Energy
From	WSP New Zealand Limited
Project advice provided for	Fast-track Application for Lake Pukaki Hydro Storage and Dam Resilience Works
Documents referred to	WSP, 2025 *“Tekapo B Weir Assessment Summary Memo”* WSP, 2026 *“Tekapo B Weir Assessment Memo Addendum”*
Qualifications and experience	ONC, HNC Civil Engineering. Chartered Engineer (CPEngNZ). 28 years of experience.
Code of Conduct	As an expert witness I have read, and I am familiar with, the Code of Conduct for expert witnesses contained in the Environment Court Practice Note 2023. This memorandum has been prepared in compliance with that Code. In particular, unless I state otherwise, this response is within my area of expertise and I have not omitted to consider material facts known to me that might alter or detract from the opinions I express.
Signature	

Memo

To: Andrew Balme

From: Jeremy Robertson

Office: Taupō

Date: 19 December 2025

File/Ref: 2-38161.00

Subject: Tekapo B Weir Assessment - Summary of Findings

REV	DATE	DETAILS
1	27/11/2025	FINAL
2	19/12/2025	FINAL Rev 2

	NAME	DATE	SIGNATURE
Prepared by:	Jeremy Robertson	18/12/2025	JR
Reviewed by:	Jeanette Tucker	18/12/2025	JT
Approved by:	Simon McSweeney	19/12/2025	SM

1. Background

Genesis Energy engaged WSP to assess the condition of the Tekapo B temporary tailrace, consisting of the submerged weir and outfall chute at the outlet of Tekapo B Power Station. The assessment included bathymetric survey, visual condition inspection, structural condition assessment, and hydraulic assessment. The scope was extended to include review of the Damwatch Engineering Limited (2025), "Hydraulic Review of Weir and Chute", Issue 3.1. The objective for Genesis is to determine if the temporary tailrace could be safely and reliably operated should the level of Lake Pukaki be lowered.

This memo summarises findings relating to the concrete control weir and concrete sill/ribs. Observations on the rock-lined outfall chute are presented where relevant.

Additional detail and recommendations are contained in the following reports, produced as deliverables for the workstream discussed above:

- Tekapo Submerged Weir Bathymetric Survey and Hydraulic Assessment, Hydraulic Assessment, Final version 3, 2025, WSP
- Tekapo Submerged Weir, Structural Condition Assessment, FINAL v2, 2025, WSP

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2. Summary of findings

- The temporary tailrace was designed and constructed as an inexpensive temporary facility and hence is currently beyond its service life. It was stated at the time of construction “that the chute is a high risk structure and minimum use only can be recommended” (Ministry of Works and Development , 1977). Therefore the present-day state of the structure is not expected to perform without a high risk of failure.

- The discharge chute has documented failures, and subsequent repair works from its single year of service, 48 years ago. Previous reporting also noted that were several springs encountered during the construction of the chute.

- The bathymetric survey conducted September 2025 shows that there are 1 m drops in the chute invert, indicating movement of the rock lining since construction. This may compromise the foundation of the chute lining due to increased interstitial flows; the rip rap protection was founded on free draining sand and gravel layers.

- Having been submerged for most of its life, there is significant risk that the foundations are not in “As Built” condition, and cannot perform its original intended purpose, if exposed and returned to service.

- The visual inspection conducted September 2025 shows movement of the chute rip rap protection since original construction, leaving concrete and reinforcement exposed and vulnerable to further degradation.

- WSP agrees with the hydraulic review methodology and modelling approach adopted in the Damwatch report; however, the assessment does not account for current bathymetry, riprap movement, deterioration of structural components, or material degradation, and therefore cannot be relied upon to represent present-day performance.

- Based on the current bathymetric survey and the observed deterioration of the sill structures, it is expected that the discharge chute will no longer perform to its original “As Built” standard. Consequently, the risk of the chute failing during lake drawdown is now higher than at the time of initial construction and operation.

- As the structure was originally intended for temporary use and has significantly exceeded its design life, any intentional drawdown of the lake and reinstatement of the weir would require both submerged and dry repairs to be undertaken. Repairs, whether completed in wet or dry conditions, will necessitate temporary closure of the discharge chute and will impact Tekapo B operations during outage windows. Allowance for outage duration, repair sequencing, and compensatory arrangements should be included in any consent conditions associated with the change in lake operation.

- Whilst repairs to the structures would be expected to return the concrete elements to a state where they could perform to their original intent, and potentially reduce the risk associated with operation, there is inherent risk associated with temporary nature of the original design and construction of the temporary tailrace that repairs would not address.

Genesis Energy

Tekapo Submerged Weir Bathymetric Survey and Hydraulic Assessment

Hydraulic Assessment

2-38161.00

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REV	DATE	DETAILS
1	18/11/2025	For review
2	27/11/2025	Final issue
3	19/12/25	Final version 3

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Prepared by:	Sailish Parbhu Maurel Borja	16/12/2025	SP / MB
Reviewed by:	Jeanette Tucker	16/12/2025	JT
Approved by:	Simon McSweeney	19/12/2025	SM

1 BACKGROUND

1.1 PURPOSE

Genesis Energy Ltd have engaged WSP NZ to undertake a review of previous hydraulic assessments and complete a hydraulic assessment of the temporary tailrace weir and outfall structure at Tekapo B Power Station. The structure consists of a concrete control weir at the upstream end that creates a pond at minimum tailwater level. Downstream of that is a sloped discharge chute, consisting of 10 transverse concrete ribs, embedded into the natural ground level, with rip rap between each rib. The chute is bounded along the sides by rock berms (Figure 1-1 and Appendix A).

The power station initially operated with a temporary tailrace weir and outfall structure following its commissioning in 1977 with minor repairs required to the structure to remediate scour holes prior to the filling of Lake Pūkaki. Since the lake level was reinstated to full operating conditions (above RL 518 m), the weir and chute have remained submerged.

Figure 1-1 Tekapo B powerhouse 1977 (source: Genesis Energy Ltd)

Our hydraulic assessment aims to evaluate whether the weir and outfall structure are suitable for reinstatement under conditions of reduced Lake Pūkaki water levels. The hydraulic assessment was conducted in conjunction with a condition assessment of the existing weir and chute.

1.2 INFORMATION

The key information obtained and used for the purposes of this report are outlined in Table 1-1:

Table 1-1 Source of information referenced

Document Title	Author	Key Information
Tekapo B Power Station Temporary Tailrace Weir and Rock Chute, Hydraulic Review of Weir and Chute, Issue 3.1, Oct 2025	Damwatch Engineering	Hydraulic assessment of the existing weir and chute based on as-built information
Tekapo B Submerged Weir and Tailrace Survey, Oct 2025	WSP	Bathymetric survey of the existing tailrace, weir and chute downstream of Tekapo B Powerhouse
Tekapo B Temporary Tailrace Design Report, June 1979	Ministry of Works and Development	Design report for the temporary tailrace, including details on weir structure and chute (before the concrete sills were designed for)
Tekapo B Temporary Tailrace As-built Drawings, 1979	Ministry of Works	As-built levels of the weir and chute, with details on the concrete sills
Tekapo B Temporary Tailrace Construction Report, 1979	Ministry of Works	Report on construction of the temporary tailrace, including details on the inclusion of concrete sills on the chute and site photos

2 PREVIOUS HYDRAULIC ASSESSMENT

A report was prepared by Damwatch Engineering Limited on behalf of Merdian Energy *"Hydraulic Review of Weir and Chute"* (Damwatch Engineering, 2025) The report presents the findings of a review of the original temporary tailrace weir and rock chute for Tekapo B Power Station, which could be exposed in the future if Lake Pūkaki is drawn down to allow use of contingent storage. The report reviewed the likely hydraulic performance and structural stability of the structure should it be required to operate again under such conditions.

WSP was engaged by Genesis Energy Ltd to review the report. Our key observations from the review are summarised below.

- The hydraulic performance of the tailrace was assessed using HEC-RAS 1D based on the information that were published prior to 1980. While WSP has not reviewed the hydraulic model used for the assessment, the assumptions and results in the report appear to be appropriate for the purposes and limitations of the Damwatch report.
- The as-built information (geometry, rock sizing, condition, etc.) were limited to referencing documents pre-dating the year 1980. The report does not validate that the geometry and condition reported prior to 1980 is representative of the asset's current state (as of 2025) with reference to any bathymetric survey data, observed information or condition assessment.
- The as-built rock rip rap D50 on the chute between 'sills 1-2. 2-3 and 3-4' (or the upstream 60 m length) are suitably sized to resist scour from the design flows considered based on the Isbash equation. The as built rock sizes in remaining downstream sections on the chute are below the minimum recommended, although assessed to be sufficient should the water level of Lake Pūkaki is drawn down to RL 513 m and flows up to 105 m^3/s flow on the chute. The Damwatch report does not confirm the scour resistance of the chute if Lake Pūkaki water level is drawn down below RL 513 m or if flows on chute exceed 105 m^3/s.
- The temporary weir and chute have been reported to be stable through the construction and early commissioning stages. The Damwatch report also suggested that any material forming on the chute may over the years may provide additional scour resistance but would rapidly erode if the lake level is drawn down below RL 520 m. Overall, the Damwatch report does not confirm long-term stability of the weir and chute structures in the event lake levels are drawn down to RL 513 m with flows exceeding 105 m^3/s. There is no comment of condition degradation impacting the performance or stability of the structure. The report acknowledges 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".

3 CURRENT HYDRAULIC ASSESSMENT

3.1 WEIR

WSP has completed a hydraulic assessment of the temporary tailrace weir and outfall structure at Tekapo B Power Station. The review examined how varying reduced lake levels and discharge flow rates over the weir affect its ability to maintain an adequate tailwater level (TWL) at the power station.

3.1.1 EXISTING INFORMATION

As-built drawings and other historical information shared by the client was reviewed to extract key weir and chute geometry details related to the hydraulic performance of the structure. These were cross-referenced to data from a survey conducted by WSP on 15/09/2025. The parameters were used to represent the weir and outfall structure are shown in Table 3-1 below.

Table 3-1: Weir and outfall structure dimensions and levels

Parameter	Value (RL, m)	Assumption	Source
Weir Crest Level (m Lyttleton 1937)	517.99	Lowest elevation along crest	WSP survey dated 15/09/2025
Weir Crest Length (m)	43.7		WSP survey dated 15/09/2025
Chute Bottom Level (m Lyttleton 1937)	502		WSP survey dated 15/09/2025
Chute Width (m)	34.2	Smallest cross-sectional width along chute	WSP survey dated 15/09/2025
Chute Length (m)	304.2		WSP survey dated 15/09/2025

3.1.2 SCENARIOS

The scenarios confirmed with Genesis Energy to be assessed in this hydraulic assessment are as follows:

- Station Discharge:
  - 40 m^3/s – Minimum station flow at 55 MW.
  - 112.5 m^3/s – Maximum station flow with both units running at 80 MW and a minimum tailwater level of RL 518.0 m.
- Lake Pūkaki Level
  - RL 513 m (Lyttleton 1937) – Proposed minimum lake level.
  - RL 517 m (Lyttleton 1937) – Slightly below weir crest level.

3.1.3 WEIR EQUATION (FIRST PRINCIPLES)

The existing weir structure is considered a trapezoidal broad-crested weir. Flow over the weir was represented by the broad-crested weir flow equation (Equation 1)

Equation 1

$$Q = C_d L H^{\frac{3}{2}}$$

Where; C_d is the coefficient of discharge

L is the length of the weir

H is the height of water above the weir crest

The following assumptions were made throughout this assessment:

- C_d = 1.67 based on field geometry (Streeter & Wylie, 1981).
- Weir dimensions and levels were based on survey data (WSP completed 15/09/2025).
- The weir crest level was taken as the minimum surveyed level along the crest at RL 517.99 m in comparison to the average surveyed crest level of 518.03 m. For the purposes of assessing with the weir equation, the weir crest was set to RL 517.99 m.
- Flow along the side slopes of the trapezoidal weir was represented by applying the broad-crested weir equation (Equation 1) at 1 m intervals.

The results of the application of the broad-crested weir equations for the two discharge scenarios are shown in Table 3-2 below.

Table 3-2: Resulting TWLs for two discharge scenarios

Station Discharge (m^3/s)	Head at Weir or TWL (m Lyttleton 1937)
40	518.65
112.5	519.26

The TWL within the tailrace channel downstream of the powerhouse is likely to be similar or greater than the above levels with the proposed minimum and maximum flow rates of 40 and 112.5 m^3/s applied.

3.1.4 HIGH-LEVEL HYDRAULIC MODEL

A high-level hydraulic model was developed using HEC-RAS 2D. This was undertaken to validate the outcomes broad-crested weir equation and identify if there are locations along in the tailrace (upstream of the weir) that could be subject to outflanking, i.e., water spilling outside the weir and chute. Note that this model is not intended to ascertain the effects of existing chute to the weir water surface elevation due to the limitations of using HEC-RAS 2D for highly turbulent flows.

3.1.4.1 GROUND MODEL

The ground model was based on the recent bathymetric survey (WSP, Sept 2025). There were missing ground levels in surfaces upstream of the weir (refer to yellow regions in Figure 3-1). For the purposes of the assessment, surveyed levels surrounding these areas with missing levels have been interpolated to generate ground levels. This would avoid assuming a "glass-wall"[1] effect but may still over- or under-estimate water storage above the weir.

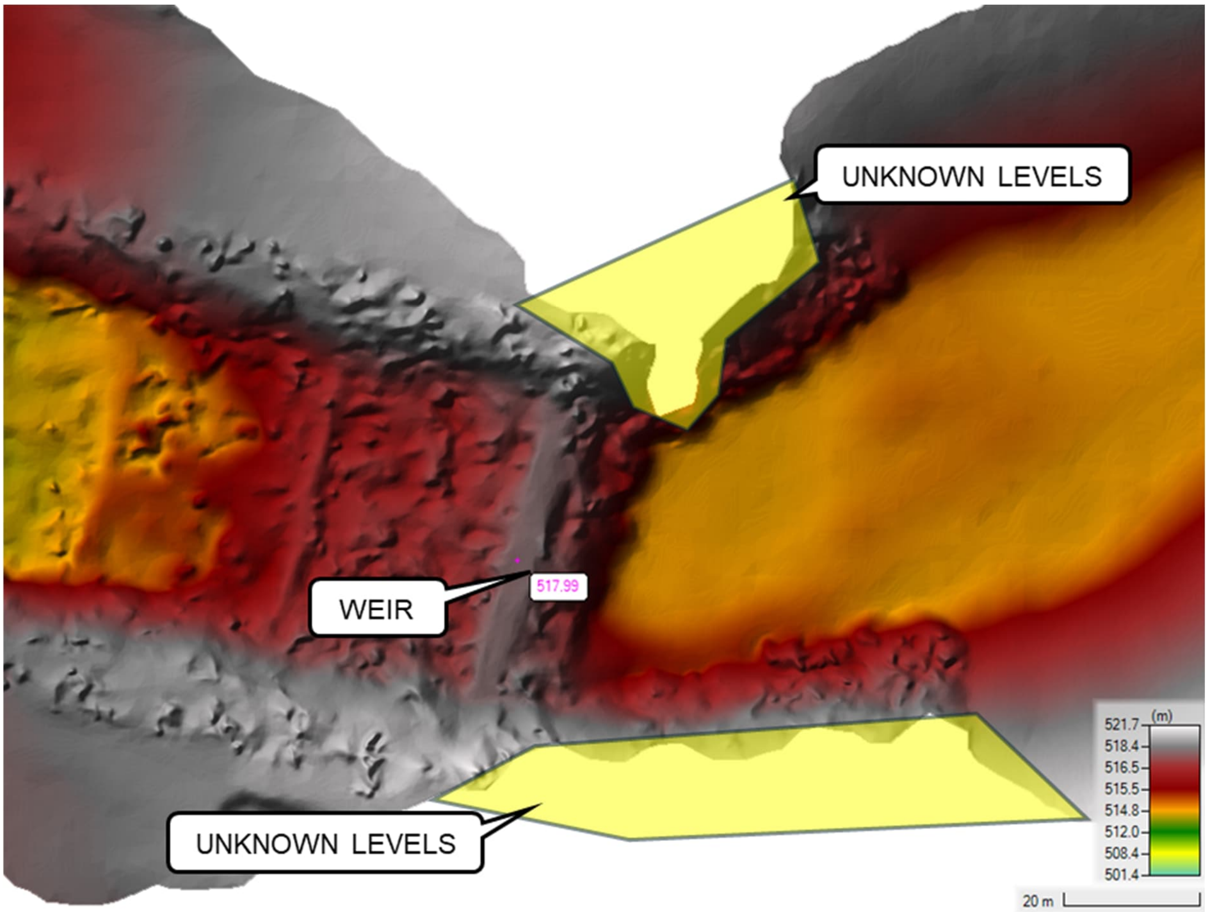

Figure 3-1 Missing ground levels upstream of the weir site from the recent bathymetric survey (WSP, 2025)

3.1.4.2 KEY HYDRAULIC MODEL PARAMETERS

Below lists the key hydraulic parameters assumed:

- Roughness (n) is around 0.030. This was considered reasonable as the invert of the tailrace is 3-4 m below the weir crest and the effects of frictional losses from the invert material have less effects to the hydraulics above RL 518.0 m.
- The minimum and maximum flows (40 and 112.5 m^3/s) were applied at a constant rate until the hydraulic results reach equilibrium (i.e. no changes in water surface elevation and velocities past a specific time).
- Shallow Water Equations: Eulerian-Largrangian Method (SWE-ELM) was used for estimating hydraulics, considering a Conservative Turbulence Model. This is more appropriate where hydraulic phenomena such as hydraulic jumps and eddies are likely to occur.
- Grids were sized to 1.5 m x 1.5 m and time steps were to achieve a maximum Courant number of 0.5 to improve model confidence particularly with abrupt changes in ground levels.

[1] Glass-wall" effect in hydraulic modelling refers to the unrealistic reflection of waves or flow at model boundaries, caused by rigid, impermeable walls that do not allow energy dissipation, leading to distorted results near the edges.

3.1.4.3 MODEL LIMITATIONS

The WSP report (New Zealand Diving & Salvage, 2025) states that the weir is covered with a layer of sediment approximately 0.030 – 0.040 m thick, affecting the surveyed crest level. If the weir was returned to service, high flow conditions will likely erode and eventually clear the sediment layer from the weir, reducing the crest level and hence the resulting tailwater level by approximately 0.030 – 0.040 m. For the purposes of ensuring accuracy in the hydraulic modelling, these tolerances are considered acceptable.

The HEC-RAS 2D model results downstream of the weir have reduced confidence due to inherent limitations of HEC-RAS when applying the SWM-ELM approach on steep terrain and under highly turbulent flow conditions.

3.1.4.4 RESULTS

Figure 3-2 shows the predicted flow extent, velocities and particle tracing under two design flows considered. The design flows through the tailrace are predicted to be contained with the tailrace banks, albeit based on assumed bank levels on the immediate upstream. Eddies are likely form on both banks of the tailrace just upstream of the constricted section towards weir, although the flow velocities are <0.3 m/s. Flow velocities upstream of the weir are below 0.6 m/s – as a reference, Austroads riprap sizing guide do not specify a class of rock protection for floodway flow velocities of less than 2 m/s (Austroads, 2023).

Figure 3-3 shows the resulting water surface elevations based on the maximum and minimum design flows. The predicted water surface elevations at the weir are RL 518.80 m and RL 519.30 m at 40 m^3/s and 112.5 m^3/s, respectively. Although differences between the weir equation and model results are 40-150 mm, all estimated water surface elevation at the weir is above the operational minimum of RL 518.00 m.

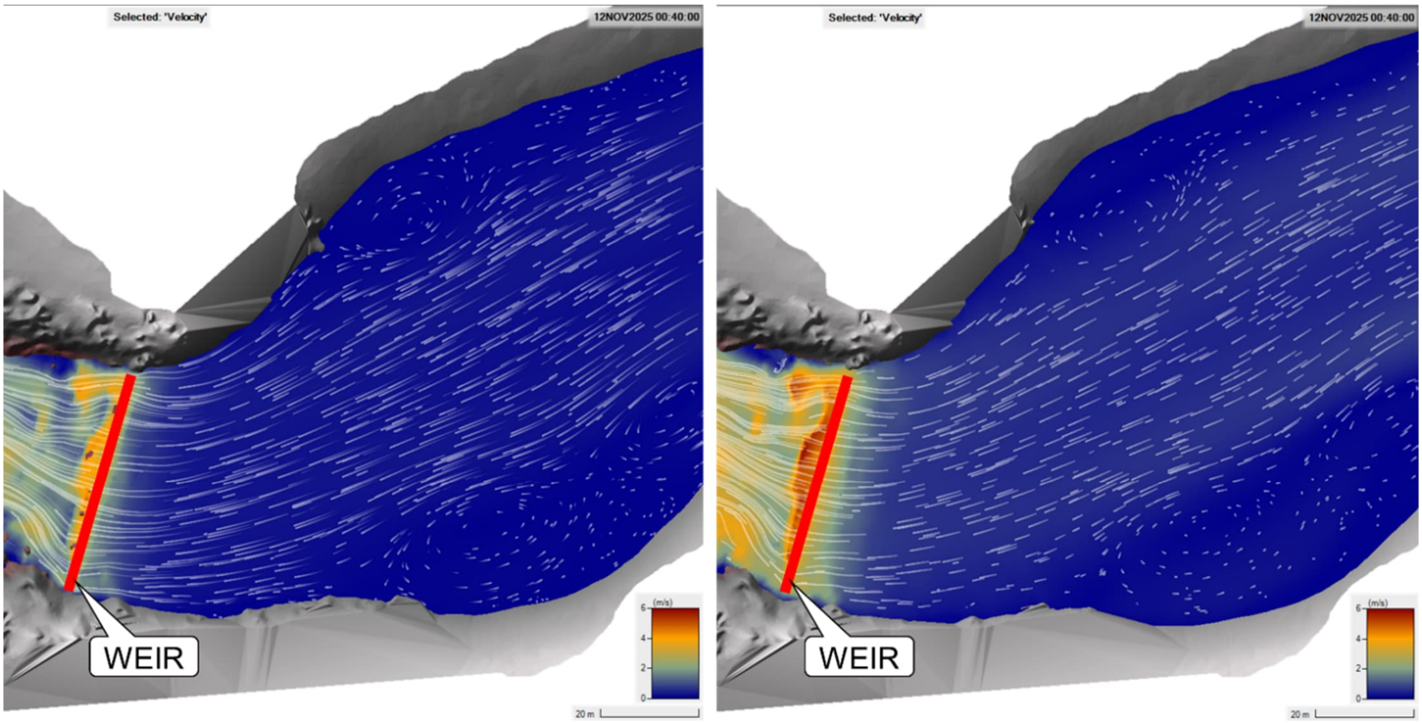

Figure 3-2 Predicted flow extents, velocities and particle tracing at 40 m^3/s (left) and 112.5 m^3/s (right)

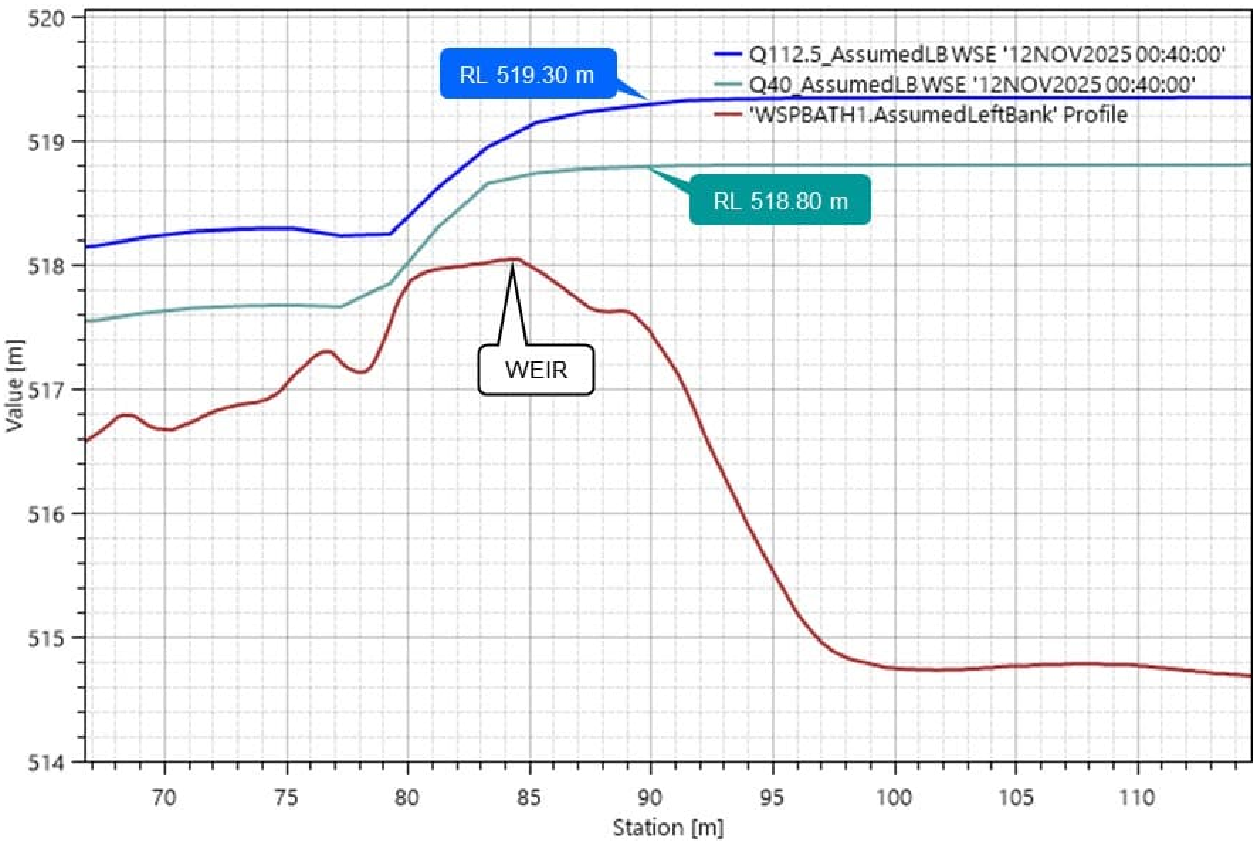

Figure 3-3 Predicted water surface elevations along the tailrace centreline

3.2 CHUTE

The performance of chute has been assessed based on the latest bathymetric survey and dive report completed by WSP.

3.2.1 GROUND LEVELS

Figure 3-4 shows the bathymetric survey of the chute completed during the WSP survey. Surface hillshading indicates that rock rip rap is either partially or fully buried under silt deposits. The dive report also suggests that the survey has likely picked up silt deposition over the sill structures and rock rip rap.

Figure 3-5 plots the longitudinal profile of the chute centreline from the weir to the downstream survey extent. The figure includes a 5% grade line that represents the original design longitudinal grade for comparison. Ground level drops of up to 1 m deep are found at around Station 50 m and 175 m.

Despite indication of change in profile compared to when the chute was designed and constructed, silt deposition may still be masking actual ground movement (e.g. scour holes, rock riprap displacement) that may have occurred after the lake level has risen above the weir.

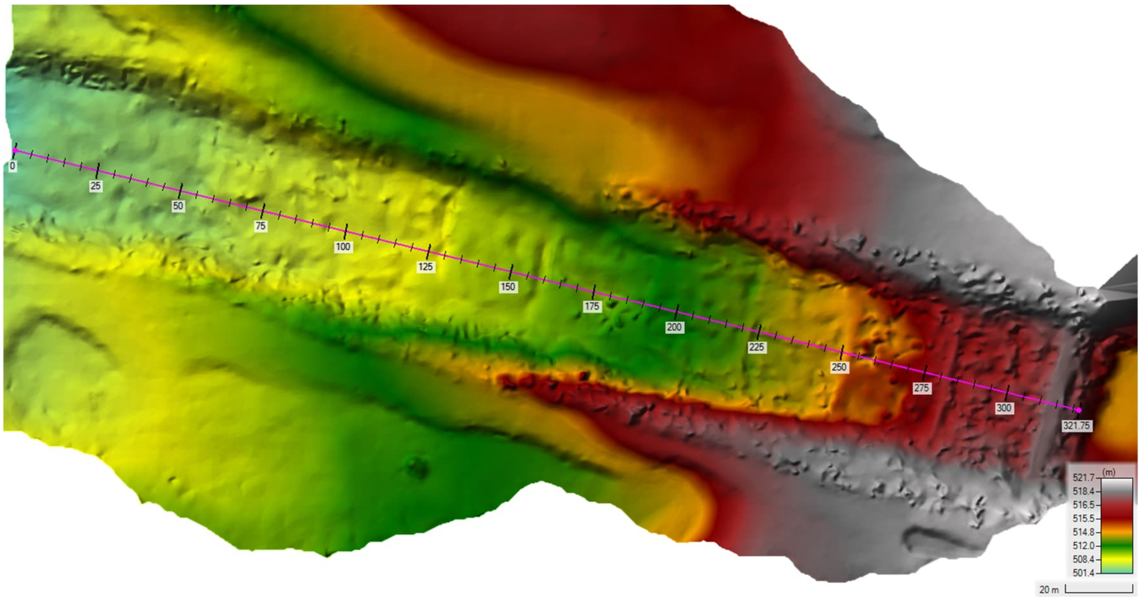

Figure 3-4 Bathymetric survey of the chute

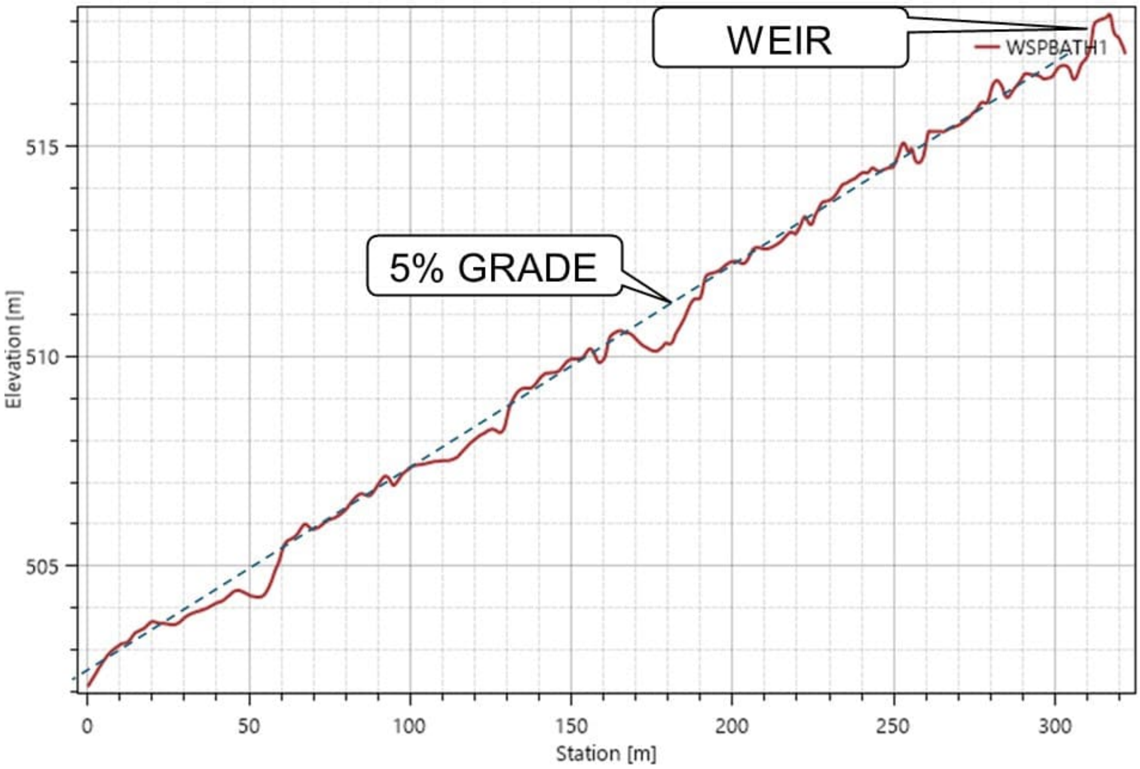

Figure 3-5 Longitudinal section of the chute centreline ground level

3.2.2 STRUCTURES

The dive report stated that there is concrete cover loss and exposed reinforcing steel at the weir structure. This could indicate that similar issues may have occurred in the sill structures that are currently covered under silt deposits.

Exposed defects, such as concrete spalling and corroded reinforcement could be repaired while submerged by a diver. However, replacement or repositioning of rip rap elements would require the water level to be lowered significantly to allow appropriate access and visibility to undertake the works.

Therefore, while minor superficial repairs to concrete elements could be carried out whilst submerged, any more significant operations would require a lowered water level. Both repair methods would require station outage.

3.2.3 CONVEYANCE

Figure 3-6 shows the water surface elevation at the chute at 60 m downstream of the weir, as predicted in the HEC-RAS 2D model. Model results show that the top of right bank is 1 m above the water surface elevation at a design flow of 112.5 m³/s. Despite the uncertainties of using HEC-RAS 2D for this application, 1 m of freeboard from the chute flow surface to the top bank improves confidence that the chute is unlikely to overtop at the maximum design flow.

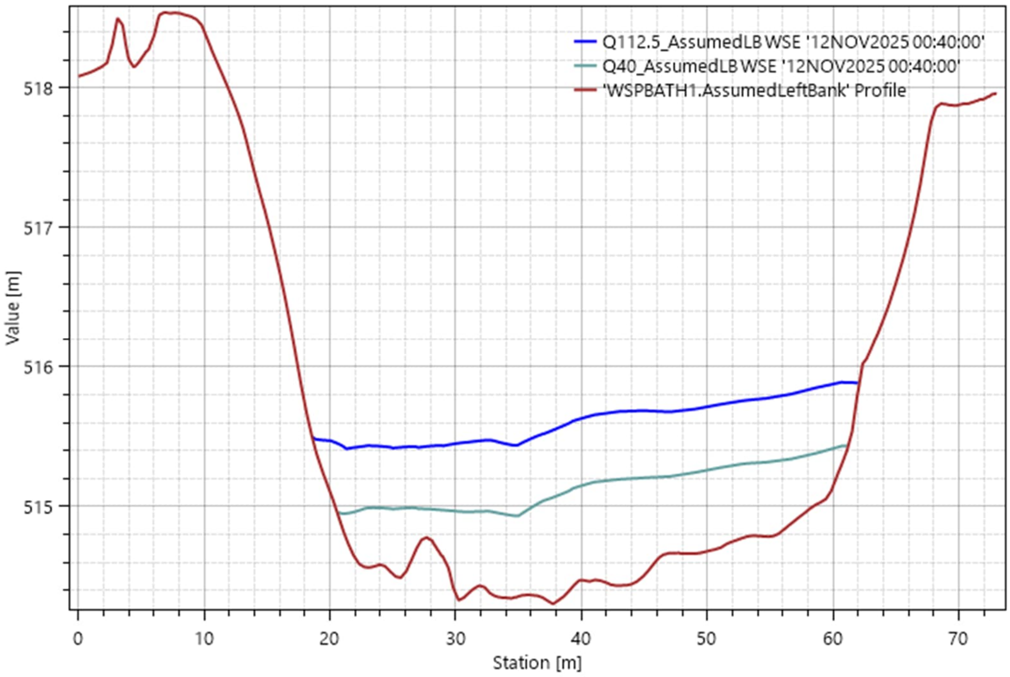

Figure 3-6 Predicted water surface elevation 60 m downstream of the weir at 40 m³/s (bottom) and 112.5 m³/s (top)

3.2.4 ROCK RIP RAP LINING

The required rock rip rap D50 for the current state of the chute is estimated using the methodologies from two publications:

- 'Design of Rock Chutes' (K.M. Robinson, 1998)
- 'Design of Rock Chutes for the Stabilisation of Channel Beds' by (Winston, 2003)

Both methodologies have resulted to similar results at a recommended minimum D50 of 600 mm. This includes a factor of safety of 1.30 to D50 sizing as recommended by Keller (2003) for 'any major structure or to a structure where failure would threaten an asset or cause major loss'. According to the construction report (Genesis Energy, 1979) , the rock sizes that were used for the first 60 m of the chute from the weir meet the estimated minimum D50, despite that the original design allowed for steeper longitudinal slope. The downstream sections however are smaller than 600 mm.

The as-built drawings indicate that rock rip rap lining is underlain by free-draining gravel (maximum particle size of 150 mm). Beneath this gravel layer is sand, which material specification document stated that it is 'uniformly graded conforming to the attached envelope'. There is no direct information from the design and construction reports that rules out the migration of fine material beneath the free draining gravel and sand layers, which could impose foundation stability risks under the rock rip rap and concrete sills from interstitial flows through the chute armouring layers. We also note from historical reports that two scour failures on the rock chute were experienced and repaired during operations of the temporary tailrace including a large hole located downstream of sill 9 (Figure 3-7 and Appendix A). Correspondence from the Ministry of Works and Development also states “it should be stressed that the chute is a risk structure and minimal use only can be recommended” (Ministry of Works and Development , 1977).

Figure 3-7 Tailrace with scour failure 1977 experience 12 days after operation (source Genesis Energy)

WSP’s survey confirmed significant silt deposition within the chute, which may be concealing underlying condition and performance issues that have developed after the lake level rose above the weir. WSP has also confirmed there is concrete cover loss and exposed reinforcing steel at the weir structure which may indicate similar issues in the sill structures that are currently covered by silts. If left untouched after the lake level is drawn down, the silt deposits may likely be washed away by the tailrace flows that could expose the rock rip rap and sill structures, which then would be subject to direct impacts from high-velocity flows. It is currently therefore unknown whether the current condition of the chute and riprap is stable enough to safely convey the maximum and minimum design flows.

4 CONCLUSIONS

WSP's review of the Damwatch Engineering report on the Tekapo B temporary tailrace weir and chute found that, while the hydraulic assessment and assumptions appear appropriate for the available historical data , the report does not consider the impact of condition degradation on the stability of the structure under reduced lake levels and high-flow scenarios.

Our hydraulic assessment of the weir structure concludes that tailrace water levels downstream of the powerhouse are expected to remain at or above RL 518.00 m, in the scenario that Lake Pūkaki is drawn down below RL 518.00 m. This is under the premise that the weir structure will continue to be in place for the remainder of its service life and while the lake level is below RL 518.00 m.

The rock rip rap sizes in the upstream 60 m of the chute are anticipated to meet the recommended minimum D50 for the maximum and minimum design flows and at the current 5% longitudinal chute grade. The rock rip rap sizes downstream of this section are below the recommended D50, although the tailwater effects of drawn down lake level may reduce the effects of scour towards for the chute section near the lake level due to high-velocity flows plunging into the lake water surface and breaking energies. As a result, the remaining downstream section may have a reduced minimum D50 rock riprap size; however, our assessment has not verified this minimum size or confirmed that these downstream sections are adequately protected to resist scour from a design chute flow of 112.5 m^3/s, which is higher than the 105 m^3/s previously assessed by Damwatch.

We have confirmed, through hydraulic modelling, that with the current chute geometry the maximum design flow will unlikely overtop the chute banks with an estimated water surface elevation that is 1 m below the top of bank. This indicates that the chute can safely convey the design flows to the lake, provided that the geometry remains stable and consistent throughout its service life.

The recent bathymetric survey shows that there are 1 m drops in the chute invert, which may cause cascading flows through the chute that could increase shear forces on top of high-flow velocities and increase vertical loading to the chute. This could compromise the foundation of chute lining resulting from increased vertical interstitial flows.

Historical reports have confirmed there has been previous failures experienced in the chute including the formation of a scour hole underneath sill 9 near the downstream end of the chute during the early months of commissioning. Our survey indicates significant silt deposition and structural deterioration at the weir and chute, creating uncertainty about their ability to safely convey design flows once lake levels are reduced.

With the understanding that the chute and sill structures were originally constructed for temporary use, have a history of documented failures, and have observed condition deterioration, the risk of failure during lake drawdown is now greater than it was at the time of their original construction.

5 LIMITATIONS

This report ('Report') has been prepared by WSP New Zealand Limited ('WSP') exclusively for Genesis Energy Ltd ('Client') in accordance with the ACENZ Conditions of Contract for Consulting Services, GE-CT25-070, and the Statement of Work No. 01, dated 14/08/2025 ('Agreement').

Permitted Purpose

This Report has been prepared expressly for the purpose of to undertake a hydraulic assessment of the temporary tailrace weir and outfall structure at Tekapo B Power Station ('Permitted Purpose'). WSP accepts no liability whatsoever for the use of the Report, in whole or in part, for any purpose other than the Permitted Purpose. Unless expressly stated otherwise, this Report has been prepared without regard to any special interest of any party other than the Client.

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Memo

To: Andrew Balme – Engineering Manager, Civil & Dam Safety

From: Jeanette Tucker, Jan Stanway, Jeremy Robertson

Cc: Lewis Thomas

Ref: 2-38161.00 Tekapo B Submerged Weir

Date: 26-03-2026

Subject: *Tekapo B Power Station Submerged Weir – Damwatch Document Reviews*

REV	DATE	DETAILS
1	26/03/2026	FINAL

	NAME	DATE	SIGNATURE
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1. Purpose of Memorandum

This memorandum provides WSP New Zealand Limited comment on the following Damwatch Engineering memorandums:

- "*Tekapo B PS Temporary Tailrace Weir and Rock Chute; Hydraulic Review of Weir and Chute – Addendum report*" dated 5/12/2025
- "*Tekapo B Power Station tailrace weir and chute – condition assessment and review of bathymetric survey data*" dated 27/02/2026

This memorandum forms an addendum to the following WSP reports:

- "*Tekapo Submerged Weir Bathymetric Survey and Hydraulic Assessment*" dated 19/12/2025
- "*Tekapo Submerged Weir – Structural Condition Assessment*" dated 16/12/2025

2. Temporary Structure and Risk Management

Both the WSP and Damwatch assessments recognise that the tailrace weir and chute were designed as temporary structures. Reiterating our original report, historical reports have confirmed there has been previous failures experienced in the chute including two scour failures during the early months of commissioning. Correspondence from the Ministry of Works and Development also states, *“it should be stressed that the chute is a risk structure and minimal use only can be recommended”* (Ministry of Works and Development, 1977).

Damwatch’s conclusions recommended risk management measures including frequent visual monitoring during operation, maintenance of readily available riprap stockpiles, and the ability to undertake rapid remedial works should any signs of erosion or abnormal behaviour be observed.

The sections in this memorandum consider the various areas of risk to the temporary tailrace weir and chute as a result of drawing Lake Pukaki down to RL 513m and provide WSP recommended actions to address the risks.

3. Validation of Current Geometry and Condition

WSP previously noted that the Damwatch hydraulic review relied predominantly on pre-1980 as-built information and did not confirm whether the geometry and condition of the tailrace weir and chute were representative of the asset’s current state. In the addendum report, Damwatch have included bathymetric survey and dive inspection commissioned by Meridian Energy (February 2026). The Damwatch assessment of the survey results against the as-built sill levels confirms that the structure remains broadly consistent with its original geometry.

We note that the summary of the Damwatch assessment Table 1 is based on the sill height only and does not assess the levels associated with the riprap. Our comparison of Damwatch’s longitudinal sections with the WSP survey, show survey levels are very similar with both surveys indicating a ground level drop of 1m deep forming a depression in the chute at approximately chainage 125. Our concerns at this location are that cascading flows through the chute could increase shear forces on top of high-flow velocities once the structure is brought into service and increase the risk of local scour and progressive chute failure.

While Damwatch characterises the loss of finer material potentially as an early-life process rather than progressive deterioration while being submerged, it nonetheless acknowledges that this condition leaves sections of the channel bed exposed to erosion under high-energy flow conditions. Damwatch also states “There is evidence of minor lateral movement of riprap material away from the edge of some vertical concrete faces of sills (probably due to the jostling of individual riprap stones into a more well-supported resting position by power station discharges during original operation of the structure in 1977-78)”.

This indicates a clear separation between the riprap and the concrete structure, with the potential for exposure of the underlying foundation sands. Additionally, differential elevations between the sill and surrounding riprap can locally increase turbulence intensity, creating zones of elevated hydraulic loading that may cause riprap instability.

4. Silt Deposition

The Damwatch assessment summarises the silt as a superficial deposit that does not materially affect structural or hydraulic performance once flows recommence stating *“The surfaces of the structure are covered only in a thin coating of silt”.*

WSP’s assessment of survey completed for Genesis, is that silt is partially or fully covering riprap, potentially masks underlying defects that may be exposed once flows recommence. From an operational risk perspective, WSP’s concern is not the presence of silt itself, but what it may be concealing, such as displacement of riprap, local scour, or foundation exposure which could be revealed rapidly once flows recommence upon re-operation.

5. Hydraulic and Rock Chute Stability at Higher Flows

WSP previously identified that the original Damwatch review confirmed rock chute stability only up to 105 m^3/s and did not explicitly assess higher flows. The Damwatch addendum report has extended the hydraulic modelling and stability assessment to a discharge of 115 m^3/s. The addendum confirms, using established riprap stability criteria, that the rock-lined chute remains stable for flows up to 115 m^3/s provided Lake Pūkaki is drawn down to approximately RL 514 m.

WSP reiterates our previous assessment that these results assume that the geometry remains stable, is well graded and consistent throughout its service life, and does not consider the impact of condition degradation on the stability of the structure. We note that condition degradation could include foundations of the chute lining or areas currently covered by silt deposits which may not be currently visible through dive inspections.

Due to the layer of silt deposition, there is insufficient evidence to confirm that the riprap is uniform in placement, grading and coverage. The presence of fine material mixed with the larger rock may winnow over time and fail. While the riprap at the calculated D_{50} size is considered stable when first installed, the long-term stability is dependent on the riprap remaining interlocked with other large particles.

In their assessment of minimum D_{50} sizing, Damwatch have applied the Ishbash criterion which assumes that there is steady, relatively uniform flow and minimal air entrainment. It is our opinion that the application of the Ishbash approach for rock chute environments tends to over-estimate long term stability (i.e. under-size rock) as it does not fully account for the turbulent high energy conditions and potential for hydraulic jumps to form.

Alternative methods such as Abt and Johnson have been empirically developed for the design of steep rock-lined spillways and thus include allowance for more turbulent flow. We consider these forms of empirically derived rock sizing equations to be more appropriate in this environment. Whilst the areas with a higher D_{50} likely have a reasonable factor of safety, areas with a D_{50} of 300 mm would not be expected to be stable long term. Especially given the channels hydraulic loading is much greater than that of say an emergency spillway. The channel may effectively run at or close to its design rate for extended periods of time and on an annual basis.

6. Structural Performance

This section provides comment on the structural performance of the weir and sills (also known as ribs) of the temporary tailrace weir and chute as discussed in Damwatch technical memorandum "*Tekapo Submerged Weir – Structural Condition Assessment*" dated 16/12/2025.

6.1 Purpose of the concrete weir and sill beams

As discussed in the Damwatch technical memorandum (Section 5.1.1), the purpose of the concrete weir and sill beams are to prevent 'washout' or 'rolling away' of the rounded boulders used as riprap. Damwatch note this is common industry practice for riprap lining of steep open channels, revetments or similar hydraulic structures.

Significant movement of the riprap can result in scouring of the chute. To achieve the required performance of the tailrace and chute structure the weir and concrete sill beams must remain in place and continue to hold the riprap in place whilst the water flows down the chute. To do this the weir and sill beams must have sufficient capacity during every draw down such that they can sustain the lateral gravity loads from the rock riprap bearing against the weir and sill beams and the hydraulic lateral force caused by water flow down the chute.

6.2 Stability of structure for future use

WSP generally agrees with the list of circumstances that will compromise the performance of the temporary tailrace and chute structure as listed in Section 6.1 of Damwatch's technical memorandum, although potential damage to the existing concrete weir and sills (also referred to as ribs) also needs to be considered. The circumstances that will compromise the structural performance of the temporary tailrace and chute structure are listed below:

- A loss of ground (undermining) around any weir or sill that results in movement (lateral displacement or rotation) of the concrete that forms the weir and sills. Any movement that results in movement of riprap across the weir and/or sills compromises the performance of the weir and chute.

- Loss of structural connection of the weir or sill 'cap' (also referred to as 'upper weir' by Damwatch) from the lower concrete base. Refer to Figure 1 below for clarification the cap and base sections of the weir and sills.
- Existing damage to the concrete weir and sills has the potential to reduce the capacity of the concrete to sustain the lateral loads on the concrete to support the riprap and the hydraulic forces from water.
- Damage to the concrete weir and sills due to concrete condition when they are exposed due to draw down and reused as a weir and chute in the future.

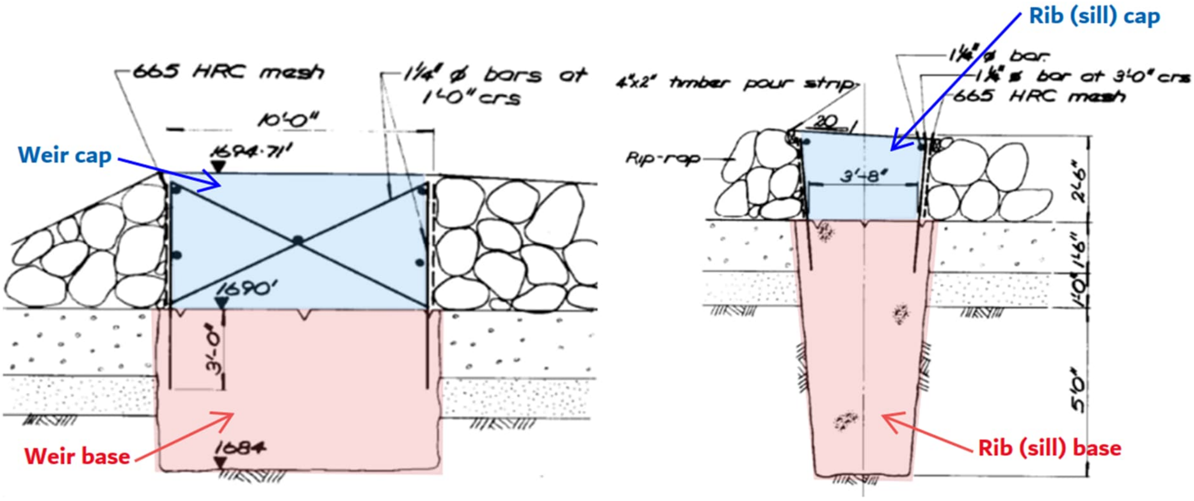

Figure 1: Weir and rib (sill) concrete structure showing the cap and base sections

6.2.1 Implications of undermining of concrete weir and sills

If the ground around the concrete weir and sills is undermined such that a gap occurs between the concrete base and the ground in which the weir and sill bases are founded, the stability of the weir or sill will be compromised. If this occurs the concrete may displace horizontally or rotate such that riprap could pass over the beam. In the worst case the foundation may be undermined to the extent that the concrete cap may be dislodged down the slope.

Significant movement of the riprap can result in scouring of the chute.

6.2.2 Implications of loss of connection of the weir and sill caps from the concrete base

The final paragraph on page 5 of the Damwatch technical memorandum states that concrete for the weir and ribs was placed in two stages during construction *"… the lower lift placed in the excavated trench after completion of the earthworks for the channel bed, and the upper lift placed against steel forms at some ribs and against riprap in others*". Based on the MOW 'Temporary Tailrace Details – Sheet 2' – drawing T38/01/8 dated October 1977 (referenced by Damwatch as figure 3) states that the final rib (sill 11 as called up by Damwatch) used standard shutters whereas the remaining ribs and weir as detailed on this drawing shows the riprap held back by mesh connected to the 1.25" reinforcement on the sides of the weirs and ribs and the concrete was poured up against the rip rap. Figure 4 in the Damwatch technical memorandum confirms this construction sequence.

Damwatch note that there is no information available on the construction of the horizontal joint between the two concrete lifts and they also note that there is no information currently available on the current condition of the lift construction joints.

Damwatch technical memorandum (1st paragraph of section 5.1.3) reiterates that ".. *the reinforcing bars and mesh were provided to prevent interference in the concrete from riprap already placed on site. This purpose was fulfilled after concrete placement and hardening. The exposure of the reinforcing steel in some areas noted in the dive inspection therefore does not present any risk of impairment of the intended functionality of the weirs*…'. The intended purpose of the concrete weir and ribs was to temporarily contain the riprap in the chute. They fulfilled the intended purpose when Lake Pukaki was filled and the weir and chute were submerged.

It appears to be understood by both WSP and Damwatch that the reinforcement that was cast into the base section of the weir and sills was provided to create a basic formwork to support the riprap. Following riprap placement the cap section of the weir and sills was poured between the riprap. It was not intended to create a reinforced concrete section; the weir and sills act as unreinforced concrete sections. Where the rebars were not surrounded by concrete the vertical 'dowel' bars do not act as normal reinforced concrete dowels to provide a shear and overturning connection between the cap and base sections of the sills. Instead, the reinforcement may have held the unreinforced concrete in place where there was no gap between the reinforcement and the concrete, and resist shear and overturning demands by bearing on the downstream reinforcement.

Whilst the weir and sill concrete would have flowed out into the voids between the riprap when the caps were poured, where the riprap was placed hard up against the reinforcement, the reinforcement would have no concrete cover in those locations. Cover concrete protects reinforcement from corrosion as the concrete is alkaline. Given that the intended function of this structure was temporary and used for around 14 months over an 18 month period, corrosion of those reinforcing bars was likely considered acceptable as they are significant bars (1.25 inch approximately 32mm diameter) and it is expected that they could have lost some section and still had sufficient capacity to resist the lateral loads during the intended temporary operation of the chute.

To bring the chute back into operation today the connection of the concrete cap to the weir is essential to enable the weir and sills to achieve their intended design purpose to hold back the riprap during design water flow. The connection of the concrete cap would need to resist the lateral loads due to the gravity load of the upstream riprap bearing on the cap as well as the lateral force on the cap as a result of the hydraulic flow of water down the cute. The lateral loads will create a shear force at the interface of the cap and the base concrete (i.e. at the construction joint) and an overturning force on the cap. Note given the dimensions of the weir cap the weight of the cap is expected to be sufficient to resist any overturning force on the cap. It is possible that the weight of the sill cap may be insufficient to resist the overturning and assuming there is sufficient section of reinforcement remaining in the vertical rebars these could provide the necessary overturning resistance if required. The forces to be resisted are shown in Figure 2 below:

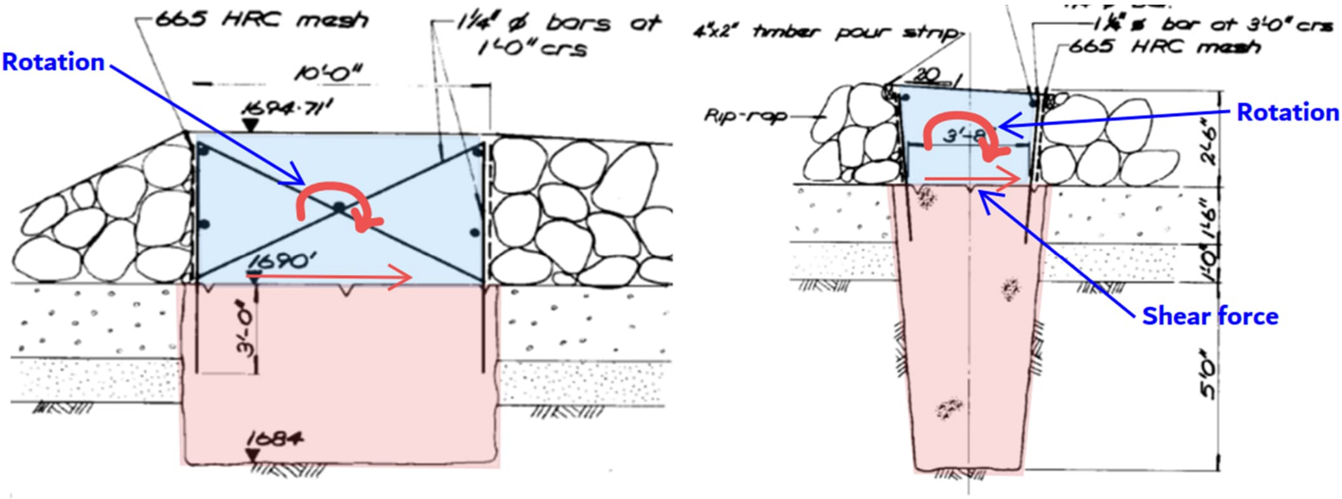

Figure 2: Weir and rib (sill) concrete structure showing loads to be resisted by concrete caps

The mechanism to resist the loads shown in Figure 2 will be by one or both of the following load paths:

1. Where the rebars were not surrounded by concrete the vertical reinforcing bars do not act as normal reinforced concrete dowels that provide shear connection between the different concrete lifts, instead they hold the concrete in place <u>if they are in contact with the concrete</u>. If there is a gap between the rebar and the concrete as shown in Figure 2-5 of WSP Structural Condition Assessment report (dated 19 December 2025), the concrete would need to slide and/or rotate to bear up against the downstream rebar to provide the shear and overturning resistance.
2. It is currently unknown if the construction joint was roughened between the concrete base and concrete cap. As noted by Damwatch (first paragraph page 6) if the standard Ministry of Works' specifications for concrete works were followed for these temporary works, then the construction joints are likely to have been cleaned and roughened before placement of the second lift. Given that this structure was temporary works intended for short duration life it is not certain that the standard concrete specification was followed and the construction joint was cleaned and roughened. If the joint was cleaned and roughened the joint would provide aggregate interlock and resist shear forces in addition to any vertical reinforcement in contact with the concrete. If the joint was not cleaned and roughened the construction joint would have little to no shear resistance and the vertical reinforcement will be required to keep the cap in place.

The current condition of the vertical 1.25 inch (approx. 32mm reinforcement) is unknown. Photographs show corrosion.

<u>WSP recommendation</u>: Given that the importance of keeping the concrete caps connected to the concrete base to achieve the structural performance of the concrete weir and sills, and given that the vertical reinforcement has an unknown extent of corrosion, and is known to have varying gaps between the reinforcement and the concrete it is recommended drilling and grouting new steel dowels along the length of the weir and sill beams, to ensure the connection between the concrete cap and base remains during use of the structure.

6.2.3 Implications of existing damage to concrete weir and sills

The Damwatch technical memorandum includes the bathymetric survey undertaken 03 February 2026. In Table 1 of the technical memorandum Damwatch presented a comparison of the surveyed and as-built levels to the top of the weir and sills. The as-built survey data was taken along the centreline of the weir and sill beams with no RL's taken to the left- and right-hand ends of the beams. The 2026 bathymetric survey captured RL's along the centreline as well as RL's at the left- and right-hand ends of each sill beam.

WSP has considered the alignment of the weir and sill beams along the length using the data from Damwatch Table 1, the results are shown in Table 1 below.

Table 1: Current alignment of top of concrete weir and sill beams

Sill (Rib)	RL (m)			Diff LH to CL	Diff CL to RH	Comment
	LH	CL	RH			
1 (Weir)	**517.86**	**517.81**	**517.75**	**50**	**60**	**tilt**
2	516.41	516.36	516.42	50	-60	50mm drop to centreline on both sides - has beam broken?
3	514.93	514.97	514.81	-40	160	LH side drops 40mm from centreline, RH side drops 160mm from centreline
4	**513.27**	**513.21**	**513.15**	**60**	**60**	**tilt**
5	511.78	511.85	511.87	-70	-20	70mm drop from centreline to LH side. RH side level
6	**510.05**	**510.05**	**510.04**	**0**	**10**	**level**
7	**508.81**	**508.83**	**508.82**	**-20**	**10**	**level**
8	**507.04**	**507.12**	**507.22**	**-80**	**-100**	**tilt**
9	505.33	505.44	505.42	-110	20	110mm drop from centreline to LH side. RH side level
10	**503.85**	**503.88**	**503.85**	**-30**	**30**	**relatively level**
11	**502.49**	**502.41**	**502.32**	**80**	**90**	**tilt**

The weir (sill 1) and sill beams 4, 6 to 8, 10 & 11 all appear relatively level or at a constant tilt from right to left or left to right, the current alignment of the weir and sill beam suggests there is no deformation of the sill beam, there may have been settlement to cause tilt of the beams or they may have been constructed that way.

Sill beams 2, 3, 5 and 9 have unusual readings that warrant further investigated to confirm if there is existing damage to the concrete weir and sills that would compromise their performance. The RL's on the top of beams 3, 5 and 9 show half of the beam essentially level and the other half drops by 70mm to 160mm. The ends of sill beam 2 have dropped 50 to 60 mm relative to the centreline.

As the concrete sill beams are unreinforced (note the discussion in Section 6.2.2 regarding how the sill beams were constructed) should one end of the sill beam foundation be undermined or settled relative to middle of the sill beam, the beam will crack. It is noted that dive photographs show what is referred to both in WSP's condition assessment report and Damwatch's technical memorandum as channels across the top surface of the sill beams. Investigations need to be undertaken to rule out the potential that the indentations observed on the top surface of the sill beams are not in fact large cracks.

If the sill beams have cracked in a way that aligns with the survey RL's this indicates that the crack would extend from top to bottom of the sill beam. It is also possible that there are multiple cracks but as it is unreinforced the potential for a large single crack is plausible. The purpose of the sill beams is to hold back the riprap on the relatively steep 1 in 20 slope under both gravity and hydraulic forces. If the beam has cracked there is a higher risk of movement of the concrete sill beams and potential for the cap of the sill be dislodged and move with the riprap than if the beam has not cracked.

WSP Recommendation: Remove all vegetation and silt and thoroughly clean the surfaces of the weir and sill beams and undertake detailed crack investigation of concrete using divers.

6.2.4 Implications of damage to concrete weir and sills due to the condition of the concrete

Implication of frozen ground during winter months

Freeze-thaw attack is a major cause of concrete degradation in cold climates, which occurs when absorbed water freezes and expands within the pores of the concrete, creating internal pressure that exceeds the concrete's tensile strength. This cyclic damage can lead to surface scaling, cracks, pop-outs, and potential structural failure.

Whilst the temperature on the top surfaces of the weir and sills will likely be held at a temperature above zero, due to the water flowing over the weir and down the chute, seasonally frozen ground occurs in this location during the winter months and this means there is a risk that the concrete weir and sills may be subject to freeze-thaw cycles if the lake is lowered during winter conditions.

Air-entrained concrete performs better under conditions of cyclic freezing and thawing. The concrete specification for the weir and sills is not available; it is therefore unknown if the concrete was air-entrained to provide protection against freeze-thaw attack but as the concrete plant would have been set up for structural concrete it is likely that the concrete was air-entrained. If it was not air-entrained the concrete will have a higher risk of freeze-thaw attack and degradation of the concrete. Taking concrete cores of the existing concrete will help to confirm if the concrete was air-entrained or not.

Implications of surface contamination

Depending on what is growing on the surface of the concrete, the type of biological growth may have biologically produced acids that have eaten away at the concrete cement, this phenomenon is known as biogenic acid attack. If this has occurred this would make the surfaces of the concrete at higher risk of damage and deterioration when put back into use as a weir and chute.

WSP Recommendations:

1. Clean the surface of the concrete weir and sills and take a number of concrete core samples for inspection and testing.
2. Monitor concrete surface performance during operation. If cracks, surface scaling or aggregate pop-out occur, consider the risk of continued use of the weir and chute. If risk deemed acceptable to continue to use the weir and chute, then continue to do so and raise water levels in Lake Pukaki (i.e. shutdown gate 18) to resubmerge the weir and sills and undertake underwater concrete repairs. If continued use of the weir and chute is considered too high risk, Tekapo B power station would need to be shutdown to stop water flow from the tailrace and assess repair options.

7. Conclusions

In summary, the key concerns raised by WSP relating to hydraulic modelling at higher flows and survey have been addressed within the Damwatch Engineering addendum. The addendum and previous reporting demonstrate theoretically that the tailrace weir and chute can perform once operating under steady-state hydraulic performance when flows are re-established and with assumptions regarding structural performance of the concrete weir and sill structures. However, Damwatch do not fully address the operational risks associated with re-activation, particularly the risk of detachment of the concrete weir and rib caps, and the risks during the initial drawdown and recommissioning phase.

Specifically, the assessments do not explicitly consider the effects of rapid silt mobilisation, the exposure of potentially masked localised defects, or the response of areas with reduced interstitial support or non-uniform riprap levels at the point when high-energy flows are first reintroduced.

Whilst Damwatch have acknowledged the risk that the concrete weir and sill performance may be compromised, they have not recommended any actions to address these potential vulnerabilities. WSP have recommended that, due to the extent of unknown risks that could cause relative displacement or dislodgement of the weir and sill caps and therefore allow movement of the riprap over the sill, that providing a new direct connection between the concrete cap and the concrete base is provided prior to drawdown.

WSP also recommend that the unusual survey data recorded for the top surface of some sill beams is investigated to have confidence that significant crack(s) are not present in these beams that may require repair prior to drawdown. This will require extensive cleaning of the concrete surfaces, and it is recommended that concrete cores are also taken at this time to enable concrete testing to be undertaken.

From the survey comparisons there appear to be localised depressions in the riprap, including one over 1 m deep; these deviations from grade may place additional pressure on the nearby riprap and could become a point of failure. We also note that Ishbash is not really appropriate for steep channels with rapidly varied flow and turbulence. The equation was developed for use under relatively uniform flow conditions.

Based on our experience, high-gradient channels are sensitive to relatively small variations in slope and D_{50} and therefore tend to require a generous FoS to achieve longer-term stability. Even with conservative design, some level of repair or supplementary riprap may still be required over time due to the inherent complexity of high-energy flow conditions and the lack of bedload from upstream. The historical damage and associated repair work has highlighted the potential for issues, even after a short period of operation.

This ultimately means that there will likely be maintenance and repair work required if the structure was recommissioned, particularly where the D_{50} is in the order of 0.3 m. Potential over-estimation of channel roughness (e.g. flow depth measurement not accounting for velocity head and air entrainment), and the potential for unevenly distributed flow entering the channel (due to approach curvature) are just two examples of why a large FoS is important for long term stability.

The structural performance of the weir and sill beams is critical to maintaining riprap stability within the chute, and continued performance is dependent on the integrity of the sill foundations and the connection between the concrete caps and bases. Survey data indicate localised elevation irregularities and potential damage to several sill beams, which, if associated with cracking, settlement, or undermining, could allow riprap displacement and initiate progressive chute failure. Given the temporary nature of the original design and the uncertainty surrounding construction joints and reinforcement condition, residual structural risk remains under re-operation. WSP recommends investigation of sill beams where unusual top surface RL's were recorded. This will require removal of silt and vegetation off the concrete to confirm if the sill beam contains significant crack(s).

Damwatch's recommendations implicitly acknowledge that residual uncertainty remains at the point of re-operation, noting the need for operational controls such as maintaining readily available riprap stockpiles and ensuring plant is on standby to facilitate intervention if erosion, instability, or abnormal flow behaviour is observed once flows recommence.

In this context, the recommendations that rely on enhanced monitoring and reactive controls acknowledge that certain risks associated with re-operation are not fully mitigated prior to recommissioning but are instead intended to be managed through operational controls during re-operation. Damwatch's approach recognises that some aspects of performance and stability cannot be conclusively confirmed in advance, particularly given the uncertainties associated with the current condition of the structure.

This position is consistent with WSP's assessment and conclusion that residual risk remains during re-operation of the temporary weir, chute, and sill structures. In particular, the potential for condition deterioration, historical performance issues, and uncertainty associated with submerged elements means that the likelihood of failure during operation is greater than it was at the time of original construction. This increased risk applies to both the chute and sill structures and reflects the temporary nature of the original design, the documented history of scour and repair, and the absence of confirmation that the existing condition is sufficient to safely accommodate the proposed operating scenarios.