Memo

To:	Brent Wilson (Meridian Energy)
From:	Viculp Lal, Grant Webby & Jayandra Shrestha (Damwatch Engineering)
CC:	David Cameron-Ellis
Date:	19 March 2026
Subject:	**Tekapo B Power Station tailrace weir and chute – condition assessment and review of bathymetric survey data**

1 Purpose of Memorandum

The purpose of this technical memorandum is to present observations from a review of a bathymetric survey and the findings of a condition assessment of the Tekapo B Power Station tailrace weir and chute structure. The technical memorandum complements a recent hydraulic review of the structure (Damwatch, 2025a; Damwatch 2025b).

The technical memorandum has been revised from the initial version dated 27 February 2026 following review and comments by Meridian Energy (Meridian) and Genesis Energy (Genesis). The summary document providing Genesis's comments is included in Appendix D of this revised version of the memorandum.

2 Background

The Tekapo B Power Station tailrace weir and rock chute is a submerged structure within Lake Pukaki which is no longer operational. It was originally designed for use as a temporary structure to enable Tekapo B Power Station to be operated for generation purposes before the Pukaki Dam was completed and the level of Lake Pukaki raised to its operating level range.

The structure forms part of the Tekapo Scheme including Tekapo B Power Station now owned and operated by Genesis Energy (Genesis).

Meridian Energy have applied for fast-track consent approval to draw the level of Lake Pukaki down to below RL 518 m. This would require the Tekapo B Power Station tailrace weir and rock chute to become operational again down to an estimated minimum level of about RL 514 m, albeit for a relatively short period of time (Damwatch, 2025a).

Due to the prospective future use of the Tekapo B Power Station tailrace weir and rock chute structure, Meridian Energy (Meridian) have commissioned Damwatch Engineering (Damwatch) to carry out a condition assessment of the structure to complement the recent hydraulic review (Damwatch, 2025a) (Damwatch, 2025b).

3 Reference Material for Condition Assessment and Bathymetric Survey Review

To facilitate the condition assessment, Meridian commissioned a bathymetric survey and a dive inspection of the Tekapo B Power Station tailrace weir and rock chute structure. These are documented in the following reports and associated imaging:

- Southern Hydrographic Survey Summary Report (Southern Hydrographic, 2026)
- Brief Report – Tekapo B Spillway 3 February 2026 (Divepro, 2026)

The above reports are incorporated as Appendices A and B to this technical memorandum.

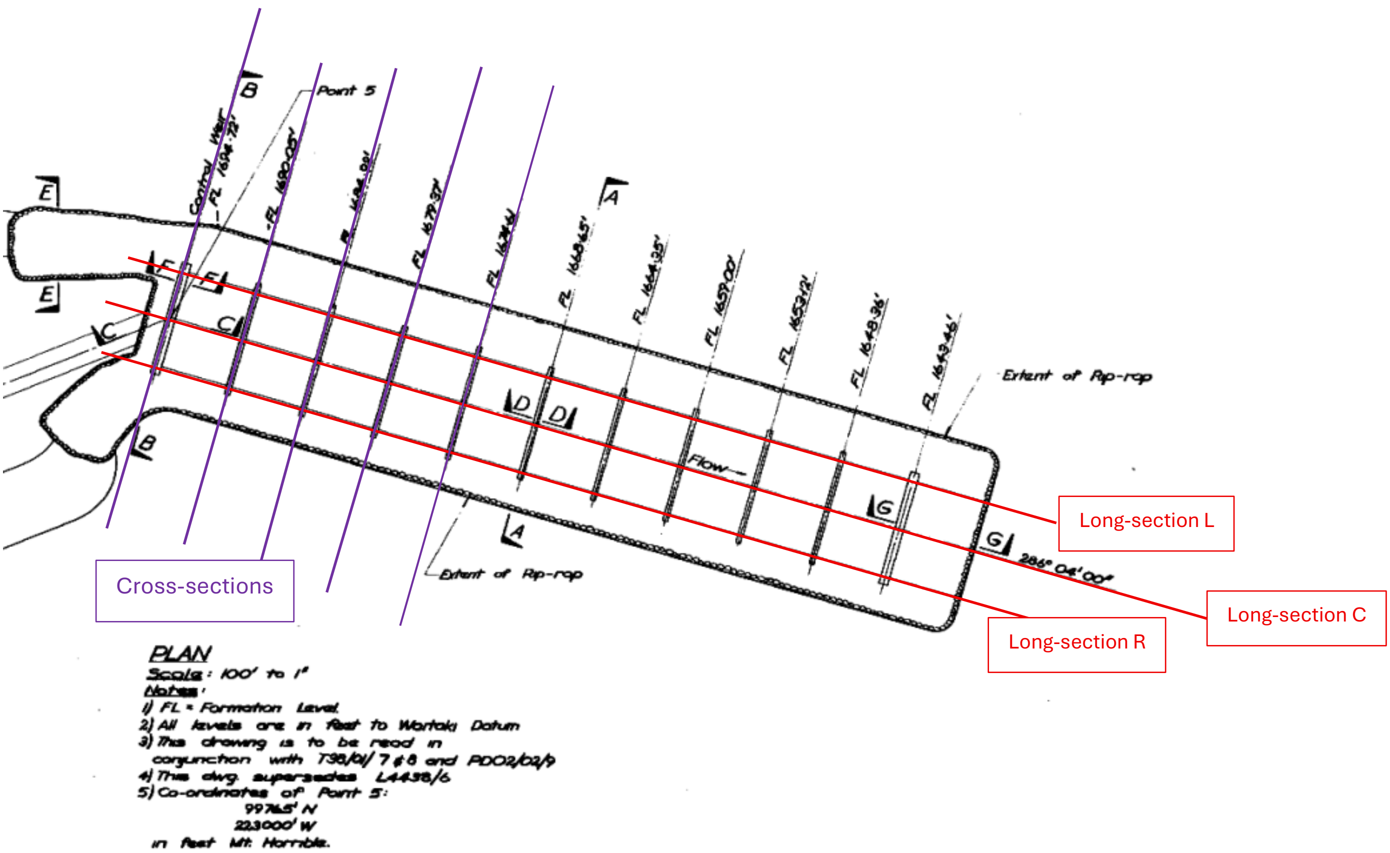
Historical documentation was also available to assist the condition assessment of the structure. This included the following:

- As-built drawings (these are included in Appendix A of Damwatch (2025a)).
- Original design report for the structure (MWD, 1979)
- Construction specification HD1147 for the structure (MWD, undated-a)
- Operating Instruction No. 307 for the structure (MWD, undated-b)
- Section 15 Temporary Tailrace of Tekapo B Powerhouse Construction Report (MWD, undated-c)
- 1979 *Proceedings of Institution of Civil Engineers* UK paper "Construction and Performance of the Rock-Lined Chute at Tekapo B Power Station" (Malan & Hancock, 1979)
- 1979 *Journal of Hydraulic Research* paper "Hydraulics of a Large Channel Paved with Boulders" (Thompson & Campbell, 1979)

4 Review of Bathymetric Survey

4.1 Presentation of Bathymetric Survey Data

Figure 1 shows a copy of the plan view of the structure from as-built drawing T38/01/6.



Source: MOW drawing T38/01/6

Figure 1: Plan view of Tekapo B Power Station tailrace weir and chute

The bathymetric survey data were used to develop the following CAD drawing outputs with reference to Figure 1:

- A plan view of the whole structure from the bathymetric survey (TEK-B\TW\CHT\OVERVIEW)
- A plan view of the structure with images of bay 1 between sills 1 & 2 from the bathymetric survey (TEK-B\TW\CHT\BAY-1)
- A plan view of the structure with images of bay 2 between sills 2 & 3 from the bathymetric survey (TEK-B\TW\CHT\BAY-2)
- A plan view of the structure with images of bay 3 between sills 3 & 4 from the bathymetric survey (TEK-B\TW\CHT\BAY-3)
- A plan view of the structure with images of bay 4 between sills 4 & 5 from the bathymetric survey (TEK-B\TW\CHT\BAY-4)
- Long-sections along the left side, centreline and right side of the structure at the invert of trapezoidal shaped chute (TEK-B\TW\CHT\PROFILE SECTION)
- Cross-sections of the structure at the centreline of sills 1, 2 and 3 with as-built levels marked (TEK-B\TW\CHT\CROSS-SECTION)
- Cross-sections of the structure at the centreline of sills 4 and 5 with as-built levels marked (TEK-B\TW\CHT\CROSS-SECTION)

These drawing outputs use the Lyttelton local mean sea level vertical datum 1937 (LYTTHT1937)[1] to define levels structure and the New Zealand Transverse Mercator 2000 projection (NZTM2000)[2] to define the horizontal position of points on the structure.

The drawings are included in Appendix C of this memorandum.

The drawings are focussed on the top part of the tailrace weir and chute structure which would be exposed if the level of Lake Pukaki was drawn down into the contingent storage range (RL 518 - 513m).

Note that Sill 1 refers to the control weir at the head of the structure in Figure 1.

4.2 Interpretation of Bathymetric Survey Data

4.2.1 Comparison of Surveyed Sill Levels with As-Built Sill Levels

The bathymetric survey data was used to compare current sill levels on the tailrace weir and chute structure with as-built levels marked on the original construction drawings. The purpose of this comparison was to detect any evidence of significant movement of the structure vertically.

Table 1 presents a comparison of surveyed and as-built sill levels at the left side, centreline and right side of the invert on each sill on the structure. Although it is difficult to precisely locate points on the left and right sides of the invert of each sill in the bathymetric survey to read off survey levels, the comparison with the as-built sill levels in Table 1 indicates that:

- 55% of the differences between the bathymetric survey and as-built levels are <0.1 m
- 73% of the differences between the bathymetric survey and as-built levels are <0.15 m
- at least 91% of the differences between the bathymetric survey and as-built levels are <0.2 m
- nearly 100% of the differences between the bathymetric survey and as-built levels are <0.25 m

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

[2] https://www.linz.govt.nz/guidance/geodetic-system/coordinate-systems-used-new-zealand/projections/new-zealand-transverse-mercator-2000-nztm2000

These invert level differences suggest there has been no significant settlement of the tailrace weir and chute structure.

Table 1: Comparison of surveyed and as-built invert levels on sills of Tekapo B Power Station tailrace weir and chute

Sill Number	Distance from Centre-line on Sill 1	Invert Level – Left Side		Differ-ence	Invert Level – Centreline		Differ-ence	Invert Level – Right Side		Differ-ence
		Survey	As-Built		Survey	As-Built		Survey	As-Built	
	(m)	(RL m)	(RL m)	(m)	(RL m)	(RL m)	(m)	(RL m)	(RL m)	(m)
1 (weir crest)	0.00	517.86	517.85	0.01	517.81	517.85	-0.04	517.75	517.85	-0.10
2	30.48	516.41	516.43	-0.02	516.36	516.43	-0.07	516.42	516.43	-0.01
3	60.96	514.93	514.87	0.06	514.97	514.87	0.10	514.81	514.87	-0.06
4	91.44	513.27	513.14	0.13	513.21	513.14	0.07	513.15	513.14	0.01
5	121.92	511.78	511.72	0.06	511.85	511.72	0.13	511.87	511.72	0.15
6	152.40	510.05	509.91	0.14	510.05	509.91	0.14	510.04	509.91	0.13
7	182.88	508.81	508.60	0.21	508.83	508.60	0.23	508.82	508.60	0.22
8	213.36	507.04	506.96	0.08	507.12	506.96	0.16	507.22	506.96	0.26
9	253.84	505.33	505.17	0.16	505.44	505.17	0.27	505.42	505.17	0.25
10	274.32	503.85	503.90	-0.05	503.88	503.90	-0.02	503.85	503.90	-0.05
11	304.80	502.49	502.23	0.26	502.41	502.23	0.18	502.32	502.23	0.09

This conclusion is supported by the evidence from the dive inspection (refer to the report in Appendix B).

There is no indication from the dive inspection that there has been any significant settlement or lateral displacement of the upper part of the structure since construction. 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). However, none of the concrete sills inspected exhibit any signs that significant settlement has occurred since initial operation of the structure after construction.

The timber staff gauges which were installed on the upward sloping ends of the concrete sills to facilitate measurement of water levels and depths during original operation of the structure (Thompson & Campbell, 1979) show some minor damage. However, the diver noted that the paint markers and numbers on one gauge board which had become detached[3] "*appeared as new*". This lends further support to the conclusion that there has been no significant settlement or lateral displacement of the structure since original construction.

[3] The detachment of this particular gauge board would have been due to normal wear and tear arising from continuous operation of the tailrace weir and chute structure over several months before the level of Lake Pukaki was raised to its new operating range.

4.2.2 Observations from Survey Imagery

There are several observations which can be made from the bathymetric survey imagery (drawings TEK-B\TW\CHT\BAY-1 to Bay-4):

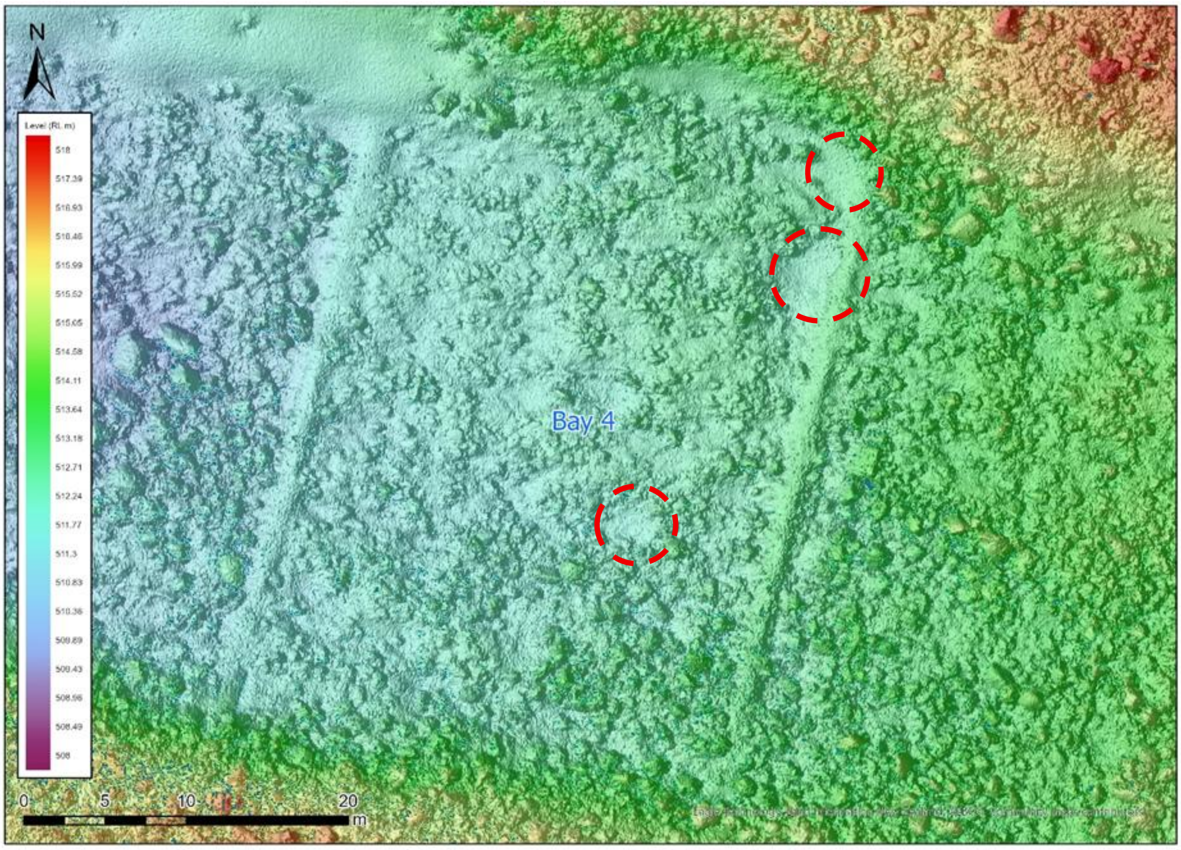
- The concrete sills on the tailrace weir and chute structure can be clearly delineated from the riprap material in the bays between adjacent sills.
- The individual riprap stones in each bay between sills can also be clearly delineated.
- The riprap material appears to be very large in bay 1 (between sills 1 and 2) with the largest material downstream of sill 1 and the smallest material upstream of sill 2. The size of the material is consistent with the gradation curve given in Thompson & Campbell (1979) and summarised in Damwatch (2025a).
- The riprap size also appears to progressively get smaller in size from bay 1 to bay 2, from bay 2 to bay 3 and from bay 3 to bay 4. Again, this is consistent with the gradation curves given in Thompson & Campbell (1979) and summarised in Damwatch (2025a).
- Within bay 2, the riprap material appears to be slightly coarser on the right side compared to the left side. The gradation curve for the material in this bay given in Thompson & Campbell (1979) would have been represented an average grading across all material based on the method of measurement which was used to determine it.
- Within bay 3, there appear to be areas of large riprap material (as large as the material in bay 2) interspersed with patches of smaller material.
- Within bay 4, larger riprap material continues to be present in clusters with smaller material filling the gaps[4]. There also appear to be some unknown ‘smooth’ patches (see red dashed circles in the image in Figure 2 below).

To assist with these observations of the current condition of the rock lining on the structure, it is useful to correlate the boulder sizes in each bay measured by Thompson and Campbell (1979) (and summarised in Table 5.1 of Damwatch (2025a)) with the thickness of the rock lining shown on the as-built drawings for the structure included in Appendix A of Damwatch (2025a):

- In bay 1, the median boulder size was d_{50} = 0.85 m with d_{15} = 0.23 m and d_{85} = 1.44 m compared to a nominal layer thickness of 1.52 m (5 feet) downstream of the control weir (sill 1) and 0.76 m (2.5 feet) at sill 2. The paving layer thickness is therefore ≈ $1.8d_{50}$ at sill 1 (effectively one to two stones thick) and ≈ $0.9d_{50}$ at sill 2 (effectively one stone thick with the top of the boulders likely projecting slightly above the top surface of sill 2 as seen in Figure 3.3 of Damwatch (2025a)).
- In bay 2, the median boulder size was d_{50} = 0.60 m with d_{15} = 0.16 m and d_{85} = 1.15 m compared to a nominal layer thickness of 0.76 m (2.5 feet). The paving layer thickness is therefore ≈ $1.3d_{50}$ throughout the bay (effectively one stone thick).
- In bay 3, the median boulder size was d_{50} = 0.48 m with d_{15} = 0.18 m and d_{85} = 0.93 m compared to a nominal layer thickness of 0.76 m (2.5 feet). The paving layer thickness is therefore ≈ $1.6d_{50}$ throughout the bay (effectively one to one and a half stones thick).
- In bay 4, the median boulder size was d_{50} = 0.30 m with d_{15} = 0.12 m and d_{85} = 0.48 m compared to a nominal layer thickness of 0.76 m (2.5 feet). The paving layer thickness is therefore ≈ $2.5d_{50}$ throughout the bay (effectively two and a half stones thick).

[4] It should be noted that Damwatch (2025b) concluded that, on the basis of the information available, the rock riprap lining material in bays 1-4 will remain stable under the highly aerated ramp-type flow conditions which develop if the level of Lake Pukaki is drawn down to RL 513 m and Tekapo B Power Station discharges up to 115 m^3/s.

- In bay 5, the grading of the riprap lining material was similar to that in bay 4 so the paving layer thickness will also be similar.



Source: Bathymetric survey data accompanying Southern Hydrographic (2026)

Figure 2: Image of bay 4 between sills 4 and 5 on from bathymetric survey of Tekapo B Power Station tailrace weir and chute

Section 3.8.4 of Damwatch (2025a) summarised observations by Thompson and Campbell (1979) on the appearance of the rock lining after one month's operation of the structure from a comparison of before and after vertical photographs. There was no evidence of movement in bay 1 where the boulders forming the lining were very large (d_{50} = 0.85 m). In bays 2-7 (between sills 2 and 8) which were lined with boulders from 0.2-0.6 m in size, it was inferred that the original paving had been modified by the flow to form a tight and stable mosaic pattern (like the armoured mosaic surface in a gravel-bed river). The downstream displacement of paving elements to form this armoured mosaic surface resulted in a depressed surface in the lee of each concrete sill. A large scour hole had formed underneath sill 4 which required repair.

There is no evidence from the recent bathymetric survey that there has been any marked deterioration in the paved lining of the rock chute since Thompson and Campbell (1979) made these observations.

Global instability of the rock chute (including the paved riprap lining) as a potential failure mode is discussed in Section 5.

5 Global Instability as a Potential Failure Mode for Rock Chute

5.1 Overview

In this section, global instability as a potential failure mode for the rock chute part the structure is examined in response to comments made to Meridian by Genesis Energy's Engineering Manager (Civil and Dam Safety). Global instability of the paved riprap lining of the rock chute could lead to unravelling of the whole structure and undermining of the concrete weir sill at the head of the structure.

The possibility of this potential failure mode occurring is examined by:

- carrying out a selected literature review of rock ramps in river situations;
- reviewing the past performance of the weir and chute structure in 1978-1979;
- reviewing the scour incidents during previous operation in 1978-1979, when they occurred, where they occurred and explanations of why they occurred;
- reviewing the likely conditions of future operation based on evidence provided by Meridian in their fast-track application for consent to draw the level of Lake Pukaki down below RL 518 m; and
- making an assessment based on the information available of the likelihood of this potential failure mode occurring in the future if the tailrace weir and chute does come into

5.2 Literature Review

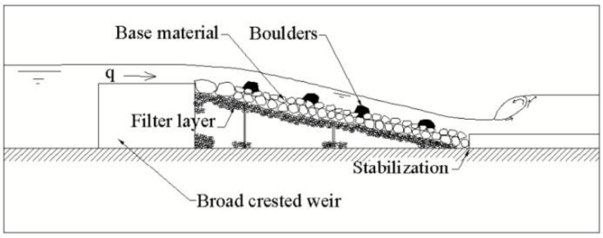
Pagliara and Chiavaccini (2007) investigated the failure mechanisms of base and reinforced block ramps with laboratory flume experiments. These types of structures are used internationally in rivers and streams as grade stabilisation and energy dissipation structures. They are relatively short and locally steep longitudinal structures in which the stream or riverbed is reinforced with large rocks.

Earlier experimental investigations by Pagliara and Chiavaccini (2004) of the stability of rock ramps reinforced with larger boulders in rows, randomly placed or in an arc configuration showed that ramp failure occurs in three phases:

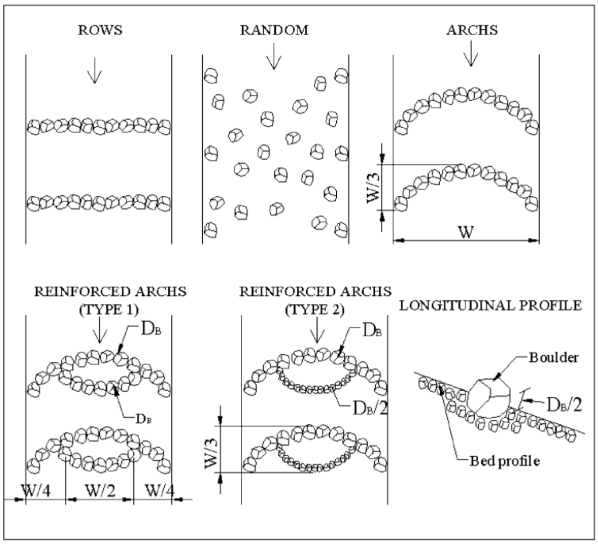
- “initial movement” of the base material forming the ramp in which stones begin to wobble and displacement of some isolated stones occurs (step 1);
- “local failure” in which one or more stones leave their position simultaneously, leaving a well-defined circular or semi-circular scour hole (step 2); and
- “global failure” of the ramp in which many local failures occur leading to the development of large longitudinal holes, especially in the downstream part (step 3).

Pagliara and Chiavaccini (2007) carried out further experimental investigations of the morphological evolution of block ramp failures for ramps with different bed slopes (1 in 4, 1 in 8 and 1 in 12) and reinforcement configurations. Figure 3(a) shows a schematic drawing of the experimental setup while Figure 3(b) shows the different reinforcement configurations using larger boulders than the base material used to form the ramps. The model block ramp setup was stabilised at the toe to prevent the formation of a scour hole in the channel bed which would undermine the toe and cause an unravelling failure of the ramp to develop.

The reinforcement configurations of more relevance to the tailrace weir and rock chute are the boulder “row” and “random placement” configurations. Figures 4(a) and (b) illustrate the observed failure mechanisms for these reinforcement configurations.



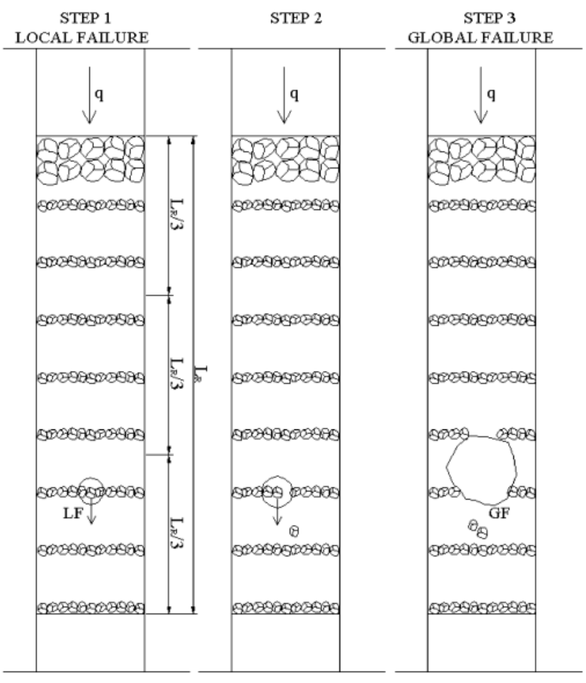
(a)



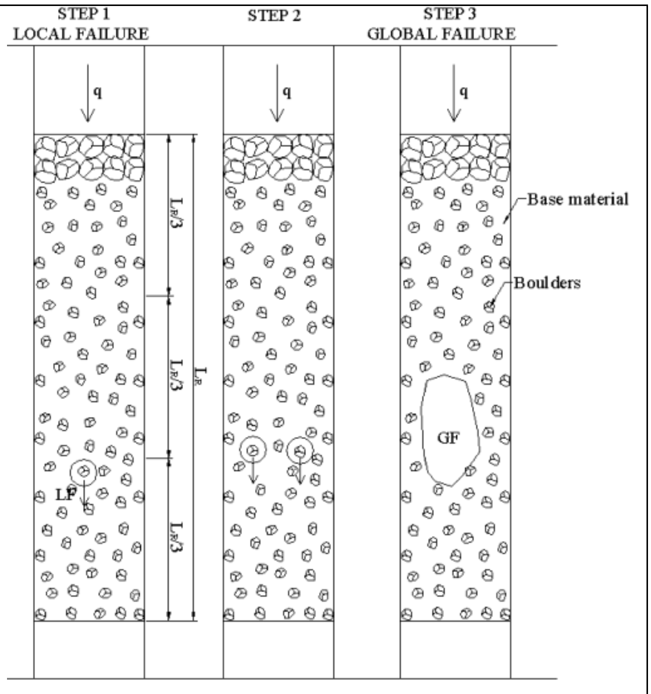
(b)

Source: Pagliara and Chiavaccini (2007)

Figure 3: Experimental setup for investigation of block ramp failures: (a) schematic drawing of block ramp model and (b) alternative boulder reinforcement arrangements



(a) Row arrangement when Db/D50 between 3.5 and 5



(b)

Source: Pagliara and Chiavaccini (2007)

Figure 4: Failure mechanisms for (a) boulder row and (b) random boulder placement configurations in rock ramps

While there are some similarities between the model block ramps tested by Pagliara and Chiavaccini (2007) and the Tekapo B Power Station tailrace weir and rock chute, there are some significant differences:

- The model rock ramps were much steeper. The flattest block ramp slope tested was 1 in 12 compared to the nominal 1 in 20 slope of the Tekapo B tailrace weir and rock chute structure.
- The reinforcement of the Tekapo B tailrace weir and rock chute structure consists of concrete sills compared to the rows of large boulders in the row arrangement for the model block ramps.
- In the model block ramps, local failures of the large reinforcing boulder rows occurred (see Figure 4(a)) whereas such failures cannot occur with the concrete sills on the Tekapo B tailrace weir and rock chute structure.
- In the model block ramps, the local failures of the large reinforcing boulder rows lead to a global failure of the base material between the rows (see Figure 4(a)). If the reinforcing boulder size was small enough, the global failure in the model block ramps was observed to extend longitudinally across several rows of reinforcement boulders.
- On the Tekapo B tailrace weir and rock chute structure, a local failure only of the riprap paving layer between the concrete sills could develop if the threshold flow for riprap stability was exceeded. If the excessive discharge occurred for a long duration, the local failure could develop into a bigger scour hole as in the global failure pattern for a random boulder reinforcement arrangement for the model block ramps (see Figure 4(b)). However, any global failure would be confined by the concrete sills.

In summary then, the concrete sills on the Tekapo B tailrace weir and rock chute structure function as reinforcement features and limit the progression of any global failure of the riprap paving liner between the sills if the threshold flow for riprap instability was exceeded.

5.3 Past Performance of Structure

Figure 5 shows the outflow from Tekapo B Power Station and the level of Lake Pukaki during the period from June 1977 to November 1978 when the temporary tailrace weir and rock chute structure functioned as an energy dissipation facility.

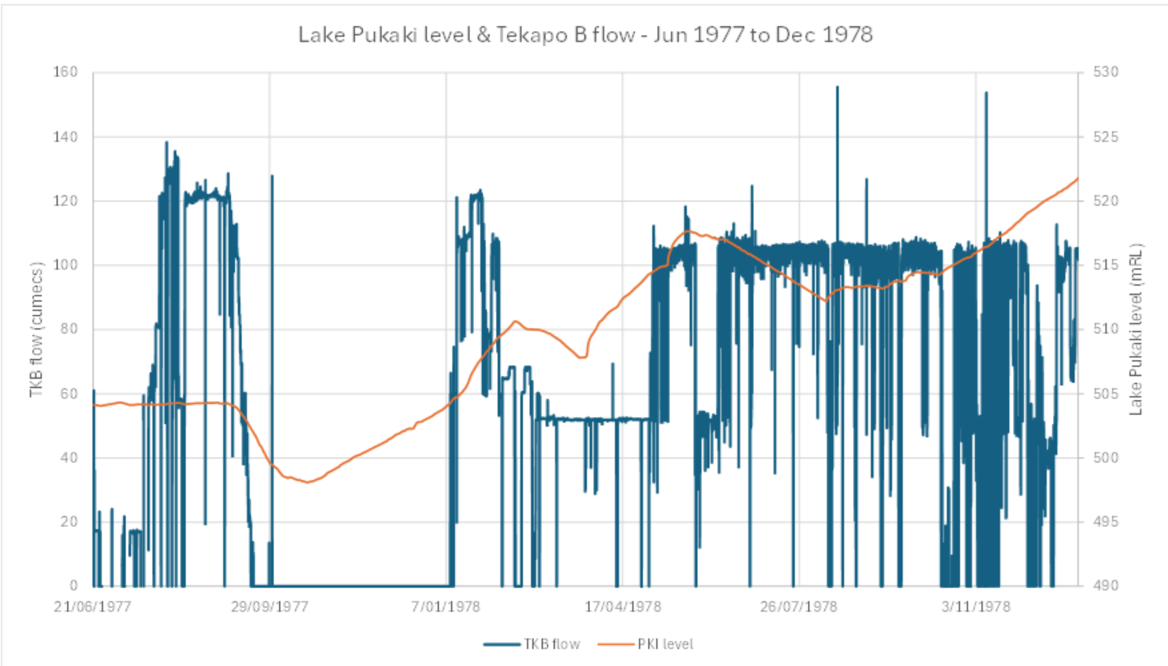


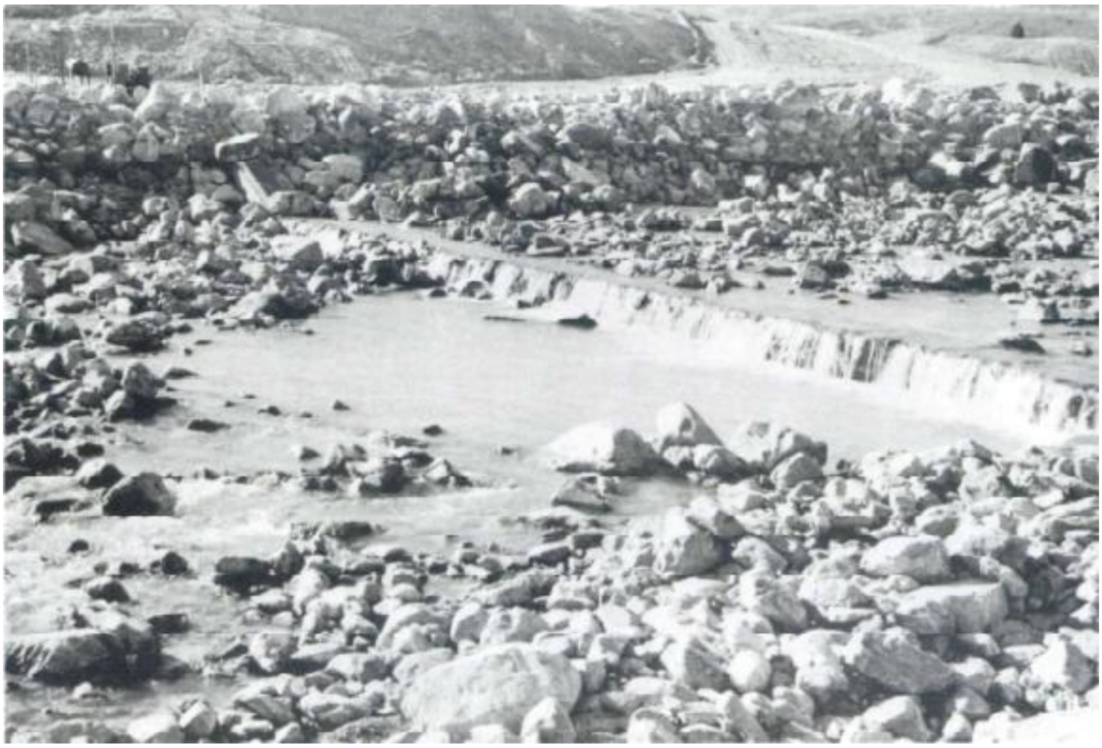
Figure 5: Tekapo B Power Station outflow and Lake Pukaki level from June 1977 to Dec 1978

Malan and Hancock (1979) describe the following sequence of flows for the commissioning of the structure up to the full rated discharge of 122 m^3/s

- Run 1 on 19 June 1977 – 4 hours operation at 30.6 m^3/s discharge (25% load at Tekapo B Power Station)
- Run 2 on 19 July 1977 – 3.5 hours operation at 60.9 m^3/s discharge (50% load at Tekapo B Power Station)
- Run 3 on 26 July 1977 – 72 hours operation at 81.3 m^3/s discharge (67% load at Tekapo B Power Station
- Run 4 on 29 July 1977 – continuous operation at 122 m^3/s discharge (100% load at Tekapo B Power Station

After 72 hours at full load, a semi-circular scour hole measuring about 14 m diameter and 3 m deep developed downstream of sill 5 (Malan & Hancock, 1979). This hole is shown in the photo in Figure 6. The hole remained stable in size for the next two months of the winter generation season (generation ceased in mid-September 1977 and did not resume until early January 1978). The scour hole was repaired after the winter generation period (MWD, undated-c).

After 12 days at full load with peak flows up to 140 m^3/s, another major scour hole measuring 24 m in diameter and nearly 5 m deep was detected immediately downstream of and underneath sill 9 during a routine inspection (Malan & Hancock, 1979). This is shown in the photo in Figure 7. A flow through Tekapo B Power Station of 60.9 m^3/s (corresponding to 50% maximum station load) was allowed overnight after the scour hole was discovered. The scour hole was repaired the next day with 1650 m^3 of gravel fill material and 665 m^3 of riprap material. Care was exercised to achieve good penetration of the gravel fill material underneath the concrete sill. The repair performed very satisfactorily afterwards.



Source: MWD (undated -c)

Figure 6: Scour hole downstream of sill 5 which developed after 72 hours of tailrace weir and rock chute operation in early August 1977

Source: MWD (undated -c)

Figure 7: Scour hole downstream of sill 9 which developed after 12 days of tailrace weir and rock chute operation in mid-August 1977

The two scour holes which formed during early operation of the temporary tailrace weir and rock chute structure were similar to the local failures on model block ramps described by Pagliara and Chiavaccini (2007). The first scour hole at sill 5 did not develop beyond the initial size while the second larger one at sill 9 was fixed before it progressed to a more global failure in bay 9 (between sills 9 and 10).

After the two scour hole repairs were completed, the temporary tailrace weir and rock chute structure continued to function as an energy dissipation structure from January to November 1978 until the level of Lake Pukaki exceeded RL 518 m and fully drowned the structure (see Figure 5). Over this 11-month period, the tailrace weir and rock chute structure showed no further evidence of any local or global failures of the riprap lining between bays. The operation instructions for the structure (MWD, undated-b) required flow over the control weir and down the rock chute to be observed at least three times daily when operating for any "*erosion, unusual turbulence, unsymmetrical flow, bypassing of the formed channel, or any other abnormal condition*". The operation of the structure while its was functioning as an energy dissipation facility was therefore rigorously observed.

5.4 Likely Conditions of Future Operation

Section 3.1 of Meridian's substantive application for fast-track consent describes the modelling undertaken to assess the likely frequency of the level of Lake Pukaki falling below RL 518 m over the next three years (2026-2028). If the level of Lake Pukaki does fall below RL 518 m, then the original Tekapo B Power Station temporary tailrace control weir and rock chute would be reactivated. However Tekapo B Power Station would still be able to operate as the control weir on the original temporary structure would maintain the tailwater level in the normal tailrace channel above RL 518 m.

Of the 91 annual inflow sequences modelled for the year 2026 with the proposed alternative stored energy management regime applied, 21 of the sequences (23%) have the level of Lake Pukaki falling below RL 518 m. Of these 21 sequences:

- 9 fall between RL 518 m and RL 517 m
- 6 fall between RL 517 m and 516.5 m

- 3 fall between RL 516.5 m and RL 516 m
- 2 fall between RL 516 m and RL 515 m
- 1 falls below RL 515 m

In the latter worst-case scenario, the level of Lake Pukaki was simulated to fall below RL 518 m in September and not return to this level until December, a period of no more than four months. The likelihood of this scenario occurring is approximately 1%.

Overall in 2026, the probability of the lake level being below RL 518 m in any given week is predicted to be about 3%. On average, based on the 91 years of modelled inflow sequences, the level of Lake Pukaki will only be below RL 518 m for 1.5 weeks.

For the two subsequent years of eased operation for Lake Pukaki (2027 and 2028), the potential frequency of lake levels dipping below RL 518 m is predicted to be broadly similar to the predicted pattern described above for the first year. However, the probability of the lake level being below RL 518 m in any given week is predicted to increase slightly to 3.5% in 2027 and 4% in 2028.

5.5 Assessment of Likelihood of Failure Mode Developing

The concrete sills on the tailrace weir and rock chute structure are intended as a reinforcement measure to prevent large-scale movement of the riprap lining between sills and exposure. The riprap lining would have developed an 'armoured surface' from early operation of the structure (involving some rearrangement of the individual boulders forming the lining surface) similar to that which occurs in a gravel-bed river from water flows. Apart from the early scour failures at sills 5 and 9 which were successfully repaired, the structure operated satisfactorily for 11 months from January to November 1978 without requiring any intervention to further remediate the rock lining. Over this period of operation, flows of up to 120 m^3/s (with occasional pulses of more than 140 m^3/s) were discharged through the structure into Lake Pukaki (see Figure 5). There is no evidence from either the bathymetric survey or the dive inspection that the rock lining has experienced any significant deterioration from being submerged for more than 47 years.

Based on these considerations, the likelihood of global instability of the riprap lining occurring is extremely low as long as the flow threshold for riprap stability is not exceeded. From Damwatch (2025b), this flow threshold is assessed to be larger than 115 m^3/s.

Before any global instability of the riprap lining on the structure occurred, a local failure of the lining would become evident. This highlights the importance of continual observational monitoring of the structure during future operations as was done in 1977-1978 before the structure was submerged by Lake Pukaki. However, the risk of a local failure developing is very low given the past performance of the structure from January to November 1978 without any failure being detected.

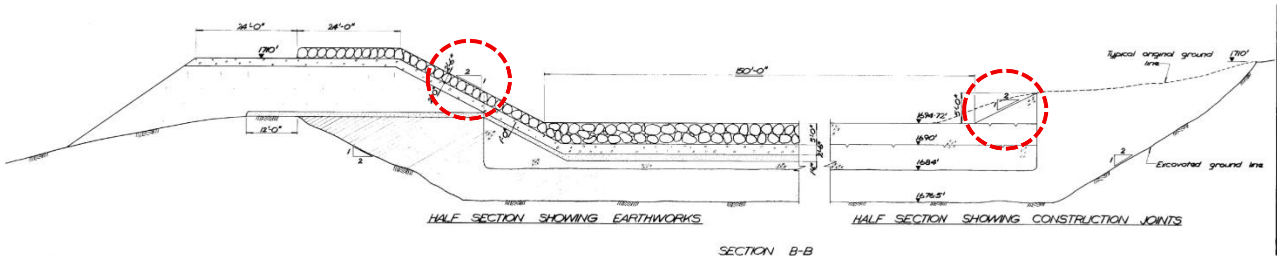
6 Condition Assessment

6.1 Concrete Structural Elements

6.1.1 Design and Construction

The riprap lined channel was designed and constructed with a concrete weir at the upstream end to serve as the main control structure, and additional sills (also called "ribs" in some documentation) spaced at 30m (typical) along the length of the channel to control scour and provide containment of riprap lining. As the channel bed has a relatively steep slope, the concrete sills also serve to prevent a "*washout*" or "*rolling away*" of the (rounded) boulders used as riprap. This is common industry practice for riprap lining of steep open channels, revetments, or similar hydraulic structures.

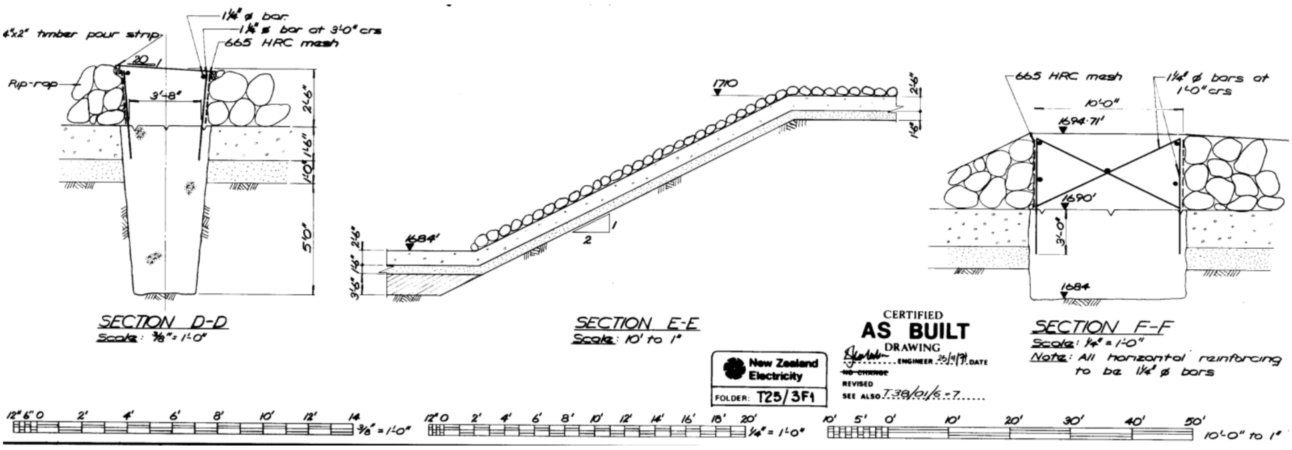
The design envisaged that the concrete elements would be constructed first, and earthworks would be carried out later, according to the grades and channel profile set out by the concrete elements. This purpose is evidenced by the triangular projections on the left and right ends of the concrete weir, for alignment of earthworks to the channel's side face profile of 1:2 vertical: horizontal (circled in Figure).



Source: MOW drawing T38/01/7

Figure 8: Cross-section of the upstream control weir

During construction, however, constraints to construction duration and methodology necessitated modification in the sequence of construction activities. The earthworks were placed first, and concrete weirs constructed later after excavation of trenches and placement of riprap. Concrete was placed in two stages – the lower lift placed in the excavated trench after completion of earthworks for the channel bed, and the upper lift placed against steel forms at some ribs and against riprap in others. To prevent interference of (already placed) riprap with concrete placement, reinforcing bars and steel mesh (665 HRC[5]) were provided on the upstream and downstream faces of the second lift (Figure). The steel mesh served as sacrificial "formwork" for the second lift of concrete. It does not appear to have been provided as a design measure for crack control, or to provide strength of the concrete. However, encapsulation of the steel mesh with concrete (during placement of the second lift) has enabled the steel mesh to provide the function of crack control (despite no such intention).



Source: MOW drawing T38/01/8

Figure 9: Details of reinforcing steel and mesh used in second concrete lifts on control weir and sills

There is no information from the dive inspection to confirm the current condition of the lift joints, nor is there any specification for placement of concrete works. However, if standard Ministry of Works' specifications for concrete works were followed for the works, then the lift joints are likely to have been cleaned and roughened before placement of second concrete lift. Typically the construction joint would be scabbled and the surface wetted before the next layer of concrete was placed, so the

[5] Hot Rolled Coil (HRC)

second lift of concrete would normally be assumed to have good bond with the first lift, and to have developed adequate tensile and shear strength of the joint.

The altered sequence of construction activities resulted in some surficial imperfections in the shape and surface finish of the concrete weirs. Based on the observed performance of the tailrace channel (refer Section 6.1.2), these imperfections do not appear to have compromised the intended function or performance requirements for the weirs.

The steel reinforcement bars acting as dowels (1¼” dia. bars at 1’0” centres for the control weir and 3’0” centres for the sills) between the two stages of concrete provide structural continuity across the joint (refer Figure and Figure). In the absence of this continuity (e.g. a complete loss of dowel strength and bond between the two stages of concrete), the second lift would be expected to behave as an independent “rigid body” subject to hydraulic actions from flow and boulder impact in the longitudinal direction (e.g., riprap displacement under flow).

A vehicle passage left in the middle of the channel of the sills (not the control weir), was later filled with second-stage concrete that would provide the action of a shear key, but only in the transverse direction. The shear strength of the joint in the longitudinal direction would be provided by the combination of steel dowels, joint roughness, and bond between the first and second stages of concrete placement.

Source: Meridian archives

Figure 10: Weir construction – bars spaced at 1’0” centres with no vehicle passage

Fig. 4. Rip-rap being placed by a Caterpillar 992 front end loader against the row of starters embedded in the concrete infill

Source: (Malan & Hancock, 1979)

Figure 11: Construction photo showing dowels projecting from first lift and riprap placement in progress

6.1.2 Performance History

The documented performance of the tailrace channel (MWD, undated-c; Malan & Hancock, 1979; Thompson & Campbell, 1979) shows that the concrete sills (both the control weir and the ribs) have performed their intended function very well, without any settlement, structural distress, or other structural damage. Despite the compressed timeframe for commissioning of the tailrace channel, the concrete weirs did not undergo any settlement, displacement or other concrete damage apart from surficial abrasion under aggressive hydraulic actions. The sills did not permit riprap displacement across them, and riprap movement was limited to minor displacements within the compartment created by any two successive sills.

6.1.3 Structural Condition

An evaluation of the information from recent bathymetric survey and dive inspection (including video footage and transcript) does not reveal any damage to the concrete weirs other than minor abrasion (exposure of coarse aggregate) and exposure of the steel mesh at some locations.

As stated in Section 6.1.1, the reinforcing bars (dowels) and steel mesh were provided to prevent interference in concrete placement 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 refers to the 665 HRC steel mesh used as a sacrificial form for pouring concrete in the second lift and therefore does not present any risk of impairment of intended functionality of the sills.

The concrete sills are stiff, buried structures in a submerged environment and are not subject to differential lateral forces that would exert significant flexural or shear demand along or across the flow direction. Additionally, they do not rely upon lateral restraint from the riprap for stability and hence, separation between concrete and riprap noted in the dive inspection does not present any risks to structural integrity, stability, or strength. Despite the formation of a large scour hole at sill 9 during early operation of the tailrace weir and rock chute structure (as described in Section 5.3 and shown in Figure 7) and the consequential partial loss of lateral restraint and foundation support, a failure of the sill did not occur. This provides confidence in the reserve strength of the weirs, should loss of lateral restraint from the ground occur, for any reason.

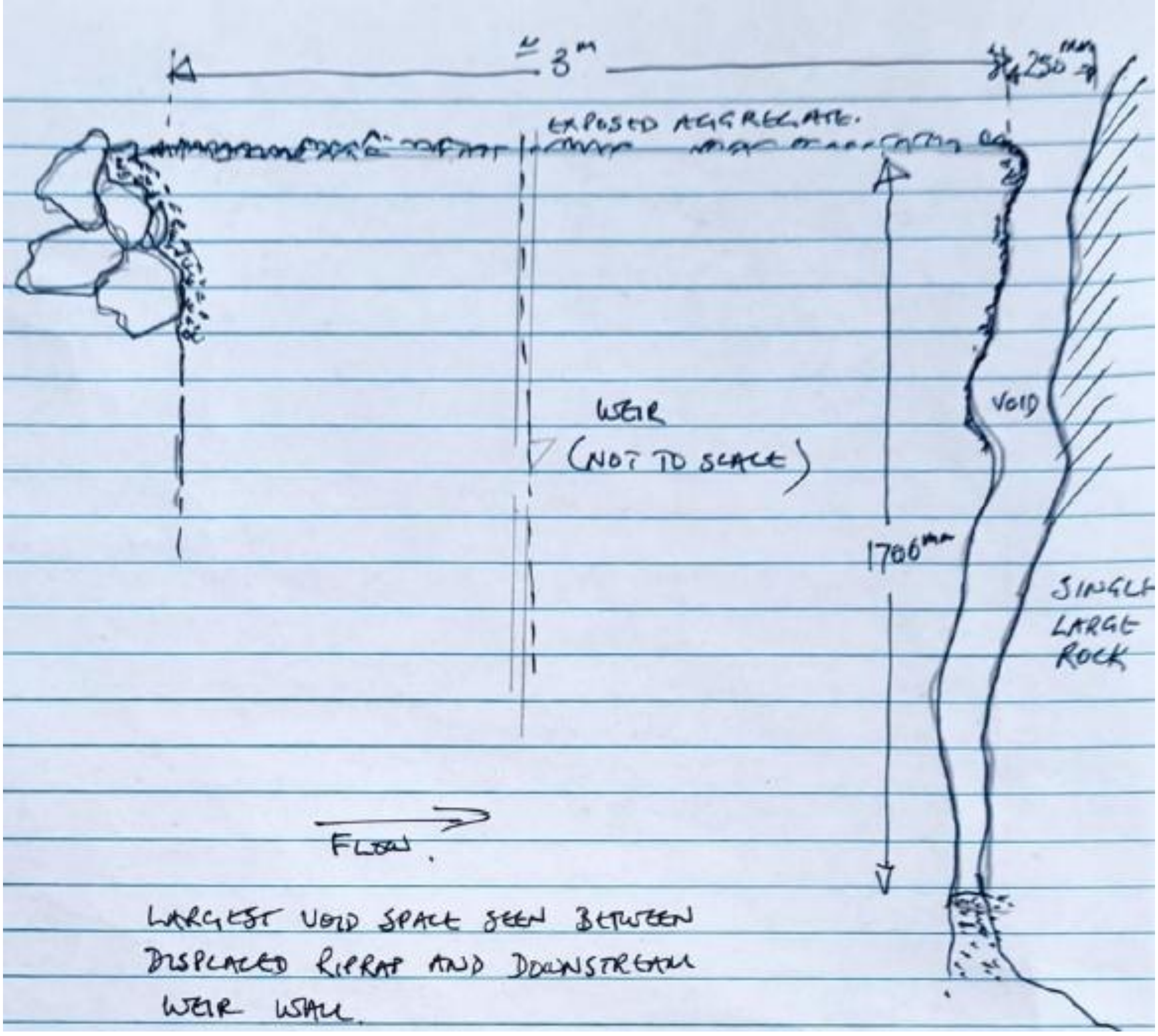
Their performance to date does not indicate any exceedance of material strength of the concrete that would have resulted in failure of the sills – i.e., the sills are still “intact” and not broken. Having been in a submerged environment for many years, the concrete is expected to have undergone long-term hydration and the current compressive strength of the concrete is likely to be higher than the specified 28-day compressive strength (Wood, 1992).

Role of Dowel Action

There is currently not enough information on the condition of the steel reinforcing bars that act as dowels between the two stages of concrete placement. The information from the bathymetric survey and dive inspection is limited in the sense that the integrity of the lift joint has not been described. If the dowels are in a good condition and can provide enough strength to the lift joint to withstand hydraulic actions exerted by the flow regime, and potential “boulder impact”, the control weir and sills are expected to provide the same functionality as they provided before submergence of the channel.

The embedded 1¼-inch diameter dowel bars are expected to continue to contribute to the strength of the lift joint, and it is highly unlikely that all the dowel bars (~180 bars each on the upstream and downstream faces) would have deteriorated completely so that they cannot be relied upon for the shear strength of the joint. In the sketch from the dive inspection (Figure 12), no exposed dowel bars or deteriorated condition of lift joints were observed where there was opportunity to view the side faces of the sills (in Figure 12 – note that the lift joint is at approximately 1.44 m depth, compared to the diver’s observed depth of sill face approximately 1.7 m deep). The dive inspection did not note any cracking of concrete, deteriorated condition of the lift joint, or other concrete defects/damage that would compromise the strength or integrity of the overall concrete weir and sills.

In the highly unlikely condition that all the dowels at a lift joint have deteriorated in condition to the extent that the second concrete lift is no longer structurally connected to the lower lift, then the shear strength of the lift joint would be dependent entirely on the frictional resistance of the joint, and any bond strength between the two concrete lifts. Under aggressive hydraulic actions or boulder impact from displaced riprap, and insufficient residual shear strength in the lift joint to withstand those actions, the second concrete lift in a weir may be destabilised and become misaligned. In a misaligned position, the displaced “upper” sill may only partially fulfil the function of riprap containment between two successive sills (or ribs).



Source: (Divepro, 2026)
Note: Lift joint is approx. 1,435 mm deep, compared to diver's observed depth of rib face approx. 1,700 mm deep

Figure 12: Diver's sketch of void (gap) noted between concrete control weir and riprap

6.2 Riprap Lining

The long-section profiles of the tailrace weir and chute structure on drawing TEK-B\TW\CHT\PROFILE SECTION sourced from the bathymetric survey data indicate that there has been minimal displacement of the riprap lining. There are no significant deviations from the constant 1 in 20 grade of the structure except for a dip along the right-hand side of bay 4 (refer to long-section C).

From the dive inspection report (Appendix B), on the upper part of the structure:

- There is no evidence of significant movement or erosion of the riprap lining.
- The riprap lining is generally flush or slightly proud of the concrete sills in each bay. This is supported by the evidence of the long-sections in drawing TEK-B\TW\CHT\PROFILE SECTION.
- As noted in Section 4.2.1, there is evidence of some minor longitudinal movement of riprap material away from the downstream edge of some vertical sill faces, likely 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 1977-1978.
- There is no evidence of any significant deterioration of the riprap lining although smaller cobble and gravel materials filling the interstices between large stones may)have been washed out leading to an armoured surface as in an armoured gravel bed river (this would likely have occurred in the first few weeks of operation of the structure. As noted in Section 4.2.2, the riprap material

sizes appear to be consistent with the measured material gradings reported in Thompson and Campbell (1979). The riprap lining performed very well for 11 months in 1978 without requiring any repairs to be made during operation of the tailrace weir and rock chute structure until the structure became submerged by Lake Pukaki.
- The surfaces of the structure are covered only in a thin coating of silt and gauge boards mounted on the sloping edges of concrete sills are showing only minor signs of wear and tear after being submerged for more than 47 years.

7 Overall Assessment of Structure and Suitability for Future Use

7.1 Anticipated Structural Performance

Based on the documented performance of the control weir and sills, and the overall tailrace channel, and the current condition of the concrete elements, the control weir and sills are anticipated to provide the same functionality that was provided by the tailrace channel before submergence. This anticipated performance is expected to be compromised only under the following circumstances, which would require extensive erosion of the channel bed, or a complete loss of structural continuity and significantly reduced shear strength at the concrete lift joints for the weirs/ribs.

- A significant loss of foundation (undermining) occurs for any weir/sill (i.e. a cavity is created under it) and the weir/sill settles into a deformed alignment that results in movement of riprap across the weir/sill.
- A significant loss of lateral support from the ground occurs for any weir/sill (i.e. a cavity is created beside it) and the weir/sill is displaced into a deformed alignment that results in movement of riprap across the weir/sill.
- Loss of shear strength of concrete lift joints results in separation between the two concrete lifts, and a destabilisation/ misalignment of the upper lift. In a misaligned position, the displaced "upper" weir/sill may only provide partial containment of riprap in contact with it.

7.2 Anticipated Hydraulic Performance

The past hydraulic performance of the tailrace weir and chute is fully documented in Damwatch (2025a) and Damwatch (2025b) and expanded on in Section 5 of this technical memorandum. The Damwatch (2025b) report concluded that the rock riprap lining material in bays 1-4 will remain stable:

- Under the highly aerated ramp-type flow conditions which develop if the level of Lake Pukaki is drawn down to RL 513 m and Tekapo B Power Station discharges up to 115 m^3/s.
- Under the impact of plunging type breaking waves generated by north-westerly waves blowing across the surface of Lake Pukaki.

There is no indication from the results of the bathymetric survey and the dive inspection observations that the hydraulic performance of the structure would be any different from the past performance.

Aside from the early scour damage at sills 5 and 9 in August 1977 which was successfully repaired, the performance of the tailrace channel control weir and rock chute over the 11-month period from January to November 1978 without any evidence of local failure of the riprap lining or damage to the concrete sills provides confidence in the structure to function as an energy dissipation facility in the future. The fact that the flow performance of the structure was closely monitored with at least three daily inspections provides added weight to this conclusion.

7.3 Suitability of Structure for Future Use

Based on the above assessments, we consider the existing structure is suitable for future use on a temporary basis if Lake Pukaki is required to be drawn down to provide water for hydro-generation purposes. There is a hypothetical risk that the top half of a sill could dislodge from the embedded bottom portion under hydraulic loading conditions. Although there is insufficient data to comprehensively evaluate this risk, a structural failure of this nature would require complete corrosion of all dowel bars to allow the failure mechanism to develop. Further mitigation of this failure mechanism would be provided by the anticipated well prepared construction joints between the top and bottom components of each sill. The risk of this failure mechanism developing is therefore expected to be minimal.

It would be prudent to apply the guidance provided by the original operation instructions (MWD, undated-b):

- Whenever the structure is in operation, it should be observed at least three times daily for any signs of abnormal flow behaviour (erosion of the riprap lining, unusual turbulence, asymmetric flow conditions, breakout of flow from the chute section etc.) (Paragraph 2).
- Maintain a stockpile of riprap material in close proximity which can be quickly accessed to place in the structure if any erosion of the lining is observed (Paragraph 6).
- Have suitable plant available close at hand to enable the stockpile of riprap material to be rapidly placed to repair any erosion of the lining (Paragraph 5).

It should be noted that the last two actions are intended as a precautionary and reactive measure for implementation during future operation of the tailrace weir and rock chute structure. They are not intended to imply proactive reinstatement of areas of the existing rock lining that are perceived or identified from future inspections to be deficient in providing adequate protection to the underlying materials.

To complement the last two measures, we also recommend appropriate construction planning to reinstate riprap in any areas where the riprap is displaced and the channel bed exposed (i.e., areas where the bed is vulnerable to erosion under free flow conditions) during future operations of the structure. This reinstatement would be carried out when the tailrace channel is no longer submerged, and the channel is accessible to construction equipment for placement of riprap material.

7.4 Recommendation for Further Investigation of Existing Structure

It was noted in Section 4.2.2 that the bathymetric survey showed up some unknown ‘smooth’ patches in areas of the riprap lining of the tailrace weir and rock chute structure. These are most noticeable in bay 4 between sills 4 and 5 as marked in the bathymetric survey image in Figure 2.

It is recommended that a dive inspection be undertaken to identify what these patches are in case they are small localised areas where the rock lining material has moved and left the filter layer underneath partially exposed.

8 References

Damwatch. (2025a). *Tekapo B Power Station Temporary Tailrace Weir and Rock Chute: Hydraulic Review of Weir and Chute.* Report prepared for Meridian Energy by Damwatch Engineering, Issue 3.1, Ref. E2567, 28 October 2025.

Damwatch. (2025b). *Tekapo B PS Temporary Tailrace Weir and Rock Chute: Hydraulic Review of Weir and Chute - Addendum Report.* Technical Memorandum prepared for Meridian Energy by Damwatch Engineering, Ref. E2567, 5 December 2025.

Divepro. (2026). *Brief report - Tekapo B Spillway 3 February 2026.* Report prepared for Meridian Energy by Divepro.

Malan, D. J., & Hancock, B. L. (1979). Construction and Performance of the Rock-Lined Chute at Tekapo B Power Station. *Proc, Instn, Civil Engrs.*, Part 1, Vol, 66, pp 667-677.

MWD. (1979). *Tekapo B Temporary Tailrace Design Report.* Power Divsion, File 92/12/77/42, 29 June 1979.

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MWD. (undated-b). *Tekapo B Power Project: Operating Instruction No. 307 - Flow Control and Naintenance of Tailrace Extension.* Power Division, File 92/12/77/40/2.

MWD. (undated-c). *Tekapo B Power Project: Tekapo B Powerhouse Construction Report.* Ministry of Works and Development.

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Pagliara, S., & Chiavacinni, P. (2004). Stability of reinforced block ramps. *River Flow 2004, Proc. Proceedings 2nd International Conference on Fluvial Hydraulics.* Naples, Italy: Balkema, Rotterdam.

Southern Hydrographic. (2026). *Survey Summary Report - Tekapo B Power Station.* Report prepared for Meridian Energy by Southern Hydrographic, Ref. MER_JN2604_.ROS_v1, 5 February 2026.

Thompson, S. M., & Campbell, P. L. (1979). Hydraulics of a Large Channel Paved with Boulders. *Journal of Hydraulic Research*, Vol. 17:4, 341-354.

Wood, S. L. (1992). *Evaluation of the Long-Term Properties of Concrete.* Skokie, Illinois: Research and Development Bulletin RD102T, Portland Cement Association.

Document history and status

Issue no.	Issue date	Description	Prepared by	Reviewed by	Approved by
1	2026-02-27	Draft technical memorandum	VL, GW & JS	DCE	DCE
2	2026-03-19	Version 2 of technical memorandum	GW & VL	DCE	DCE

Appendix A Bathymetric Survey Report (Southern Hydrographic, 2026)

Date: 05/02/2026 **Survey No:** MER-JN2604 **Ref:** MER_JN2604_ROS_v1	SouthernHydrographic **SURVEY SUMMARY REPORT**	**Client:** Meridian

Port	Survey Dates	Survey Area(s)	Survey Order
LAKE PUKAKI	03 February 2026	• Tekapo B Power Station	**LINZ SO**

Hydrographic Surveyor (Supervising)	Certification
Dave Mundy Southern Hydrographic Ltd PO Box 61 Mapua 7048	IHO Category A / AHSCP Level 1

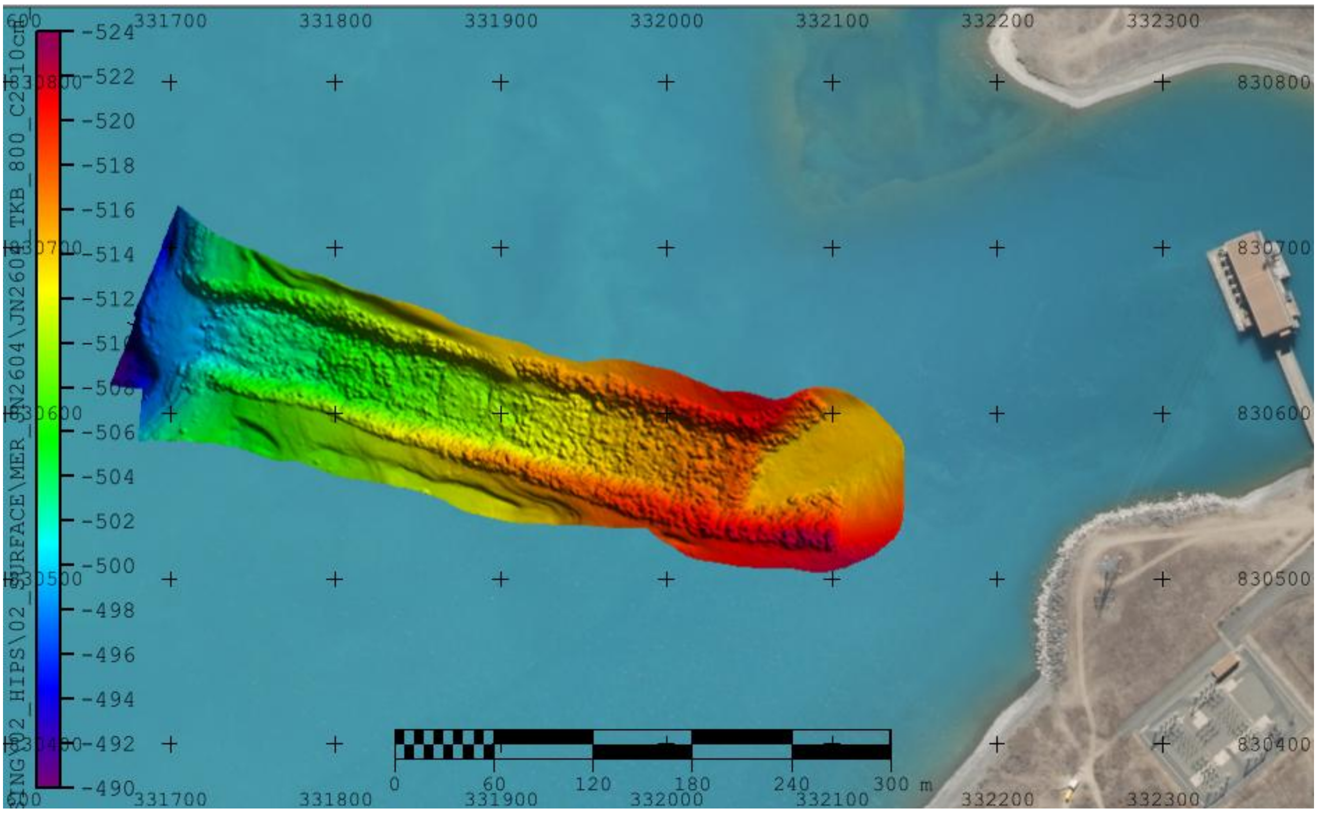
Purpose of Survey

Purpose of Survey	• Monitoring

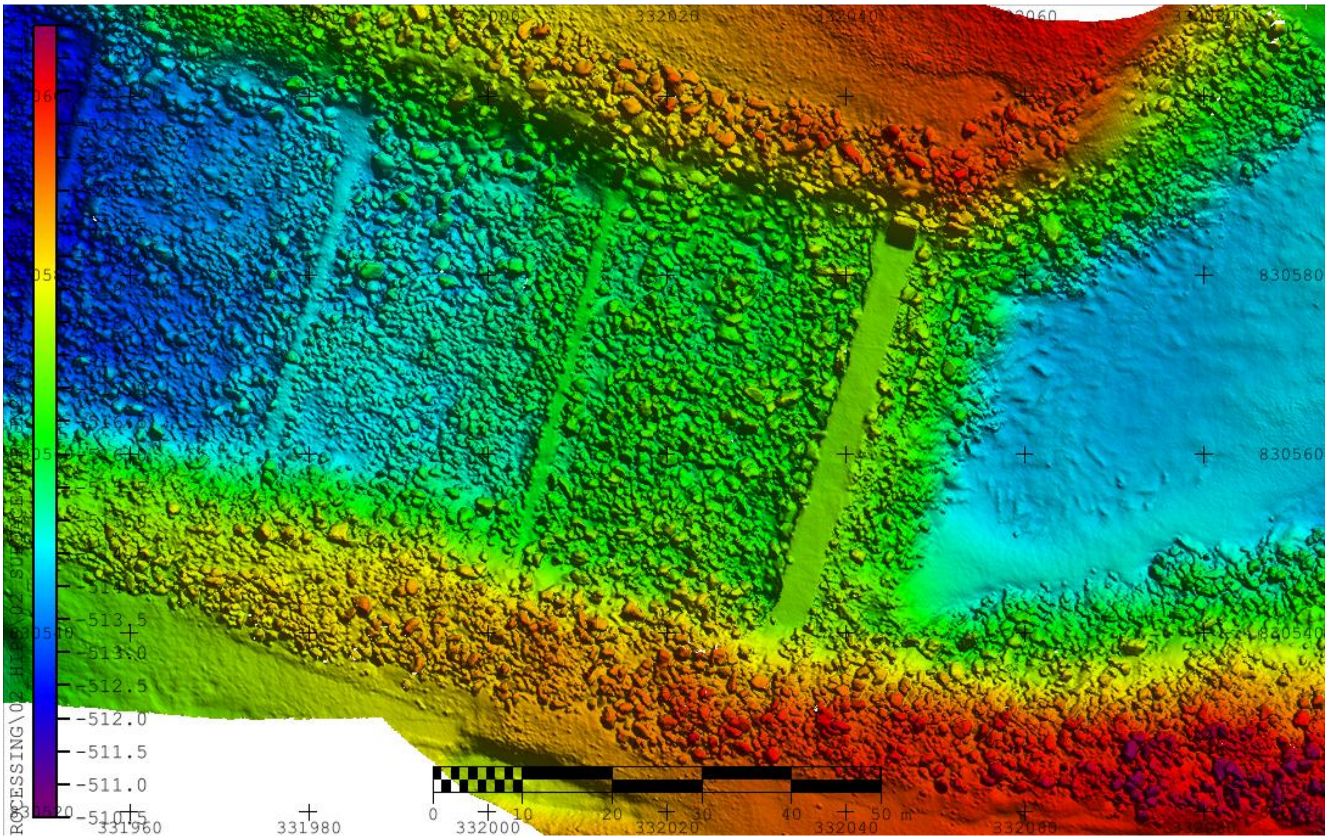
General Description and Location Plots

General Description	A Multibeam Echosounder (MBES) survey of the spillway weir at Tekapo B Power Station, Lake Pukaki, was conducted on 03 February 2026. This Survey Summary Report details the methodology used, QA checks and results obtained.

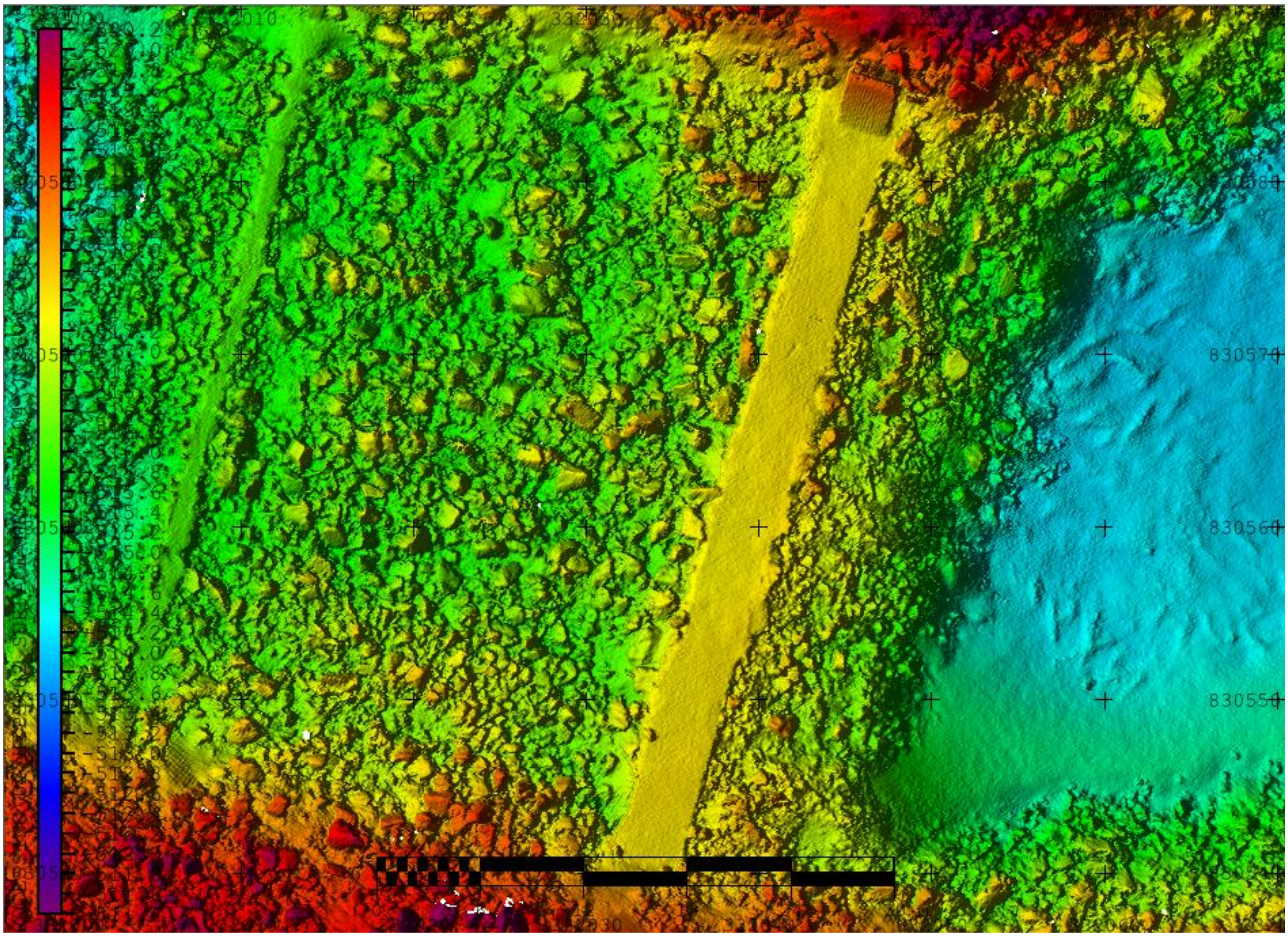
Survey Area – Overview



Sill 1 (Light source 45° azimuth, 45 elevation, 3 x vertical exaggeration)



Sill 1 (Light source 281° azimuth, 45 elevation, 3 x vertical exaggeration)



Horizontal Positioning	**Datum: NZGD2000** **Projection: TIMATM2000**
Connection to Horizontal Datum	Horizontal Control for the survey was generated from an RTK Base Station established at the existing LINZ Geodetic Mark – IT XIV SO 14616 (LINZ Code EQ1H). Raw GNSS observables were also logged at this mark during the survey and subsequently submitted to LINZ via the PositioNZ Post Processing Service. Results confirm the integrity of the coordinates used, specifically the NZGD2000 ellipsoid height which was within 10mm of the value published in the Geodetic Database. Details of both the existing (LINZ GDB) the new (as-surveyed) coordinates for LINZ Mark EQ1H are tabulated below:

RTK Base Station - Coordinates

Parameter	**Detail**			
Station Name	IT XIV SO 14616 (LINZ Code EQ1H)			
Date Established	Not known			
Reference:	=101605616564211770579954&action=+setfoundmarks+updatelist&foundmarklist=EQ1H&listaction=clear+add&mark=			
Reference Point	IT set in concrete, 3.8km along Hayman road from SH 8. Marker post nearby			
Coordinates	Source - LINZ Geodetic database			
	Easting	Northing	Ellipsoid Height	Chart Datum
NZTM20 Timaru Circuit	332721.264	828739.088	546.677	N/A
	Latitude	Longitude	Ellipsoid Height	Chart Datum
NZGD2000 Geographic	44° 08' 24.76893" S	170° 12' 59.10600" E	546.677	N/A

Coordinates	Source - SHL (Fieldwork February 2026)			
	Easting	Northing	Ellipsoid Height	Chart Datum
NZTM2000 Timaru Circuit	332721.2311	828739.0783	546.688	N/A
	Latitude	Longitude	Ellipsoid Height	Chart Datum
NZGD2000 Geographic	44° 08' 24.76923" S	170° 12' 59.10453" E	546.688	N/A

Methods of Obtaining Horizontal Position	Horizontal position was obtained in real time on the survey vessel utilising RTK corrections generated from a temporary Base Station operated by SHL, established over geodetic IT XIV SO 14616 (LINZ Code EQ1H). The RTK base equipment comprised a Trimble NetR9 GNSS, transmitted on the itinerant frequency of 469.0125kHz. RTK corrections in CMR+ format was broadcast in terms of NZGD2000, latitude/longitude/ellipsoid height.

<table>
<tr><th>Vertical Datum</th><th align="right">Datum: Lyttelton 1937</th></tr>
<tr><td>Connection to Vertical Datum</td><td>The vertical datum adopted for the survey is Lyttelton 1937 (LYTHT1937).</td></tr>
<tr><td>Method used to reduce soundings</td><td>Soundings were reduced using RTK heights generated from the SHL Base Station at Geodetic Mark IT XIV SO 14616 (LINZ Code EQ1H). Height data from base station was referenced to the NZGD2000 ellipsoid, which was subsequently converted to LYHY1937 heights by applying:

• A variable offset to account for the variation in the geoid across the survey area. The LINZ NZGeoid2016 model (downloaded from the LINZ website) was used for this purpose.

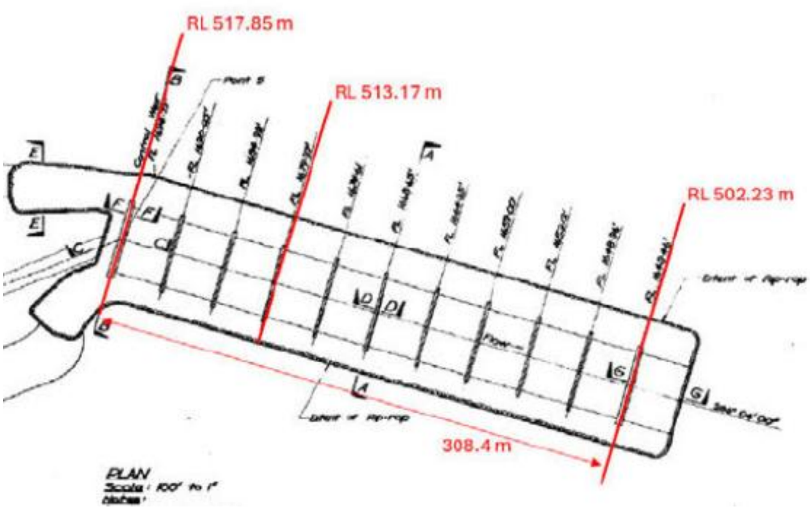
• A fixed offset of 0.348m to bring the height in terms of Lyttelton 1937 Datum. This offset was derived by calculating the difference between the NZVD2016 heights of adjacent LINZ Geodetic Marks (LINZ Codes EQ1H and EQ1N) detailed in the LINZ Geodetic Database) and LYTHT1937 heights using the LINZ Online Datum Conversion Tool.</td></tr>
<tr><td colspan="2">Conversion of NZVD2016 heights to Lyttelton 1937 heights from nearby LINZ Geodfetic Marks

Vertical datum conversion New Zealand Vertical Datum 2016 to Lyttelton 1937 (from NZVD2016)

New Zealand Vertical Datum Conversion Results
Input coordinates: Timaru Circuit 2000
Input heights: New Zealand Vertical Datum 2016
Output heights: Lyttelton 1937 (from NZVD2016)

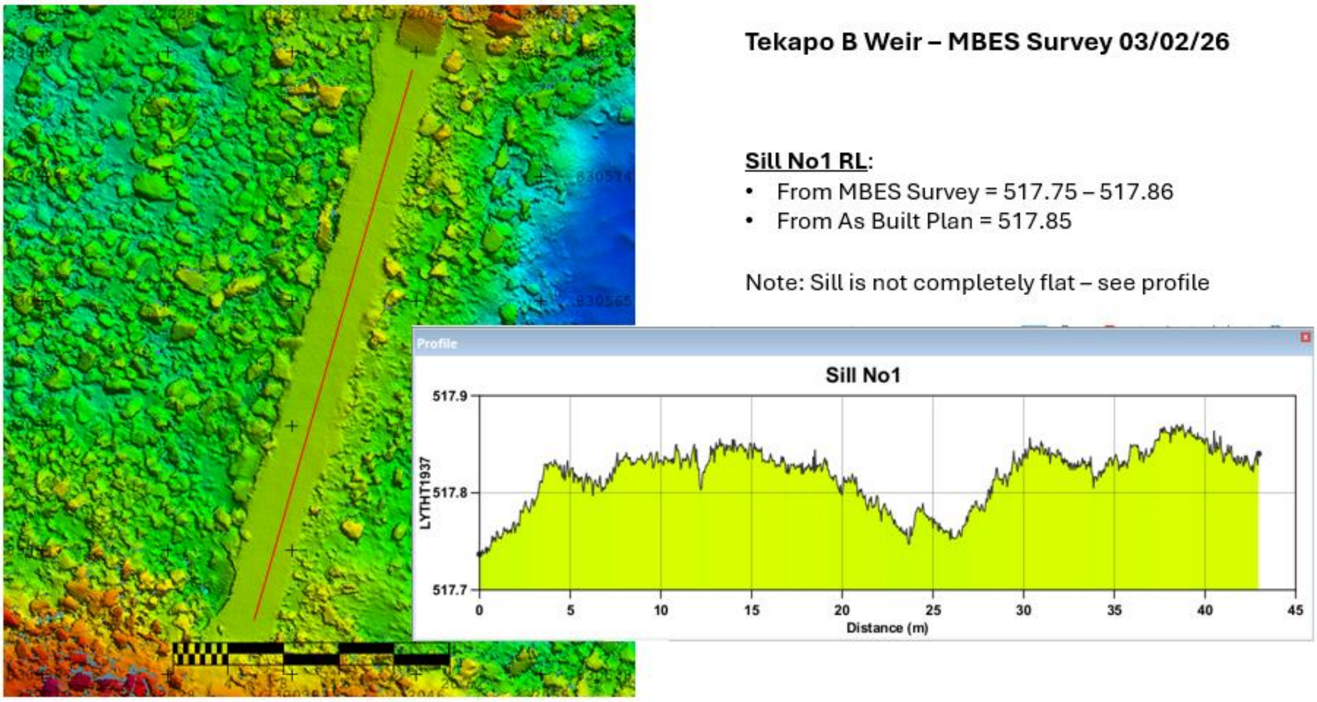
Note
The offset to the Lyttelton 1937 local vertical datum is computed using a vertical datum relationship grid.

EQ1H 332721.264 828739.088 536.470 332721.264 828739.088 536.818
EQ1N 331930.566 832095.798 549.050 331930.566 832095.798 549.398
4024 336891.577 832713.293 871.358 336891.577 832713.293 871.718</td></tr>
<tr><td>Reduced Depth Validation</td><td>The validation of the reduced depth (level) from survey data can be demonstrated by comparison with a common point of known height (depth). For this, reference is made to the “as built” drawing of the weir, detailing the RL of the concrete beams across the weir (source: 12656630 - RPT - Lake Pukaki Engineering Structures Assessment – FINAL, dated 29/10/25):


</td></tr>
</table>

Static Validation of Vertical Height Solution – Results

The MBES survey revealed the top surface of the No1 Sill is not completely flat/uniform along its centreline, ranging in height between 517.45 to 517.86m – see below. Notwithstanding this, the results confirm the vertical height solution is valid (within +/-2.5cm of "as built" height).



Depth Measurement

Survey Vessel Description (Length, Beam, Hull Type)

Vessel Details:

Survey Vessel Name:	SMB PANDORA II
Official No.	MNZ 135467
Owner:	Southern Hydrographic Limited
Hull Type:	Alloy Catamaran
Length Overall	7.1m
Beam:	2.5m
Draught (hull)	0.42m
Displacement	2.5Tonne
Engines	2 x Yamaha 115 four stoke
Electrical	12 VDC and 230VAC power

Method(s) to be used to Determine Least Depths

Multibeam Echosounder Specifications:

Multibeam Make/Model	Teledyne Reson T51-R
Frequency	400kHz and 800kHz (400kHz used)
Beam-width	1° x 0.5° at 400kHz, 0.5° x 0.25° at 800kHz
Maximum Ping Rate	50Hz
Number of Beams	1024 (equidistant) at 400kHz
Max swath angle	140° in equidistant mode; 165° in equiangle
Depth Resolution	0.006m

Echo Sounder Frequency(s)

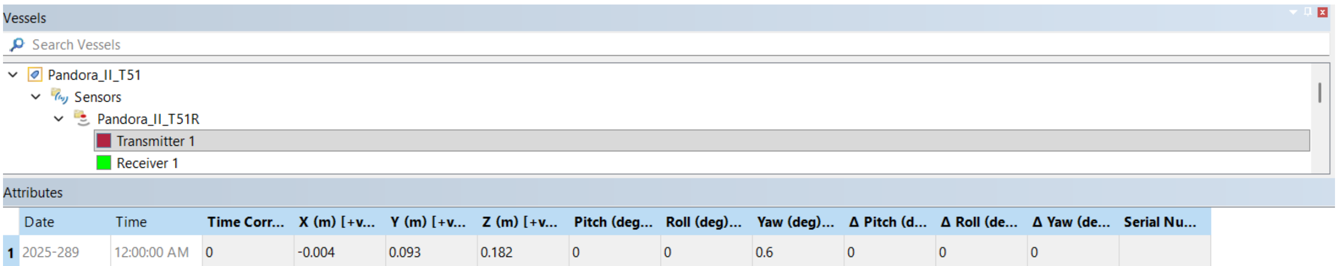
The Reson T51-R was operated at 800kHz during the survey.

Method and Frequency of Echo Sounder Calibration

A patch was conducted on the 19/10/2025 during T51 MBES installation trials in Nelson. The angular bias of the MBES transducer with respect to the POS/MV IMU was determined using the Caris HIPS calibration tool which were then subsequently set in the Qinsy Data Acquisition System and reapplied to raw ".s7K" files during post processing in Caris HIPS.

An extract from the Caris Vessel File showing the transducer bias angles (receiver and transmitter) derived from the patch test is shown below.

Vessel File – Patch Test Results – 19/10/25



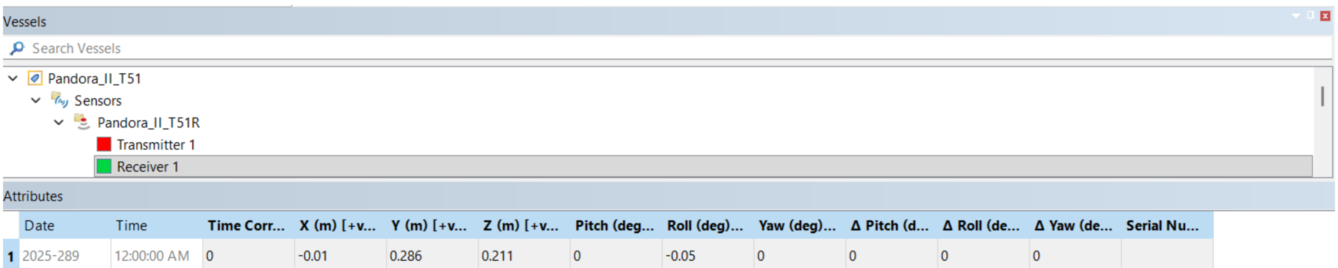
Vessels

Search Vessels

Pandora_II_T51
Sensors
Pandora_II_T51R
Transmitter 1
Receiver 1

Attributes

	Date	Time	Time Corr...	X (m) [+v...	Y (m) [+v...	Z (m) [+v...	Pitch (deg...	Roll (deg)...	Yaw (deg)...	Δ Pitch (d...	Δ Roll (de...	Δ Yaw (de...	Serial Nu...
1	2025-289	12:00:00 AM	0	-0.004	0.093	0.182	0	0	0.6	0	0	0	



Vessels

Search Vessels

Pandora_II_T51
Sensors
Pandora_II_T51R
Transmitter 1
Receiver 1

Attributes

	Date	Time	Time Corr...	X (m) [+v...	Y (m) [+v...	Z (m) [+v...	Pitch (deg...	Roll (deg)...	Yaw (deg)...	Δ Pitch (d...	Δ Roll (de...	Δ Yaw (de...	Serial Nu...
1	2025-289	12:00:00 AM	0	-0.01	0.286	0.211	0	-0.05	0	0	0	0	

Method to Compensate for Transducer Motion	Vessel attitude (heave, pitch, roll and heading) data was provided by an integrated POS/MV Wavemaster II (Serial No. 6448C3118) coupled with a Teledyne Type 20 Inertial Measurement Unit (IMU, Serial No. A3002319). Delayed heave was subsequently applied during post processing in Caris HIPS to reduce motion artefacts arising from settling time and long period swell.
Sound Velocity	The velocity of sound in water was continuously logged at the transducer head and employed for beam steering. Two (2) SV profile observations (dips) were conducted in the vicinity of the weir using an AML Base X SVP probe and used in HIPS to correct for refraction errors during post processing in Caris HIPS.
Limiting Sea Conditions affecting Survey Quality	MBES survey operations were conducted in ideal conditions.

Seabed Coverage

Method to Ensure Seabed Coverage Criteria is met	MBES sounding lines were run to ensure 200% coverage was achieved along the entire length and width of the weir.
Echo Sounder Pulse Repetition Rate	Maximum ping rate of the Teledyne Reson T51-R is 50Hz; typically ping rates of between 30-40Hz were achieved during the survey.
Beam Widths - Along Track and Across Travel	The following beam widths were achieved (at 800kHz) • Along Track = 1° • Across Track = 0.5°
Survey Vessel Speed	Survey vessel speed was typically 5-6kn during the survey.
MBES Coverage (overlap)	MBES survey lines were run at variable spacing to ensure 200% coverage. This was achieved by running along the edge of the previous line (also known as "half-stepping).
Sounding Orientation	Sounding lines orientated parallel to weir/channel.
Sounding Density – plot of soundings per 1m node	The density node functionality within the Caris HIPS software was used to produce sounding density plots of all sub-area. Inspection the density surface statistics of these plots show sounding density to be typically better than 500 per 1m grid cell.

Sounding Reduction and Data Presentation

Methods to Reduce Raw Data to Sounding Datum	The following process was employed to reduce raw sounding data to LTYHT1937: 1. Raw .s7K MBES data imported into Caris HIPS 2. GNSS tide, derived from RTK ellipsoid heights, applied to MBES data, together with LYTHT1937-NZVD2016 fixed offset (0.348) and NZGeoid2016 model offset (variable) 3. Sound Velocity Refraction correction (SVC), delayed heave and vessel offsets applied to MBES data 4. MBES data merged with position to produce reduced depths.
Structure Infrastructure and Topography/Imagery	Detailsof the Tekapo B weir was sourced from the NZ Government Fast Track Projects website: https://www.fasttrack.govt.nz/projects/lake-pukaki-hydro-storage-and-dam-resilience-works Background imagery from LINZ Data Service, Canterbury 0.3m Rural Aerial Photos (2017-2018) Reproduced under Creative Commons Attribution 4.0 International Public License.

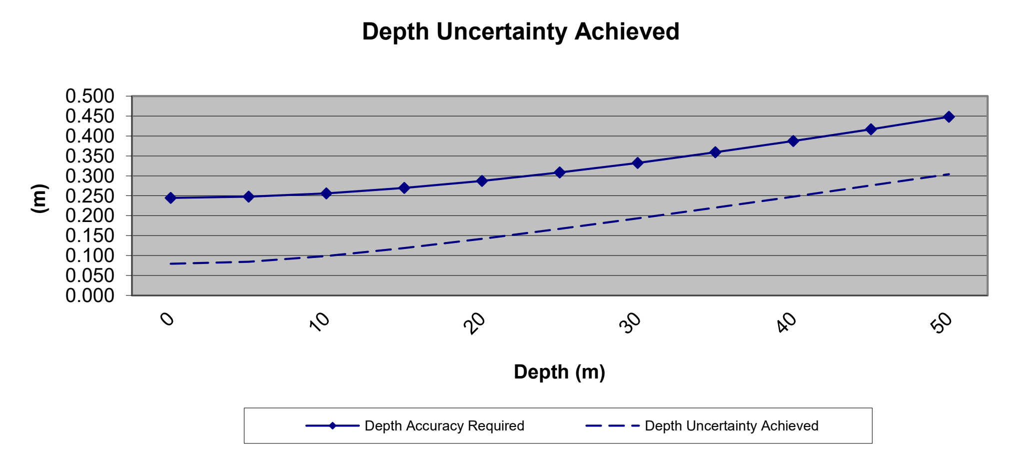
Data Quality

The Method(s) used to Derive the Quality of the Data and Ability to meet the Depth Tolerance as Required in the Standards	In lieu of a specified uncertainty (accuracy) standard for this survey, the LINZ Special Order standard has been adopted. The requirements for this order of survey are detailed in LINZ Contract Specifications for Hydrographic Surveys Version 2.0. To demonstrate LINZ Special Order standard has been met, individual sources of uncertainty (commonly referred to as errors) have been identified and assessed as part of a combined model or Total Propagated Uncertainty (TPU).

MBES Vertical TPU	Swath Width		3 X WD	Note	Depth (m)	Depth (m)	Depth (m)
Source of Uncertainty	Depth Independent Uncertainty		Depth Dependent Uncertainty		5	10	15
Vessel Draught Setting	0.00			a	0.00	0.00	0.00
Variation of Vessel Draught Setting	0.00			b	0.00	0.00	0.00
Vessel Settlement and Squat	0.00			c	0.00	0.00	0.00
MBES Instrument Accuracy	0.05	±	0.50% d	d	0.06	0.07	0.09
Roll Uncertainty			0.0013 d	e	0.01	0.01	0.02
Heave Uncertainty	0.05			f	0.05	0.05	0.05
Sound Velocity Measurement			0.0021 d	g	0.01	0.02	0.03
Sound Velocity Spatial Variation			0.0013 d	h	0.01	0.01	0.02
Sound Velocity Temporal Variation			0.0013 d	i	0.01	0.01	0.02
Tide Data Accuracy	0.03			j	0.03	0.03	0.03
Co-Tidal Uncertainty	0.02			k	0.02	0.02	0.02
Combined Total	0.08	±	0.0059 d		0.08	0.10	0.12
Requirement IHO/LINZ Special Order	0.25	±	0.0075 d		0.25	0.26	0.27
Standard Met					YES	YES	YES

Comments

a. Draught error eliminated with use of RTK Heighting methodolgy
b. Varaition of draught error eliiminated with use of RTK Heighting methodolgy
c. Squat/Settlement error eliminated with use of RTK Heighting methodolgy
d. Determined from manufacturers specifications of 1cm resolution and experience from previous surveys using the Reson "T" series MBES systems. An accuracy of 5cm + 0.5% of depth conservative estimate of the depth accuracy.
e. Maximum depth error in outer beams. Based on roll error used in position error TPU, ie +/- 0.05°.
f. Applanix POS/MV specifications state equipment real-time accuracy of 5% of heave experienced (heave experienced less than 1.0m metres during survey). Application of delayed heave during post-processing reduced residual heave artefact in worst case to 0.05m.
g. Manufacturers estimate of sound velocity sensor accuracy (+/-1%)
h. Spatial variation in sound velocity solution estimated to be better than 2m/s (0.0013d).
i. Temporal variation in sound velocity solution estimated to be better than 2m/s 0.0013d).
j Estimated accuracy of tidal observations at the gauge. Tides not used in this instance but figure retained as representing vertical height uncertainty using RTK heighting methodology.
k Not a co-tidal error as such, but representative of NZGeoid16 uncertainty when used in conjunction with RTK heighting.



Deliverables

Digital Deliverables	The table overleaf lists the Digital Deliverables that accompany this Summary Report of Survey. Note: Bathymetric surfaces in Caris .csar format can be interactively viewed and interrogated in the Caris "EasyView" software, which can be downloaded from the Caris website (registration required): https://www.teledynecaris.com/en/products/easy-view/

MER_JN2604 - Deliverables

Deliverable (Folder)	Description	Format	File	Rendered
01 - XYZ Data	Processed, CUBE	ASCII XYZ	MER_JN2604_TKB_800_C2_5cm_030226_TIMATM2000_v2 MER_JN2604_TKB_800_C2_10cm_030226_TIMATM2000_v2	09/02/2026
	Processed, Point Cloud	LAS	15 x Survey Lines in following name descrition: YYYYMMDD_HHMMSS_XX-MER_JN2604_2-NNNN.las	09/02/2026
02 - Bathymetric Surface	CUBE Surface	Caris .csar	MER_JN2604_TKB_800_C2_5cm_030226_TIMATM2000_v2 MER_JN2604_TKB_800_C2_10cm_030226_TIMATM2000_v2	09/02/2026
03 - GeoTIFF	GeoTiff Raster	.tiff .twf	MER_JN2604_TKB_800_C2_5cm_030226_TIMATM2000_v1 MER_JN2604_TKB_800_C2_10cm_030226_TIMATM2000_v1	09/02/2026
04 - Report	Summary Report of Survey	PDF	MER_JN604_ROS_v1	09/02/2026

I certify that this Survey Report and the methods described herein conform to the hydrographic survey meeting the Survey Standard.

D.L. MUNDY

Supervising Surveyor

Certified Practitioner Hydrography Level 1 (AHSCP)

Appendix B Dive Inspection Report (Divepro, 2026)

Brief Report

Tekapo B spillway 3 February 2026.

Divepro Ltd |11 Parata Ave, Karitane, Dunedin 9471
Email: rob@divepro.co.nz Phone:+64 21762 734 Web: www.divepro.co.nz

On 3rd February 2026 a dive team inspected the structure of the Tekapo B powerhouse spillway.

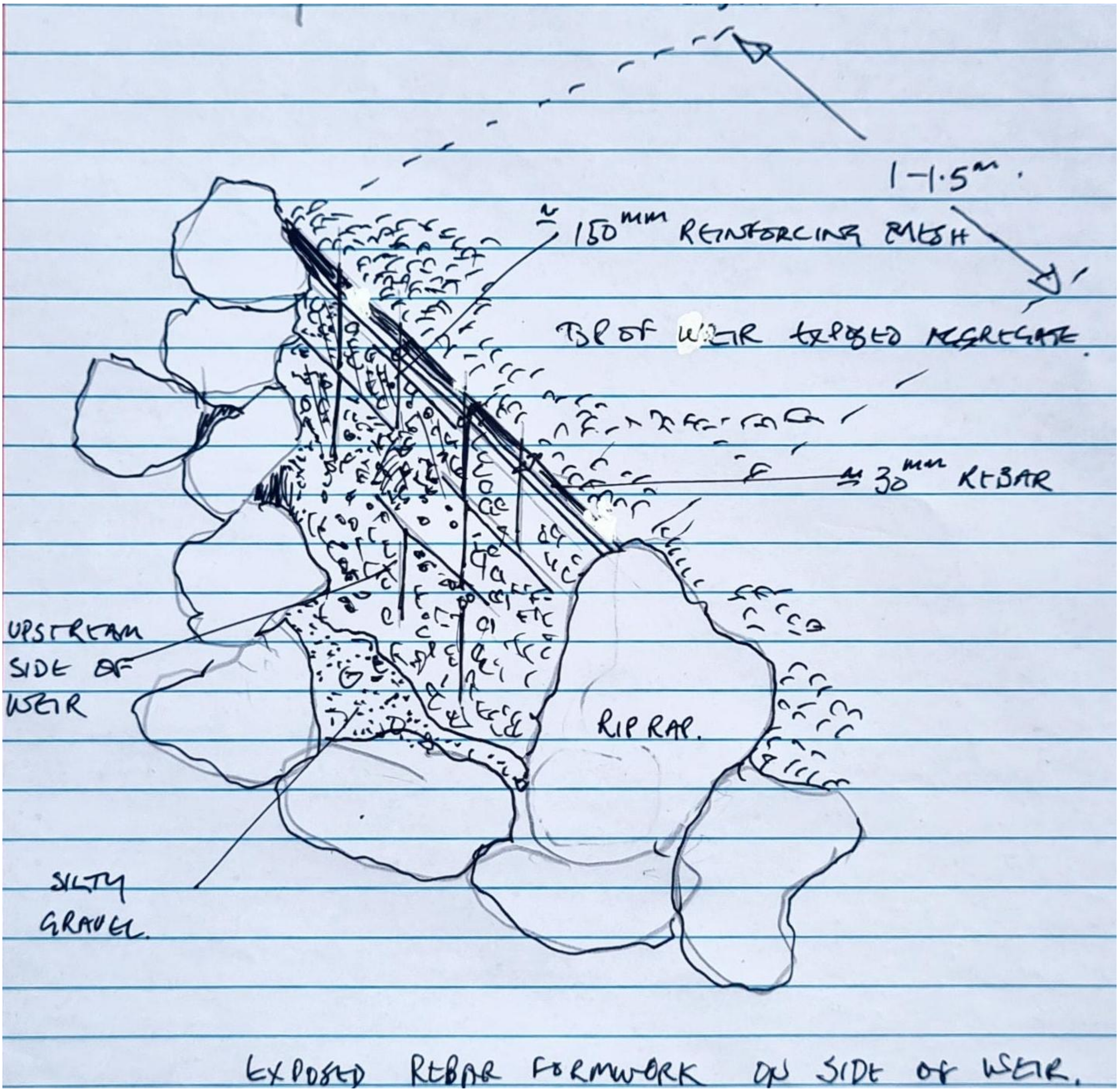
The weather was cold, visibility was limited to between 150mm to 250mm. Depth was approximately 15 metres.

The riprap on the upstream or inshore side of the weir was generally smaller than the riprap on the offshore or downstream side.

The ground, where exposed upstream, was a gritty dusty gravel about 300 millimetres below the top of the weir.

The riprap was level or slightly higher than the top of the weir.

The line of the weir's upstream edge was intermittently a vertical concrete face with exposed aggregate and exposed steel mesh (approximately 150 x 150) tied into 32mm steel re-bar on the top corner by way of a form rather than reinforcing.

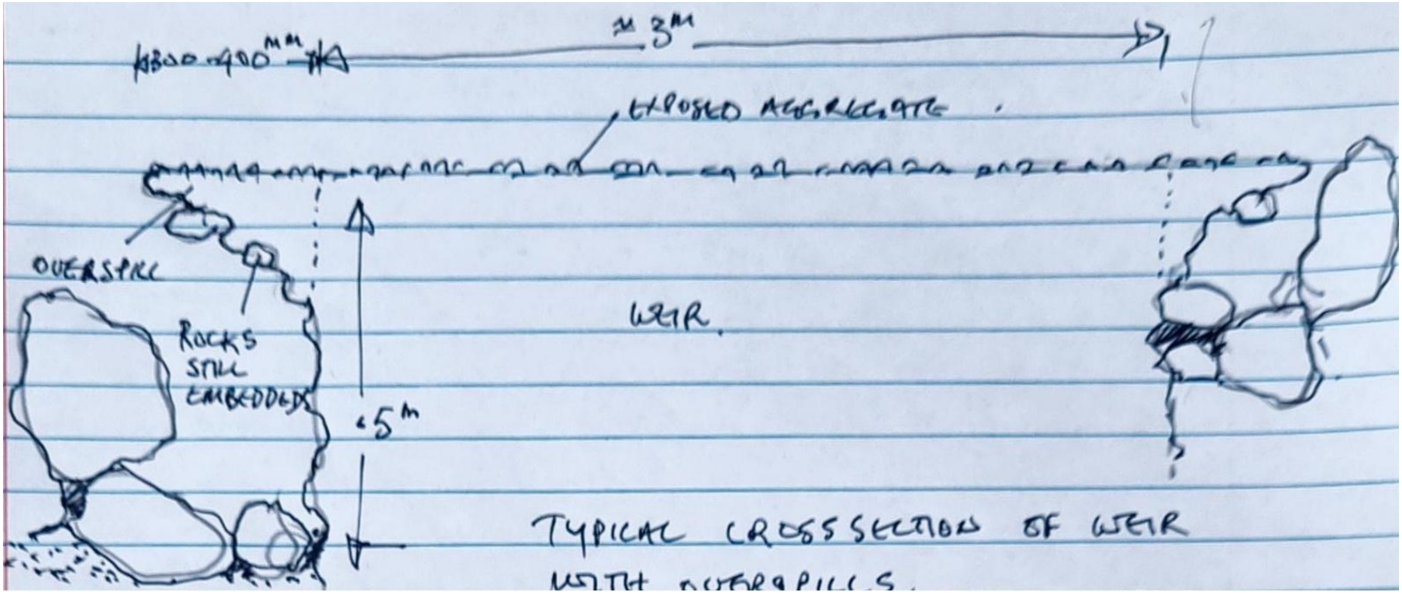


The steel was partially exposed due to it being the outside of the pour (not erosion).

These sections were not more than one to 1.5 metres long and approximately 0.5 metre high.

The upstream edge also had protrusions of concrete overspill locking in rocks and cobbles where the riprap was level or higher than the top of the weir.

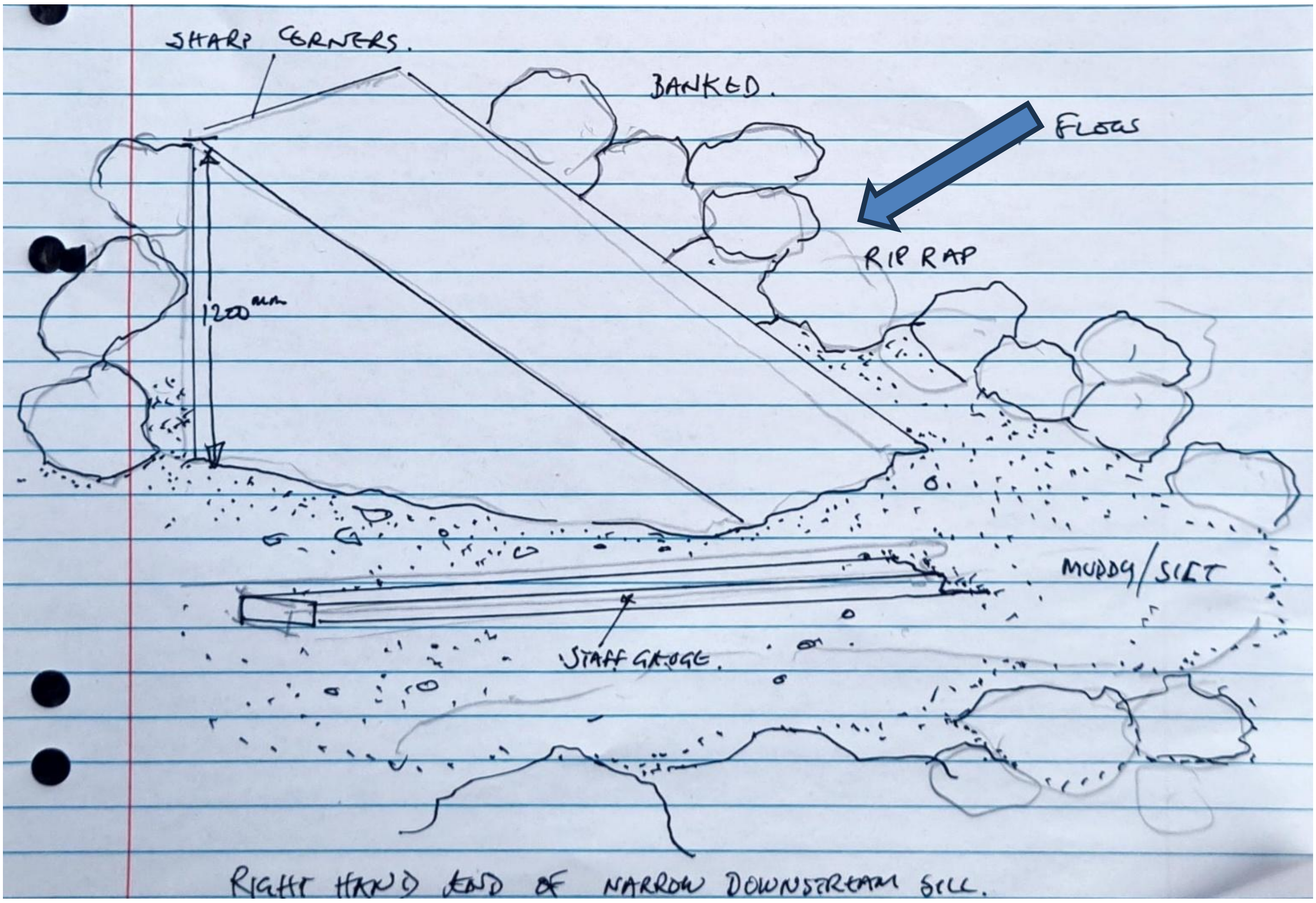
In other places the material trapped by the concrete overspill remained as a cantilevered tongue overhang, and any loose material had been washed away from underneath it, but no further than the line of the weir.

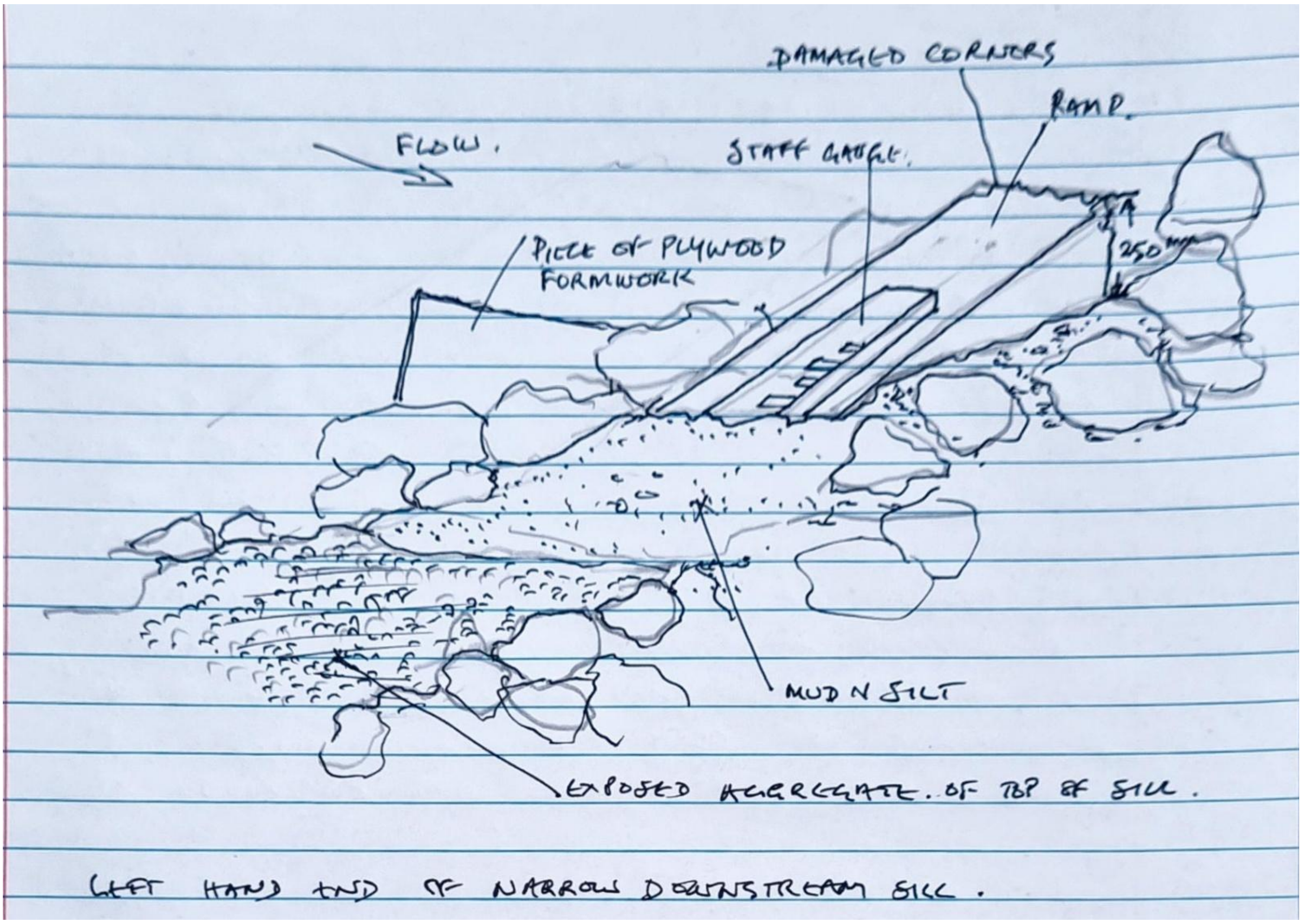


This pattern continues across the upstream face.

At the ends of the Weir where the ground level rose, a 45° ramp was cast into the sill that is about 1 metre wide relative to the weirs 3 metres or more, and on the central line of the top of the weir. The ramps have been formed with plywood and the surfaces have no exposed aggregate unlike the top of the sill.

On the right, the top of the ramp is 800 millimetres above ground and the corners are sharp and unbroken.





An inclined wooden staff gauge is attached to the 45° angle. The bottom of the ramp and staff gauge are buried in mud and riprap.

The left hand end is similar in construction to the right, but the top of the ramp is 250 millimetres above ground level.

A staff gauge is in place, but the corners of the ramp are beaten and broken to be rounded like chamfering and there is more mud and riprap at the base.

The downstream edge is similarly ragged but always in excess of the line of the weir.

The riprap is larger and where it hasn't remained cast in to make the line of the edge, some large rocks have become detached, but have left the pattern of their surfaces on the separated concrete edge

The fines on the downstream side have been washed out to a greater extent, destabilising the larger riprap to allow separation usually 200 or 300 millimetres.

The lake bed is more riprap and rock than gravelly mud bottom.

In places the downstream face is exposed to 1500 millimetres below the top of the sill, but not undermined more than the depth of the overspill.

The top surface of the weir is essentially flat with exposed aggregate where the top few millimetres of cement has washed out.

No weed was growing on it or the rocks, and everything had a light coating of silt or dust.

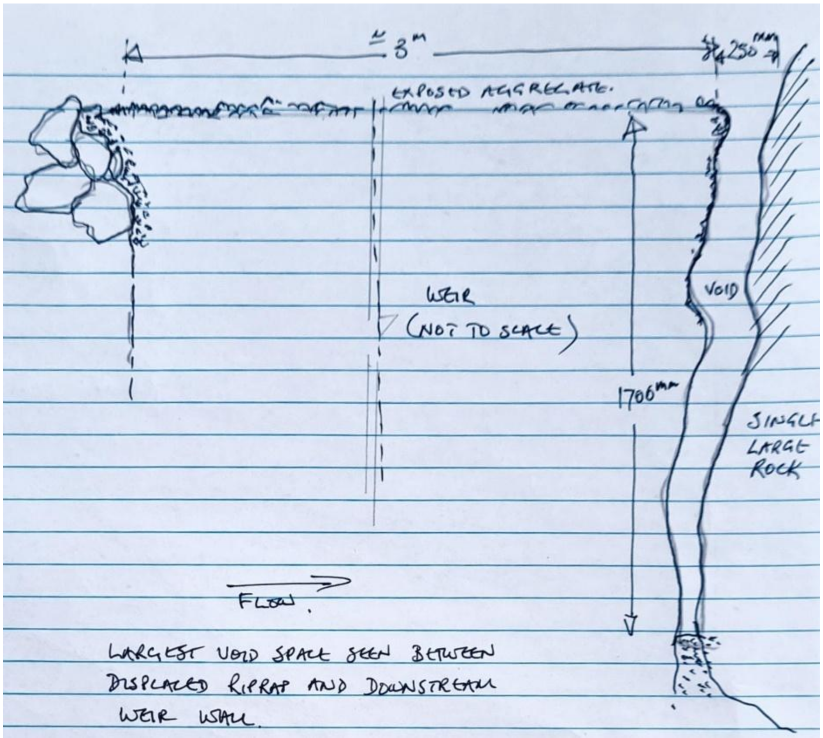
No cracking or spalling was seen and apart from some movement of the riprap the structure appears to remain as built.

The first sill downstream of the weir while smaller in width by (i.e. 3 metres versus 1 metre) is in a very similar condition.

Mostly the upstream riprap is level with, and cast into, the upstream wall as the form.

There are some patches where the lake bed is flat, and mud impregnated gravels are approximately 300mm lower than the top of the sill to form areas around four or five square metres.

The deepest exposure of the sill was in a 250-millimetre slot between a very large rock and the downstream side of the sill where the rock had settled and separated. The slot was approximately 1700 millimetres deep.



The top of the ramp on the right-hand end was standing 1200 millimetres higher than the bank/ lakebed and the staff gauge had become detached and was lying on the lakebed beside the ramp.

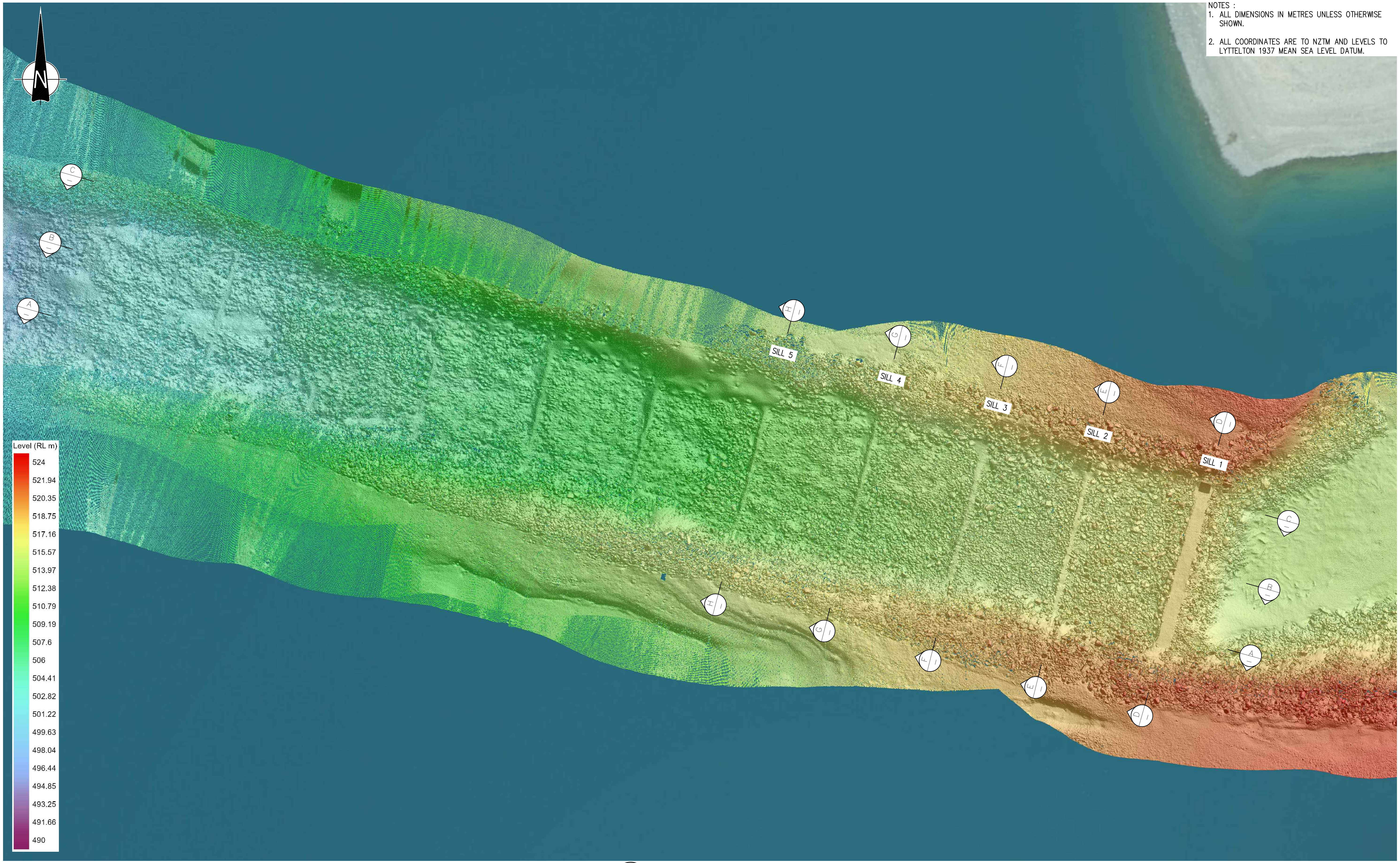
The marks and numbering on the gauges appeared as new (paint on timber)

The left-hand end was similar to the weir, 250mm above lakebed and some sharp corners battered, but with no wear obvious on the edges of the wooden staff or lettering.

No sign of cracking or tipping, the top surface is exposed aggregate and the screeding not as level as the weir, otherwise as built.

Appendix C Bathymetric Survey Drawings

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
2. ALL COORDINATES ARE TO NZTM AND LEVELS TO LYTTELTON 1937 MEAN SEA LEVEL DATUM.



LAYOUT PLAN
SCALE 1:1000

1:1 000 0 10 20 30 40 50 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

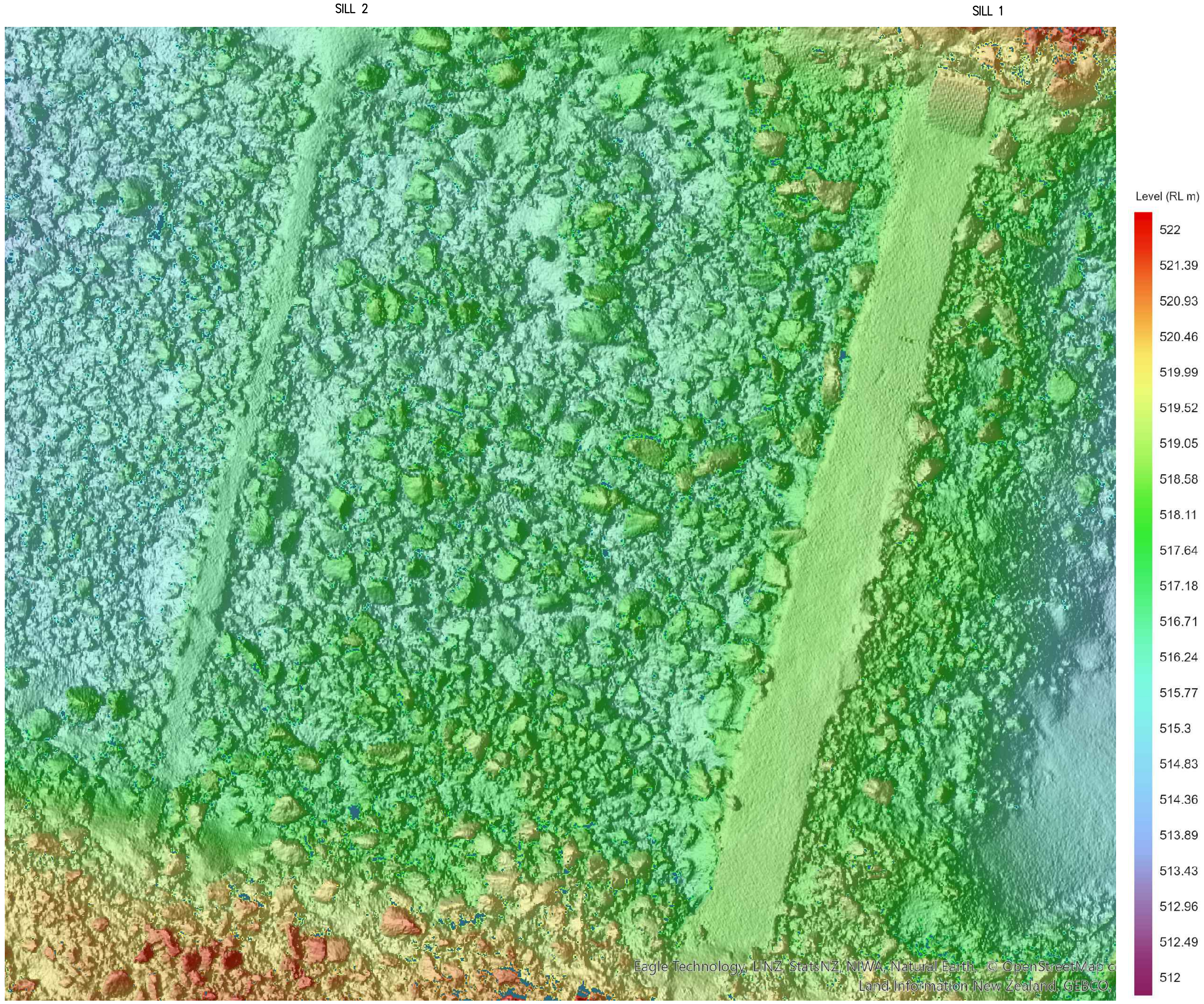
Meridian.

TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
LAYOUT PLAN

FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\OVERVIEW
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
2. ALL COORDINATES ARE TO NZTM AND LEVELS TO LYTTELTON 1937 MEAN SEA LEVEL DATUM.



PLAN – BAY 1
SCALE 1:300

1:300 0 2 4 6 8 10 12 14 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
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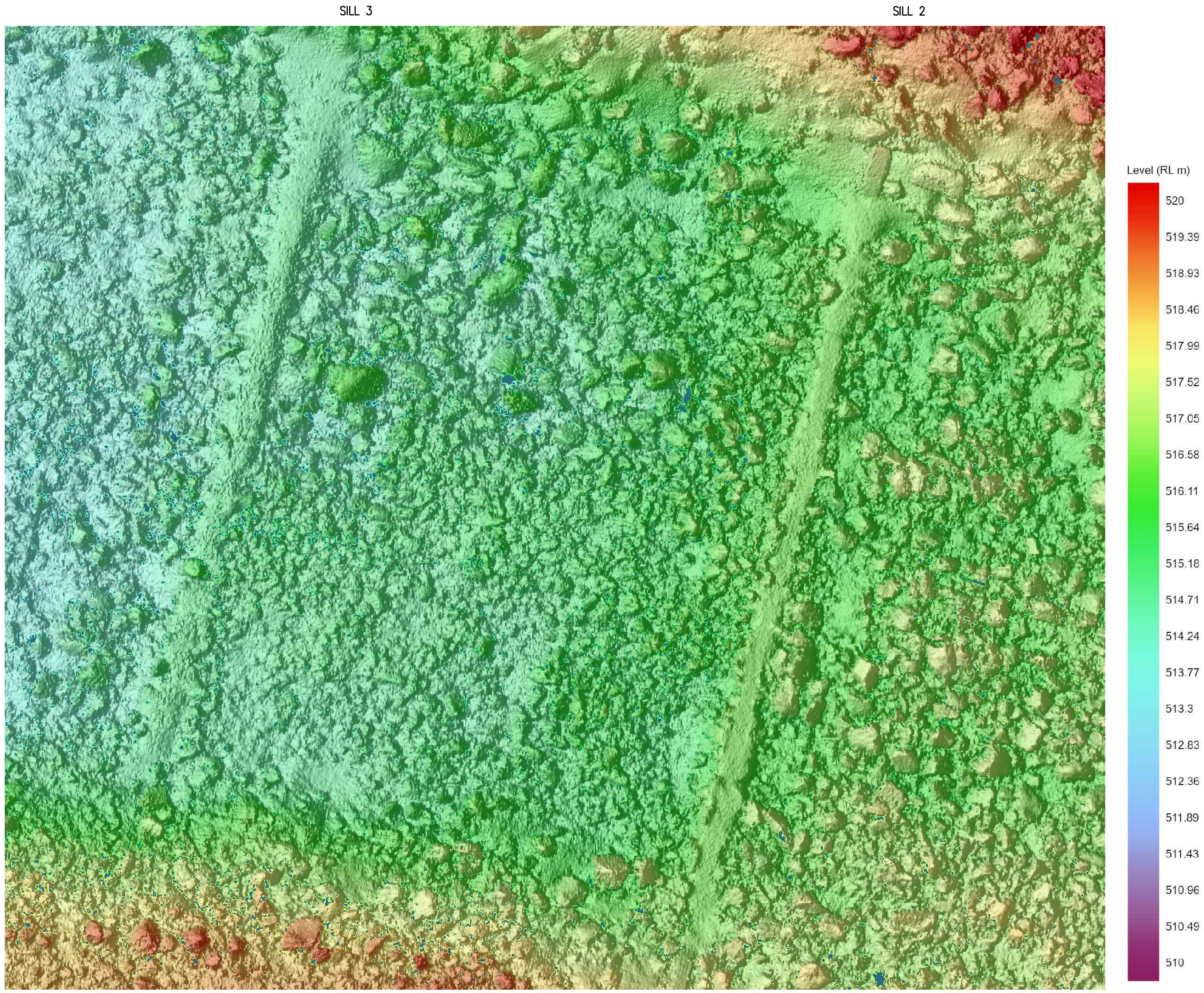
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
BAY 1
PLAN – 1 OF 4

FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK-B\TW\CHT\BAY-1
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
2. ALL COORDINATES ARE TO NZTM AND LEVELS TO LYTTELTON 1937 MEAN SEA LEVEL DATUM.



PLAN – BAY 2
SCALE 1:300

1:300 0 2 4 6 8 10 12 14 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

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TEKAPO B PS TAILRACE WEIR

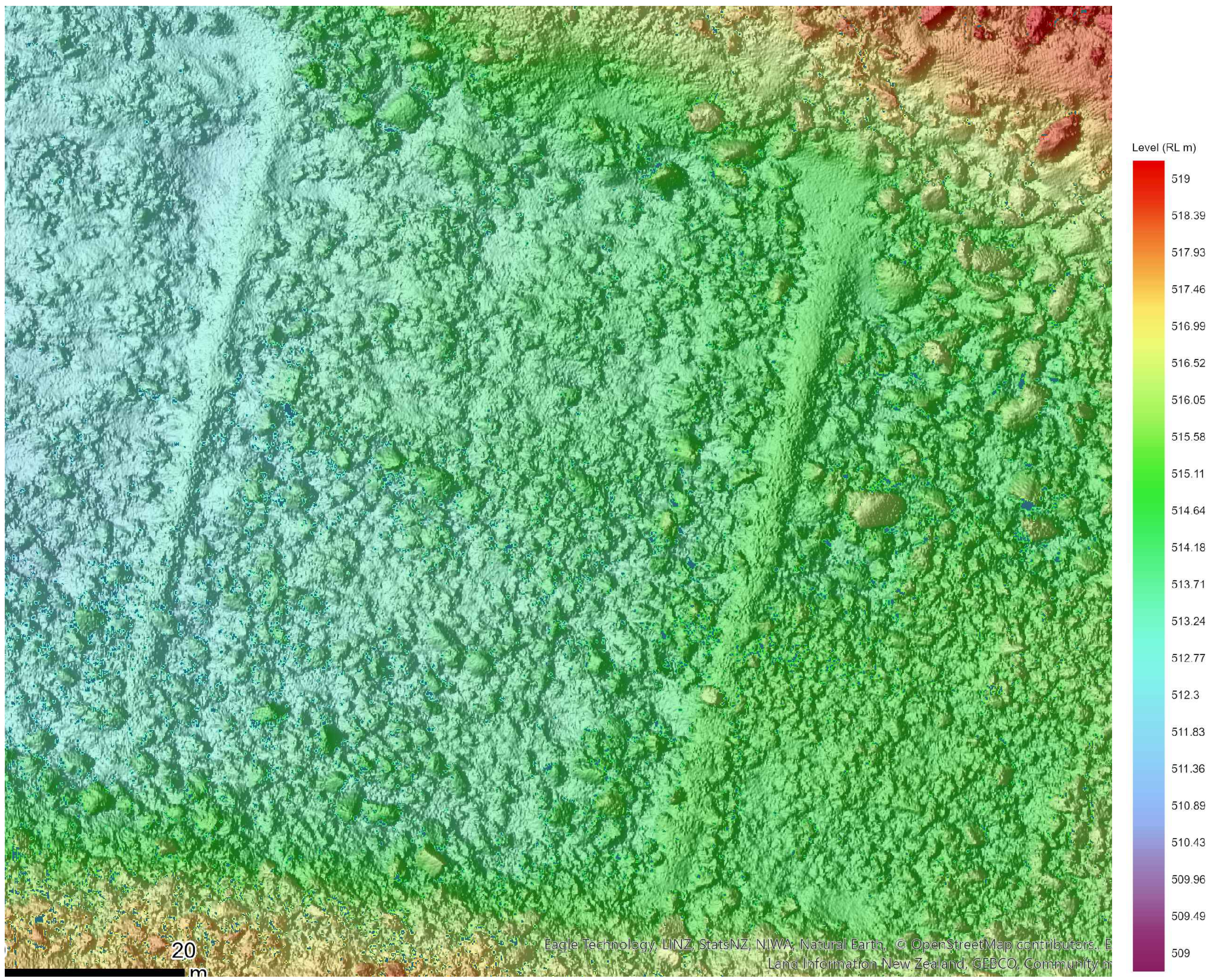
CONDITION ASSESSMENT
BAY 2
PLAN – 2 OF 4

FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK-B\TW\CHT\BAY-2
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
2. ALL COORDINATES ARE TO NZTM AND LEVELS TO LYTTELTON 1937 MEAN SEA LEVEL DATUM.

SILL 4

SILL 3



PLAN – BAY 3

SCALE 1:300

1:300 0 2 4 6 8 10 12 14 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

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TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
BAY 3
PLAN – 3 OF 4

FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK-B\TW\CHT\BAY-3
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
2. ALL COORDINATES ARE TO NZTM AND LEVELS TO LYTTELTON 1937 MEAN SEA LEVEL DATUM.

PLAN – BAY 4
SCALE 1:300

1:300 0 2 4 6 8 10 12 14 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
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Meridian.

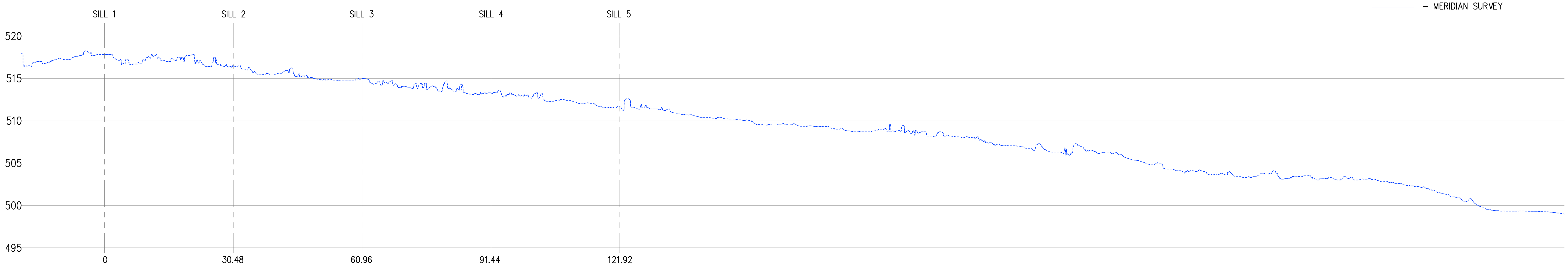
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
BAY 4
PLAN – 4 OF 4

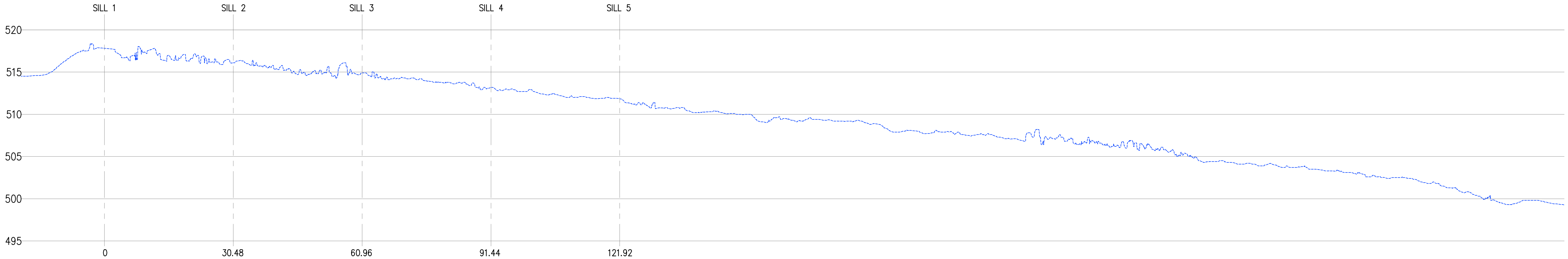
FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\BAY–4
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.

KEY:
— MERIDIAN SURVEY



A – LONG SECTION ALONG LEFT SIDE OF STRUCTURE
SCALE 1:1000 H
1:500 V



B – LONG SECTION ALONG CENTRELINE OF STRUCTURE
SCALE 1:1000 H
1:500 V

C – LONG SECTION ALONG RIGHT SIDE OF STRUCTURE
SCALE 1:1000 H
1:500 V

1:1 000 — 0 10 20 30 40 50 m
1:500 — 0 4 8 12 16 20 24 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

Meridian.

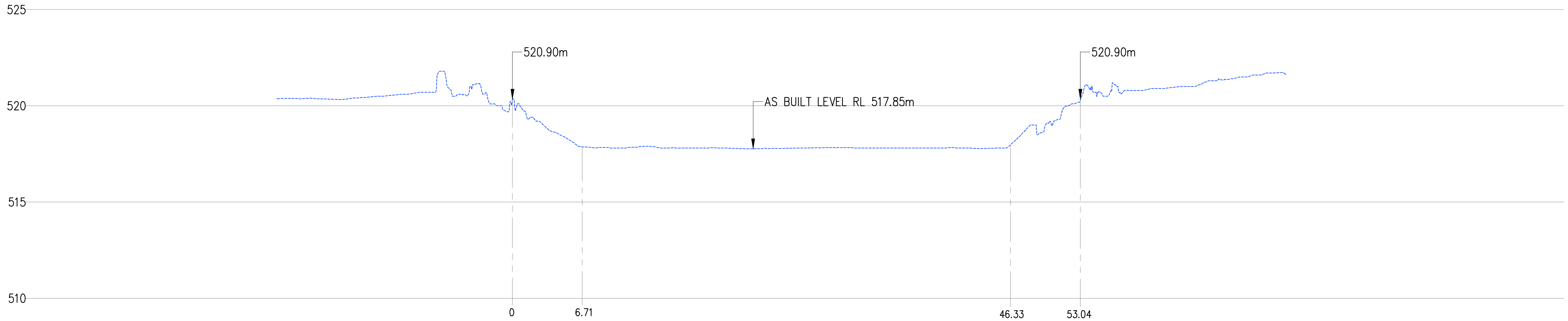
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
LONG–SECTIONS

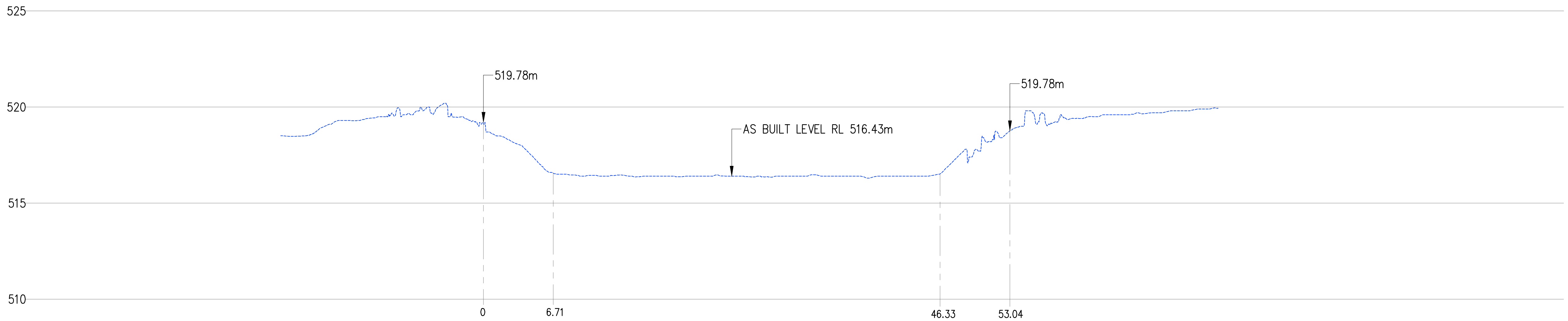
FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\PROFILE SECTION
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.

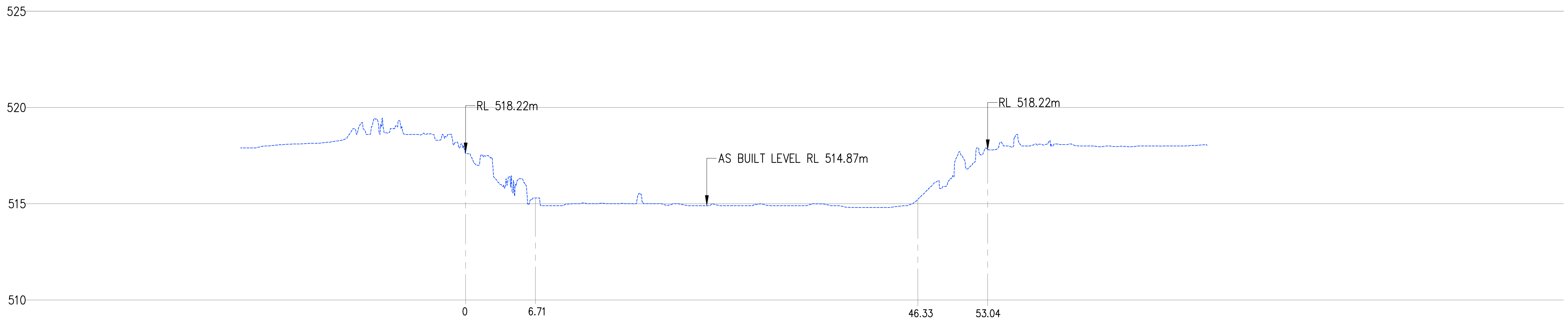
KEY:
— MERIDIAN SURVEY



D
CROSS–SECTION OF STRUCTURE AT CENTRELINE OF SILL 1
SCALE 1:500 H
1:250 V



E
CROSS–SECTION OF STRUCTURE AT CENTRELINE OF SILL 2
SCALE 1:500 H
1:250 V



F
CROSS–SECTION OF STRUCTURE AT CENTRELINE OF SILL 3
SCALE 1:500 H
1:250 V

1:500 0 4 8 12 16 20 24 m
1:250 0 2 4 6 8 10 12 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

Meridian.

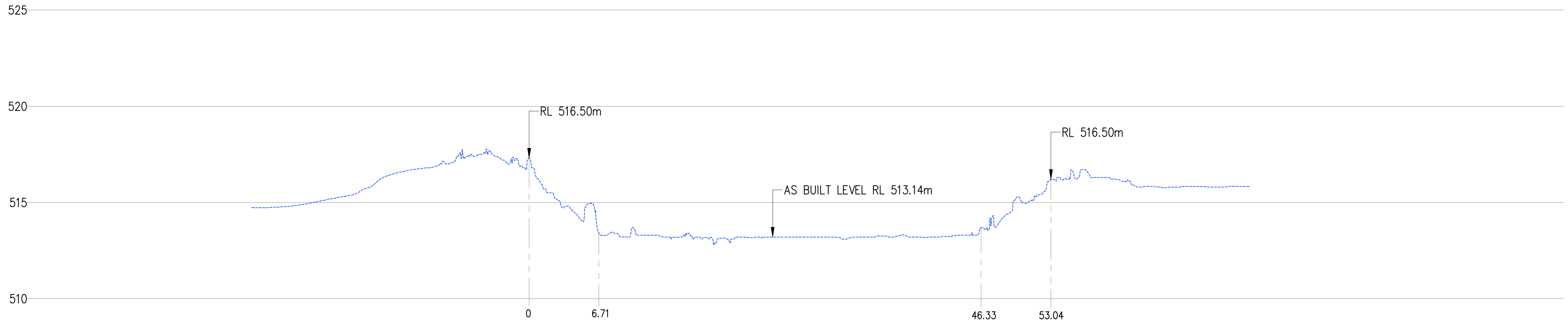
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
CROSS–SECTIONS
1 OF 2

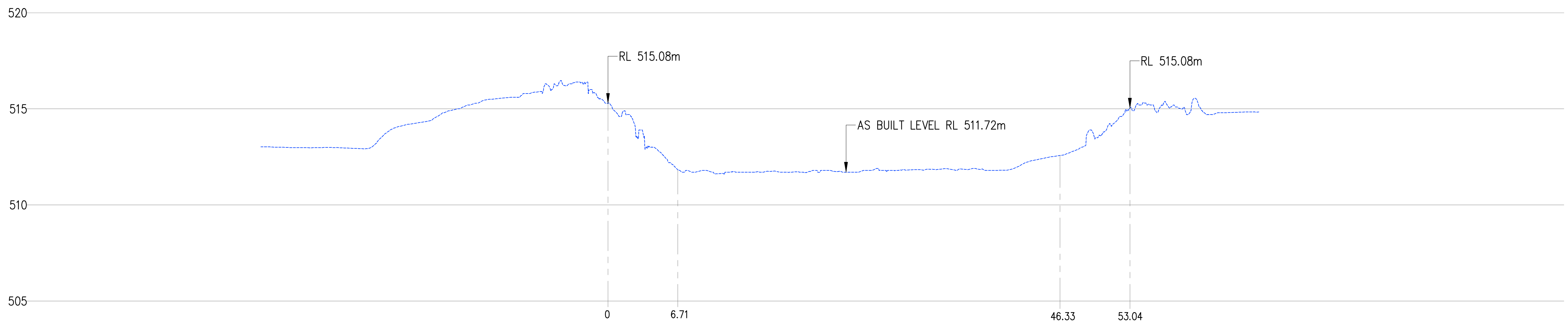
FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\CROSS–SECTION
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

NOTES :
1. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.

KEY:
— MERIDIAN SURVEY



G CROSS–SECTION OF STRUCTURE AT CENTRELINE OF SILL 4
SCALE 1:500 H
1:250 V



H CROSS–SECTION OF STRUCTURE AT CENTRELINE OF SILL 5
SCALE 1:500 H
1:250 V

1:500 0 4 8 12 16 20 24 m
1:250 0 2 4 6 8 10 12 m

DRAWING STATUS: FOR INFORMATION

ISSUE	AMENDMENT	BY	CH'D	COMPANY	PROJECT	APP'D	DATE
R0	ISSUED FOR INFORMATION	JA	Dr.GW	DAMWATCH	E2621	DCE	02/26

Meridian.

TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
CROSS–SECTIONS
2 OF 2

FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\CROSS–SECTION
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

Appendix D Genesis Energy Comments on 27 February 2026 Version of Memorandum

Meridian Response to Genesis Review Comments

No.	**Genesis Review Comment**	**Location in Revised Damwatch Memorandum (18 March 2026)**	**Meridian Response**
1	Hydraulic conclusion wording Change from "should" to "will" regarding riprap stability	Footnote 4 on page 5; Section 6.2	See updated Section 6.2 and Footnote 4
2	Interpretation of riprap reinstatement recommendation	Section 7.3	See updated Section 7.3
3	Operating regime assumptions	Section 5.4	See the Lake Pūkaki Hydro storage and dam resilience works - Substantive Application - Sections 3.1 and 3.1.1

Genesis Energy Limited
155 Fanshawe Street
Auckland CBD 1010
New Zealand

9 March 2026

Genesis' comments on Damwatch's report "Tekapo B Power Station tailrace weir and chute – condition assessment and review of bathymetric survey data" dated 27 February 2026

1. Background

Genesis understands that the Damwatch memorandum was commissioned by Meridian in support of Meridian's proposed drawdown operations of Lake Pūkaki. Genesis' opportunity to review and comment on the memorandum arises from the conditions agreed between the parties in connection with Meridian's request for access to Genesis infrastructure and associated data relating to the Tekapo Scheme. As part of those arrangements, Meridian agreed that draft reports prepared using information obtained through that access would be provided to Genesis for comment prior to finalisation, with Genesis' comments appended to the final report to ensure transparency and factual completeness. The comments below are provided on that basis.

2. Hydraulic conclusion wording

During verbal discussion between Andrew Balme and Brent Willson on 04/03/2026, and as confirmed in an email from Brent Wilson 05/03/2026, Meridan have advised there are changes to be made to the version of the memorandum shared with Genesis following Meridians own review. Genesis appreciates Brent outlining these proposed changes.

Of note was a fundamental change in the conclusion drawn around the anticipated hydraulic performance of rock lining of the temporary tailrace chute.

The memorandum, as provided to Genesis, makes the following statements:

a. Footnote 4, page 5. *"It should be noted that Damwatch (2025a) and Damwatch 2025b) concluded the rock riprap lining material in bays 1-4* ***should*** *remain stable under the highly aerated ramp-type flow conditions which develop if the level of Lake Pukaki is drawn down to RL 513 m and Tekapo B Power Station discharges up to 115 m3/s."*

b. Section 6.2, page 9. "*The past hydraulic performance of the tailrace weir and chute is fully documented in Damwatch (2025a) and Damwatch (2025b). These reports concluded that the rock riprap lining material in bays 1-4* ***should*** *remain stabl*e"

Meridian has advised that the two occurrences of "should" in the statements (underlined for clarity) above are to be changed to "will" with regards to anticipated performance of the rock riprap lining material. The change from "should" to "will" represents a materially stronger statement of certainty regarding the anticipated hydraulic performance of the structure.

Further, this appears inconsistent with conclusions in the referenced reports:

a. Damwatch 2025a concluded *"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."*

b. Damwatch 2025b states "*the original conclusion in the main report about lining stability of the rock chute can be extended to the additional Tekapo b Power Station discharge case of 115 m^3/s."*

   yet then is followed by the statement

   "The tailrace chute ***will*** *remain stable under Tekapo B Power Station discharges up to 115 m3/s if the level of Lake Pūkaki is drawn down to RL 513 m."*

   The latter represents a stronger conclusion than the former.

It is not clear to Genesis what the engineering rationale is for this change from a probabilistic ("should") to deterministic ("will") conclusion. Nor is such rationale set out in the report.

Genesis' concern arises from the potential reliance on the temporary tailrace structure to accommodate drawdown operations of Lake Pūkaki within the Tekapo Scheme, as contemplated under the current application. Genesis notes that the current application does not appear to specify operational limits governing the frequency, duration, or notification of drawdown events. In considering the implications of the application as drafted, Genesis must therefore assess the potential reliance on the temporary tailrace structure under the range of operating conditions that could be permitted in the absence of such constraints.

For completeness of the technical record, it would be helpful for the memorandum to clarify the assumed operating conditions and the extent of reliance on the temporary tailrace structure under which the assessment has been undertaken. In this context, the different between should and will is fundamental to understand. In this context, the distinction between "should" and "will" is material and requires clear technical justification.

3. **Riprap reinstatement interpretation**

It became apparent during the verbal discussion 04/03/2026 that the following statement (Section 6.3, page 9) was open to interpretation:

> *"Additionally, we recommend to reinstate riprap in any areas where the riprap has been displacement and the channel bed has been exposed (i.e., areas where the bed is vulnerable to erosion under free flow conditions)."*

The dive inspection report contained in the appendices of the memorandum appears to show areas of riprap displacement and exposed channel bed, hence it was interpreted by Genesis that this statement recommended reinstatement prior to operation.

Meridian advised this is not the intention of this statement and that it will be revised. The rational for this change is unclear given the evidence to the contrary.

Genesis notes that the memorandum currently contains an explicit recommendation for reinstatement where riprap displacement and exposed channel bed are observed. If this is not intended to imply reinstatement prior to operation, clarification of the intended meaning in the final memorandum would be helpful.

4. Operating regime assumptions

Genesis notes that the memorandum assesses the structure as suitable for temporary use.

Genesis further notes that Meridian's current application does not appear to specify operational constraints governing reliance on the temporary tailrace structure.

The implications of repeated or extended reliance on the temporary tailrace structure, including the potential frequency and duration of drawdown events, do not appear to have been assessed in the memorandum and would benefit from clarification.

Genesis understands that the conclusions in the memorandum regarding the anticipated hydraulic and structural performance of the temporary tailrace structure are based on assumptions regarding the nature and duration of its use. For completeness of the technical record, it would be helpful for the memorandum to clearly state the assumed operating conditions under which the assessment has been undertaken.

5. Technical record

Genesis notes that the assessment and conclusions presented in the memorandum have been prepared by Damwatch for Meridian in relation to Meridian's proposed operation of Lake Pūkaki.

Genesis' comments above are provided to assist clarification of the technical assumptions and conclusions presented. They should not be interpreted as Genesis endorsing the conclusions reached in the memorandum, or relying on them for operational decision-making.

END OF COMMENTS

Genesis Energy