Before a panel appointed under the
Fast-Track Approvals Act 2024 **FTAA-2510-1120**

UNDER: the Fast-track Approvals Act 2024 (**Act**)

IN THE MATTER: an application for approvals for the Lake Pūkaki Hydro Storage and Dam Resilience Works

BY: **MERIDIAN ENERGY LIMITED**
Applicant

STATEMENT OF EVIDENCE OF MURRAY GRANT WEBBY ON BEHALF OF MERIDIAN ENERGY LIMITED

TEKAPO B POWER STATION TAILRACE WEIR AND ROCK CHUTE

Dated: 15 April 2026

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INTRODUCTION

1 My full name is Dr Murray Grant Webby.

2 I hold a Bachelor of Engineering (1st Class Honours) degree in Civil Engineering, which I obtained in 1978. I also hold a Doctor of Philosophy degree in Civil Engineering, which I obtained in 1981. Both degrees were from the University of Canterbury.

3 I am a Chartered Professional Engineer and a Recognised Engineer (Potential Impact Classification & Dam Safety Assurance Programme). I am a Fellow of Engineering New Zealand and a member of the NZ Society on Large Dams, the NZ Hydrological Society, the Engineering NZ / Water NZ Rivers Group and the International Association for Hydro-Environment Engineering and Research.

4 I am currently employed by Damwatch Engineering Limited where I hold the position of Senior Principal Civil Engineer (Water Resources). I have worked for Damwatch Engineering Limited since June 2016. Prior to my current position, I held the position of Principal Hydraulic Engineer with Opus International Consultants Limited. I was employed by Opus International Consultants Limited and its predecessor organisations, Works Consultancy Services Limited, Works and Development Services Corporation Limited and the Ministry of Works and Development, for 35 years.

5 I have nearly 45 years' experience as a hydraulic engineer, working for power generation companies in New Zealand and overseas, New Zealand government departments and agencies, regional and district councils, and private companies. This includes the analysis and design of several rock ramp and weir type structures on rivers and flood conveyance facilities for dams.

6 I have been asked by Meridian Energy Limited (Meridian) to provide a response to specific matters within my area of expertise contained in the written comments on the application from persons invited by the Panel to comment under section 53 of the Act. These are:

a. Genesis Energy

b. WSP New Zealand on behalf of Genesis Energy

7 I have prepared this statement within the limited time available to me. Consequently, it is necessarily at a high level. I am able to provide a more

fulsome response ot the issues covered in this statement if the Panel requires further assistance from me.

CODE OF CONDUCT

8 I confirm that I have read the Code of Conduct for Expert Witnesses as contained in section 9 of the Environment Court Practice Note (2023), and have complied with it in preparing this evidence. I confirm the issues addressed in this evidence are within my area of expertise, and I have not omitted material facts known to me that might alter or detract from my evidence.

RESPONSE TO COMMENTS FROM WSP NEW ZEALAND

9 I have reviewed the expert evidence statement of Mark Groves which consists of three separate documents prepared for Genesis Energy by WSP New Zealand. These documents are:

a. Memorandum titled "Tekapo B Weir – Summary of Findings" from Jeremy Robertson of WSP NZ to Andrew Balme of Genesis Energy, dated 19 December 2025 (Final Revision 2).

b. Report titled "Tekapo B Submerged Weir Bathymetric Survey and Hydraulic Assessment", dated 19 December 2025 (Final Revision 3).

c. Memorandum titled "Tekapo B Power Station Submerged Weir – Damwatch Document Review" from Jeanette Tucker, Jan Stanway and Jeremy Robertson of WSP NZ to Andrew Balme of Genesis Energy.

10 My comments in response to the three documents appended to Mr Groves' statement of evidence cover several themes related to the hydraulic performance of the Tekapo B Power Station control weir and rock chute (hereafter referred to as the weir and rock chute structure). In my response comments on these themes, I refer to the following supporting documents prepared by Damwatch Engineering for Meridian Energy and which have been provided to the Panel.

a. Report titled "Tekapo B Power Station Temporary Tailrace Weir and Rock Chute: Hydraulic Review of Weir and Chute", dated 28 October 2025 (issue 3.1).

b. Memorandum titled "Tekapo B Power Station Temporary Tailrace Weir and Rock Chute: Hydraulic Review of Weir and Chute – Addendum Report" from Grant Webby of Damwatch Engineering to Brent Wilson of Meridian Energy, dated 5 December 2025.

c. Memorandum titled "Tekapo B Power Station tailrace weir and chute – condition assessment and review of bathymetyric survey data" from Viculp Lal, Dr Grant Webby and Dr Jayandra Shrestha of Damwatch Engineering to Brent Wilson of Meridian Energy, dated 18 March 2026.

WSP New Zealand have reviewed the first and second Damwatch documents in their document b in Paragraph 9. They have reviewed the third Damwatch document in their document c in Paragraph 9.

Potential Degradation of Structure

11 WSP NZ and Damwatch both developed separate computational hydraulic models of the Tekapo B Power Station tailrace channel to predict flow depths and velocities for various discharges and lake levels. The key difference between the two models was that each defined the tailrace channel geometry differently. In the WSP model, the geometry was defined by a recent bathymetric survey of the tailrace channel. In the Damwatch model, the geometry was defined by an as-built survey of the original tailrace channel dating from 1977. Despite this difference and the form of the equations of fluid motion solved by the two models, they appear to give very similar flow depth predictions down the chute for a discharge of 112 m^3/s.

12 In the WSP Summary Memorandum (WSP document a – Paragraph 9 above), WSP make a number of comments with respect to the use of the original as-built geometry of the tailrace channel in the Damwatch model:

a. *Having been submerged for most of its life, there is significant risk that the foundations are not in "As Built" condition, and (the structurre) cannot perform its original intended purpose, if exposed and returned to service*.

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

c. *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*.

13 In responding to these comments related to the potential degradation of the weir and chute structure from 47 years of submergence, it is useful to first consider the following description provided by Dr Stephen Thomspon and Peter Campbell of the Minstry of Works in their 1979 Journal of Hydraulic Research paper "Hydraulics of a Large Channel Paved with Boulders" (and referenced in the Damwatch documents a and c – Paragraph 10 above). This description stems from their extensive field programme to evaluate the hydraulic performance of the weir and chute structure after an initial one-month period of operation and the rock lining had stabilised.

> A comparison of vertical photographs taken just prior to the first flow and after one month's flow when the paving had stabilised, showed that about half of the area of the bed was occupied by groups of large stones that can be identified in both photographs. These groups had in places moved downstream by as much as 3 m. The reach between sills 1 and 2 was paved with larger stones than further downstream and these stones had not been significantly modified by the flow.
>
> Between sills 2 and 8 large areas of the stabilized paving consisted of a tight mosaic of stones measuring between .2 and .6 m across. The original paving had been modified by the flow to arrange this mosaic, and there had been an associated downstream drift of the surface layer causing a low patch immediately downstream of each sill and a "frown" over the upstream edge of each sill. The regular pattern in the water surface evident on Fig. 1 was caused by this drift.

14 Key points from this description are:

a. The initial operation of the weir and chute structure had caused movement of the rock lining to occur although the lining had stabilised.

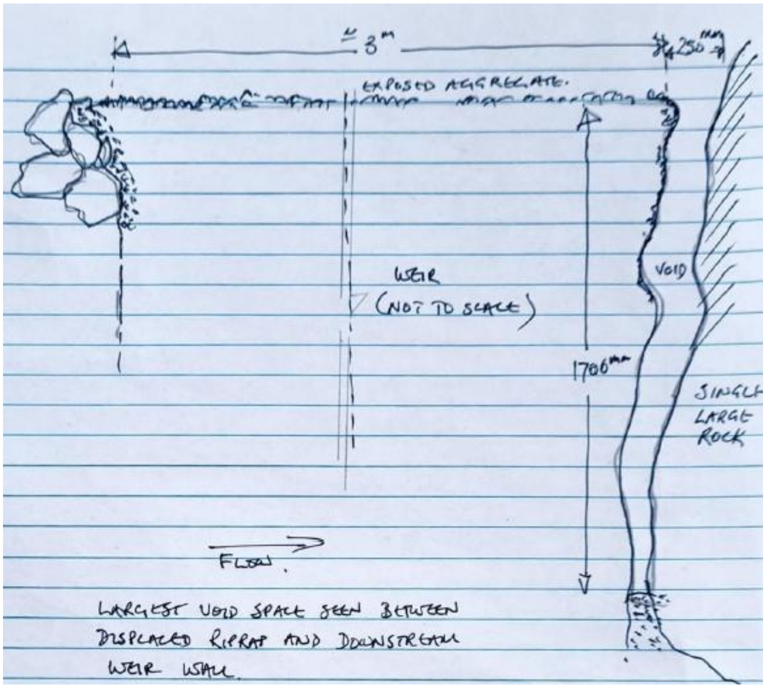
b. Less movement had occurred in bay 1 between sill 1 (the control weir at the upstream end of the structure) and sill 2 where much larger boulders formed the lining.

c. Between sills 2 and 8, the stabilised lining consisted of a "*tight mosaic of stones measuring between 0.2 m and 0.6 m across*".

d. The tight mosiac of boulders between sills 2 and 8 (bays 2 to 7) had been "*arranged*" by the flow and there had been an "*associated downdrift of the surface layer causing a low patch immediately downstream of each sill and a 'frown' over the upstream edge*".

15 Measurements by Thompson and Campbell in their 1977 field programme quantified the mean bed level along the centreline of the chute. They observed no significant deviations from the average slope of the chute although individual boulders or roughness elements did project above the mean bed level. The observed downdrift of the rock lining in each bay between sills 2 and 8 would have caused the effective bed slope in each bay to reduce slightly below the average bed slope. Thompson and Campbell noted that this downdrift and the associated frowning on the upstream side of sills 2 to 8 caused the regular longitudinal wavy pattern seen in the oblique aerial photograph of structure operation at 100 m^3/s discharge **in Figure 2.1 of Damwatch document a (Paragraph 10 above)**.

16 The gap between the control weir and rock lining in bay 1 observed in the recent dive inspection (refer to the sketch in **Figure 1** below - reproduced from **Figure 12 of Damwatch document c** (Paragraph 10 above)) most likely developed during the initial period of structure operation when stabilisation of the chute lining ocurred. Similar gaps could exist on the downstream side of other sills due to the “downdrift” behaviour of the chute lining in each bay described by Thompson and Campbell.

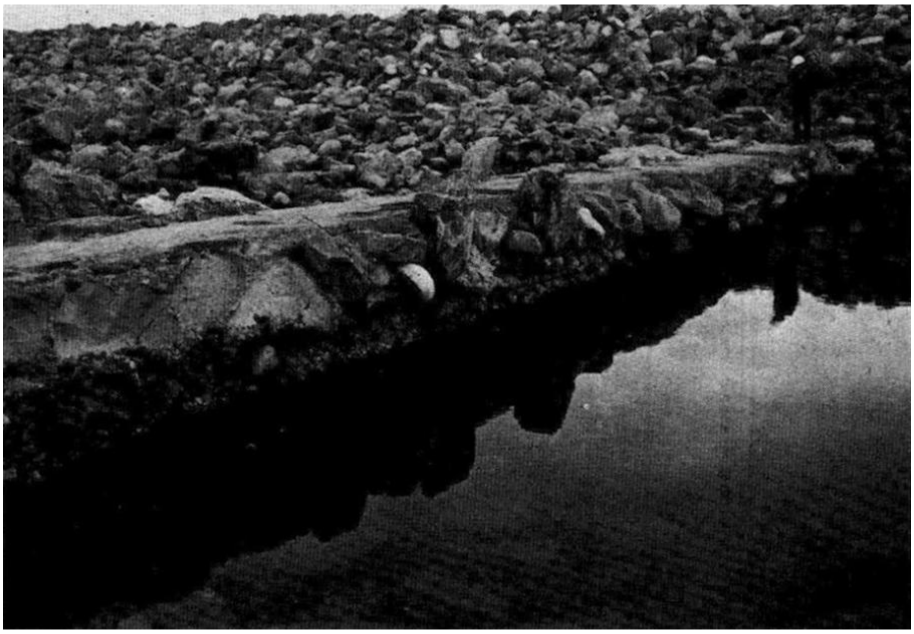


Source: (Divepro, 2026)

Note: Lift joint is approx. 1,435 mm deep, compared to diver’s observed depth of rib face approx. 1,700 mm deep

Figure 1: Diver’s sketch of void (gap) noted between concrete control weir and riprap

However, there is also other evidence of rock elements of the chute lining partially embedded within the concrete sill as seen in the in the photograph in Figure 2 below sourced from the 1979 technical note “Construction and Performance of the Rock-Lined Chute at Tekapo B Power Station”: by Ted Malan and Bruce Hancock of the Ministry of Works and published in the Proceedings of the Institution of Civil Engineers UK (referenced in Damwatch documents a and c – Paragraph 10 above),



Source: Malan and Hancock (1979)

Figure 2: Scour hole which formed downstream of sill 9 in July 1977 showing rock elements of chute lining partially embeded in concrete of sill

17 Despite the deposition of silt (glacial flour) on the structure from 47 years of submergence since December 1978, the high quality bathymetric images of the rock lining of the chute on the engineering drawings included in **Appendix A** (reproduced from Appendix C of Damwatch document c - Paragraph 10 above) clearly show that the tight mosiac of boulders described by Thompson and Campbell remains. While no attempt has been made to quantify the distribution of boulder sizes from the bathymetric survey images, the images clearly reflect the differences in the boulder size distributions from bay 1 down to bay 4 which were measured by Thompson and Campbell.

18 During its period of submergence over the 47 years, there are no external forces other than a significant seismic event which could have caused deformation of the weir and chute structure to have occurred. The effects of

wind-generated waves only occur at the lake surface and the shallowest part of the chute immediately downstream of the control weir (sill 1) incorporates the largest boulders. Currents induced by waves and lake circulation would be far too weak to cause movements of any boulders. These latter effects can therefore be excluded from contributing to any potential deformation of the structure.

19 WSP in their document a (Paragraph 9) note that the longitudinal bed profile of the chute shown in Figure 3-5 (and sourced from their bathymetric survey) exhibits a 1 m drop below the mean bed slope at about distance 140 m from control weir sill 1. This would place this apparent depression in the rock lining between sills 4 and 5 (refer to the long-sections sourced from the separate bathymetric survey carried out for Meridian Energy on **Drawing TEK-B\TW\CHT\PROFILE SECTION** in **Appendix A**.

20 The depression apparent from the WSP bathymetric survey does not show up in the longitudinal centreline profile of the chute on the Damwatch Drawing TEK-B\TW\CHT\PROFILE SECTION in Appendix C of Damwatch document c (Paragraph 10). However, a dip in the longitudinal right side profile in the same drawing does show up immediately downstream of sill 4. A clear patch also shows up in the same location in the bathymetric survey image of bay 4 bewteen sills 4 and 5 on **Drawing TEK-B\TW\CHT\BAY-4** in **Appendix A**.

21 Meridian plan to shortly undertake another dive inspection of the weir and chute structure when conditions allow to investigate these and other anomalies identified from the bathymetric survey imagery. Note that there are some lighter patches on the survey images of bay 3 and bay 4 on paper copies of **Drawings TEK-B\TW\CHT\BAY-3 and BAY-4** in **Appendix A** which appear to indicate an absence of boulders in the tightly packed mosaic of the rock lining. However, when viewed on a digital version of these drawings, it is possible to distinguish boulders within these patches. It may be that the boulders in these patches are masked to some extent by deposited sediment.

22 In the same planned dive inspection, Meridian have also instructed the divers to inspect sills 2, 3, 4 and 5 for any signs of damage or deformation due to foundation subsidence.

23 In summary, the visual evidence available from the bathymetric survey imagery and the dive inspection does not indicate any visible evidence of degradation of the structure apart from corrosion of external steel components

used as formwork for the capping part of the concrete sills. The tight mosaic of the boulders forming the rock lining of the chute between sills 2 and 8 and described by Thompson and Campbell in their 1979 paper appears to be intact with deposited sediment covering it. The sizes of the boulders in bay 1 between sill 1 (the control weir) and sill 2 are too large for significant movement of the rock lining to have occurred when the structure was operated from June 1977 to December 1978.

24 The conclusion about the lack of any visible evidence of degradation of the structure is supported by the visual observation from the dive inspection of the control weir that one of the timber gauge boards fitted to the sloping sides of the weir for the 1977 field measurement programme remains intact apart from some minor damage and that the other gauge board which had detached "*appeared as new*" with the painted depth scale still clearly visible (Section 4.2.1 of Damwatch document c – Paragraph 10 above).

Stability of the Chute Lining

25 In Section 3.2.4 of WSP document b (Paragraph 9 above), WSP refer to a couple of empirical engineering design formulae sourced from the technical literature for determining stable rock sizes for rock ramps in riverine and dam situations. These formulae predicted a similar rock size of about 0.6 m with a factor of safety of 1.3 for a ramp with a 1 in 20 slope and a design discharge assumed to be 112.5 m^3/s (although this is not stated in Section 3.2.4 of the WSP document).

26 In responding to this comment and a similar comment in WSP document c (Paragraph 9 above), it is again helpful to consider some of the statements made by Thompson and Campbell in their 1979 paper. In introducing their investigations, they stated:

> It was feared that the flow might be supercritical with a violent dissipation zone where the channel met the lake, leading to channel failure from the downstream end. Alternatively it was feared that subcritical flow with large depth and consequently large shear stresses would cause failure of the weaker parts. The flow proved to be sub-critical throughout with depths in the range 0.5 m to 1.5 m. Dissipation appeared to be distributed evenly over the whole channel.

27 It was concern on the part of the designers of the weir and chute structure which motivated the detailed field measurement programme of the hydraulic performance of the structure which Thompson and Campbell then proceeded to describe in their paper. As noted in Damwatch documents a and b

(Paragraph 10 above), the flow was found to be sub-critical across all trial discharges with energy dissipation evenly distributed across the whole channel.

28 Thompson and Campbell note that application of the Shields critical entrainment function (referenced in most standard open channel flow and sedimentation engineering text books) for estimating the smallest uniform gravel particle on a gravel riverbed that will not be entrained by a water flow passing over it indicates that the boulder size for a stable lining on the weir and chute structure would need to be at least 0.62 m in diameter (similar to the size that WSP estimated with a factor of safety of 1.3 applied). However, they then observed that, from their measured boulder size distributions on the structure, "*about 60% of the channel area is paved with boulders smaller than this*", although "*the paving did not stabilise until after a considerable amount of movement had occurred*".

29 It was this finding that motivated Thompson and Campbell to then explore using the Ishbash criterion for evaluating "*the diameter of the smallest boulder that will not move when dropped though running water onto a stable rock fill of similar boulders*". Their application of this criterion indicated a boulder size of larger than 0.25 m for stability under the largest flow discharged through the tailrace channel up till then (140 m^3/s). They still noted that "*yet 30% of the channel area is paved with boulders smaller than this, although nowhere is the median smaller than 0.25 m*".

30 In respect of the stability of the rock lining on the weir and chute structure, the true test of this is not the application of empirical design equations but the performance of the structure under sustained operation with discharges up to 140 m^3/s. As discussed in Section 5.3 of Damwatch document c (Paragraph 10 above), apart from the two scour failures which occurred in the first month of operation of the structure and which were successfully repaired, the structure functioned satisfactorily without any further damage being detected for 11 months from January to November 1978. It should be noted that Genesis have advised that the maximum discharge through Tekapo B Power Station at the net head between the headpond and the minimum tailrace level of RL 518 m is 112 m^3/s.

31 The proven historic stability of the weir and chute structure is due to the combination of the concrete stills reinforcing it (which empirical equations for

the design of rock ramps do not consider) and the rearrangement of the rock lining into a tightly packed mosaic of boulders by the initial flows passed down the structure.

32 The experimental research by Pagliara and Chiavaccini (2007) on the reinforcement of rock ramps with larger boulders either in rows, randomly placed, or in an arc configuration referenced in Section 5.2 of Damwatch document c (Paragraph 10 above) clearly demonstrated the enhanced stability of the base matrix forming a rock ramp functioning as a grade stabilisation and energy dissspation structure. The same conclusion about enhanced stability can be extended to rock ramp structures fitted with concrete sills as in the context of the Tekapo B Power Station tailrace weir and chute structure.

Potential Failure Modes

33 In the three WSP NZ documents referenced in Paragraph 9 above, several potential failure modes for the weir and chute structure are postulated. These include:

a. Potential instability of the structure foundation

b. Potential scour failure of the chute lining initiated by hydraulic jump formation

c. Global instability of the chute lining

d. Potential structural failure of a sill cap

Potential instability of the structure foundation

34 WSP NZ have raised potential destabilisation of the gravel and sand filter layers from injection of water flow through the gap between the downstream side of each concrete sill and the rock lining in the downstream bay of the chute structure (Section 6.2.1 of WSP document c – Paragraph 9 above).

35 As discussed in Paragraph 16 above, any gap between the downstream side of each concrete sill and the rock lining in the downstream bay of the chute structure is likely to have formed during the initial period of operation of the weir and chute structure when stabilisation of the rock lining occurred with rearrangement of the lining into a tight packed mosaic of boulders. With the structure having performed satisfactorily as an energy dissipation facility over

a period of 11 months of continuous operation in 1978 and these gaps having likely developed in the initial period of operation, if potential destabilisation of the gravel and sand filter layers underneath the rock lining was a credible failure mode, then it would have occurred in that 11 month period of continuous operation in 1978, or earlier. The fact that no destabilisation of the foundation layers was observed during the operation of the structure after the gaps between the downstream side of each concrete sill and the rock lining in the downstream bay of the chute structure are likely to have developed suggests that this failure mode is very unlikely to occur during any future operation of the structure.

36 The maximum flow velocities down the chute structure at a maximum discharge of 115 m^3/s are estimated to be less than 3 m/s. In my opinion, this flow velocity is not likely to be large enough to result in injection of water flow into any gap between the downstream side of each concrete sill and the rock lining in the downstream bay of the chute structure, and thereby destabilising the gravel and sand filter layers underneath the rock lining. This potential failure scenario is completely different from the injection of water flow and high hydrodynamic pressures through a narrow gap between slabs on a dam spillway into the spillway foundation with flow velocities upwards of 20 m/s which can lead to the uplift and failure of spillway slabs[1] [2].

37 The entrainment of finer gravel material in a steep mountain stream lined with large boulders by high velocity water flows of similar magnitude to the maximum flow velocity on the weir and chute structure is severely constrained by the hiding effect of the large stones[3] and must be taken into account when estimating bedload sediment transport volumes.

38 Destabilisation of the foundation gravel and sand filter layers on the weir and chute structure due to injection of water flow through the gaps between the downstream side of each concrete sill and the rock lining in the downstream bay, as a potential failure mode is therefore not seen as credible.

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[1] Wahl, T L and Heiner, B J (2024a). "Laboratory Measurements of Hydraulic Jacking Uplift Pressure at Offset Joints and Cracks". *ASCE Journal of Hydraulic Engineering*, Vol, 150(4), Paper Ref. 04024016.

[2] Wahl, T L and Heiner, B J (2024b). "Discharge Through Open Joints and Cracks in Spillway Chutes". *ASCE Journal of Hydraulic Engineering*, Vol, 150(5), Paper Ref. 04024032.

[3] Parker, G (2008). "Transport of Gravel and Sediment Mixtures". Chapter 3 in *Sedimentation Engineering: Processes, Measurements, Modelling and Practice*, ASCE Manuals and Reports on Engineering Practice No. 110, ed. M H Garcia, prepared by ASCE Task Committee to Expand and Update Manual 54 of the Sedimentation Committee of the Environmental and Water Resources Institute, pub. by the American Society of Civil Engineers.

39 In respect of potential instability of the structure foundation, WSP document a (Paragraph 9 above) refers to several springs which were encountered during construction of the weir (Section 2, second bullet point). These springs are described in the 1979 technical note “Construction and Performance of the Rock-Lined Chute at Tekapo B Power Station”: by Ted Malan and Bruce Hancock of the Ministry of Works and published in the Proceedings of the Institution of Civil Engineers UK (referenced in Damwatch documents a and c – Paragraph 10 above).

40 The springs encountered during weir and chute construction occurred between sills 9 and 11. These sills, with approximate invert levels of RL 505.2 m and RL 502.2 m respectively, are well below the minimum RL 513 m level which Lake Pukaki could be drawn down to in the next 3 years under Meridian’s proposal. Foundation instability due to reactivation of these springs is therefore not a credible failure mode under Meridian’s proposal.

Potential scour failure of the chute lining by hydraulic jump formation

41 The WSP NZ documents referenced in Paragraph 9 allude several times to the potential for hydraulic jumps to form on the rock-lined chute, thereby causing either a local scour failure leading to global instability of the rock lining or unravelling of the rock lining of the chute from the bottom end of the structure.

42 As described by Thompson and Campbell in their 1979 Journal of Hydraulic Research paper referenced in Damwatch documents a and c (Paragraph 10 above), energy dissipation down the length of the weir and chute structure appeared to be “*distributed evenly over the whole structure*”. The flow behaviour down the structure at a flow of 100 m^3/s is illustrated by the oblique aerial photo shown in **Figure 2.1 of Damwatch document a**. This photo and the close-up photo of the aerated water flow in **Figure 3.4** of the same report (both photos are also included in Thompson and Campbell’s 1979 paper) give no hint of any hydraulic jump formation at a flow of 100 m^3/s. The aerated water flow at the bottom end of the structure in the photo in **Figure 2.1 of Damwatch document a** can be seen to enter smoothly into Lake Pukaki also without any evidence of hydraulic jump formation.

43 The latter observation is particularly pertinent because the original designers of the weir and chute structure were concerned that supercritical flow could occur with discharges down the structure with the consequent formation of a

hydraulic jump at the entry point into Lake Pukaki. As demonstated through the comprehensive field measurement programme reported by Thompson and Campbell in their 1979 Journal of Hydraulic Research paper, the flow regime on the chute structure during all flow trials up to 140 m^3/s discharge was determined to be sub-critical. Under such a flow regime, hydraulic jumps do not form.

44 As noted in Section 5.3 of Damwatch document c (Paragraph 10 above), two scour holes formed within the first month of operation of the weir and chute structure in 1977 (although these were subsequently repaired and the structure functioned satisfactorily as an energy dissipation facility under continuous operation throughout 1978 without any further damage). Thompson and Campbell in their 1979 Journal of Hydraulic Research paper observed that, when the scour failure at sill 9 developed, the removal of gravel material from the eroded scour hole "*seemed to be greatest at low flows, when the difference in head across the roughness projections was greatest*". Under such conditions a very localised hydraulic jump could have occurred.

45 If the rock lining on the weir and chute structure remains intact under any future operation of the structure with maximum discharges up to 115 m^3/s, the potential for localised hydraulic jump formation leading to the occurrence of a local scour failure of the rock lining is very unlikely. The historic performance of the weir and chute structure during the 11-month period of operation in 1978 after the successful repair of the initial scour failures in 1977 demonstrates that the rock lining is very likely to remain intact.

Global instability of the chute lining

46 Global instability as a potential failure mode for a rock chute involves the plucking of individual boulder elements from the lining of the rock chute when the threshold for riprap liner instability is exceeded and progressive enlargement of the "hole" formed in the lining through erosion of further boulder elements occurs. The failure mechanism is similar to that resulting from local hydraulic jump occurrence although it does not need a local hydraulic jump to form. Section 5 of Damwatch document c (Paragraph 10 above) examines this potential failure mode.

47 Section 5.2 of Damwatch document c (Paragraph 10 above) discusses the global instability of rock ramp structure reinforced with boulders larger than the base rock material forming the ramp based on experimental research by

Pagliara and Chiavaccini (2007) (referenced in Damwatch document c). **Figure 4 in Damwatch document c** (sourced from the Pagliara and Chiavaccini (2007) paper) illustrates the failure mechanisms for different large rock reinforcement configurations. Based on the findings of this experimental research, Section 5.2 concludes that the concrete sills on the Tekapo B Power Station tailrace weir and chute structure function as reinforcement features and would limit the progression of any global failure of the rock lining of the chute if the threshold flow for riprap instability was exceeded.

48 Section 5.4 of Damwatch document c concludes that, based on considerations of the historic performance of the weir and chute structure in 1978 prior to submergence by Lake Pukaki and the visual evidence from the bathymetric survey and dive inspection of the structure, the likelihood of global instability of the of the rock riprap lining is extremely low as long as the flow threshold for riprap instability is not exceeded. Based on the findings of the comprehensive field programme of measurements and observations of the hydraulic performance of the weir and chute structure reported by Thompson and Campbell in their 1979 Journal of Hydraulic Research paper, this flow threshold was assessed to be larger than 115 m^3/s.

Potential failure of sill cap

49 WSP NZ in their document c (Paragraph 9 above) postulate a number of mechanisms for potential degradation or deterioration of the concrete sills on the weir and chute structure. The credibility of most of these mechanisms is addressed by my colleague Mr Viculp Lal in his statement of evidence. I address here the potential failure of a sill cap on one or more of the concrete sills (Section 6.2.2 of WSP document c – Paragraph 9 above).

50 In Section 6.2.2 of WSP document c, WSP postulate the potential detachment of the cap of one of the concrete sills (the sills were constructed in two stages as described in the Damwatch document c – Paragraph 10 above) under the hydraulic loading conditions which would occur in future operation of the weir and chute structure. The potential failure mechanism is illustrated in **Figure 2** of the WSP document c.

51 For this failure mechansim to develop, all dowel bars providing connectivity between the top (cap) and bottom parts of each concrete sill would need to have been completely corroded and the bond across the joint between the two parts would need to be completely broken. My colleague Mr Lal's view on this

in Section 6.2.1 of the Damwatch document c is that this condition is extremely unlikely to be the case, even after 47 years of submergence of the weir and chute structure.

52 The lateral hydraulic-related loadings on each sill from the passing water flows include the bed shear stress exerted on the crest of each sill, a drag force exterted on part of the upstream face of each sill cap, and a lateral force transferred from the rock lining in the upstream bay to the upstream face of each sill due to the bed shear stress exerted on the rock lining.

53 WSP have made no attempt to quantify the magnitude of these lateral forces on a concrete sill cap relative to the base friction induced by the submerged weight of the cap and to demonstrate the credibility or otherwise of this potential failure mode.

54 Even if this failure mechanism was to develop (it would require the assumptions of total corrosion of all steel dowel bars linking the top and bottom sill parts and total breakage of the bond across the joint between the two sill parts to be realised), there is no certainty that it would then lead to failure of the rock lining in the downstream bay of the chute. Frictional resistance at the construction joints would resist lateral forces exerted by hydraulic actions and, depending on the magnitude of the lateral forces and frictional resistance, the water flows may push the sill cap slightly downstream leading to a localised flow disturbance at the sill location on the chute. However, this would not necessarily lead to exposure of the foundation under the rock lining and initiation of global instability of the lining.

55 WSP NZ have not yet demonstrated the credibility of this potential failure mode.

Likely Conditions of Future Operation of the Structure

56 Section 3.1 of Meridian's substantive application for fast-track consent outlines modelling undertaken using historic hydrological inflow sequences to assess how frequently Lake Pukaki is likely to fall below RL 518 m in the period 2026 to 2028 and the likely duration of these low-level events. The modelling indicates that lake levels below this theshold are expected to be relatively infrequent over this three year period.

57 Should the lake level fall below RL 518 m, the original tailrace channel would be reactivated assuming Tekapo B Power Station remains operable. The control weir on the weir and chute structure would continue to maintain water levels in the upstream tailrace channel above RL 518 m, thereby enabling Tekapo B Power Statioon to operate.

58 In a worst-case scenario where Lake Pukaki is drawn down to approximately RL 513 m, only the upper portion of the weir and chute structure would be exposed and function as an energy dissipation facility. This would extend down to around sill 4 with an invert level of approximately RL 513.14 m.

Suitability of Structure for Future Use

59 Subject to the findings of further assessments of the potential instability of the structure foundation and potential deterioration of the gravel and sand filter layers underneath the chute lining discussed in Paragraph 34 above, and the findings of the additional dive inspection Meridian plans to undertake, Damwatch remains of the view that the existing tailrace weir and chute structure is suitable for future use on a short-term basis if Lake Pukaki is required to be drawn down below RL 518 m to provide water for hydro-generation purposes.

60 As with the original operation of the structure in 1977-1978, it is acknowledged that there are risks associated with future operation. It would be prudent to apply the risk mitigation measures adopted previously as discussed in Section 7.3 of Damwatch document c (Paragraph 10 above):

a. Continued monitoring of the structure during any future operation.

b. The maintenance of stockpiles of rock riprap and gravel material to enable rapid repair of any damage to the structure that might occur during any future operation.

c. Having suitable plant readily available to undertake these repairs.

61 These risk mitigation actions are intended as a precautionary and reactive measure for implementation during future operation of the weir and chute structure should damage occur. The actions are not intended to imply pro-active reinstatement of areas of the chute lining that are perceived or found from future inspections to be deficient in provding adequate protection or pro-active repair of damage to concrete elements.

62 In the unlikely event that a repair to the weir and chute structure needs to be implemented during any future operation, the 1977 experience with the repair of the second more serious scour hole which developed in bay 9 and under sill 9 provides a useful reference for the magnitude and time scale of repair effort. As described in Section 5.3 of Damwatch document c (Paragraph 10 above), when the scour hole measuring 24 m in diameter and 5 m deep was first detected during a routine inspection when Tekapo B Power Station was offline for minor machine repairs, The power station was allowed to resume operation at half-load overnight, discharging about 61 m^3/s. The scour hole was repaired in 8 hours the next day with 1650 m^3 of gravel fill material and 665 m^3 of riprap material before Tekapo B Power Station was allowed to return to full service again.

63 To complement the second and third risk mitigation measures listed in Paragraph 60, appropriate construction planning should be undertaken beforehand to facilitate rapid implementation of a damage repair plan should this prove necessary in any future short-term operation of the weir and chute structure.

Conclusion

64 Based on the documented hydraulic performance of the weir and chute structure, its operational history over an 11 month period in 1978 without requiring any repairs, and its current condition as indicated by a bathymetric survey and a dive inspection, it is anticipated that the structure would provide the same functionality for any future short-term use as it did back in 1977-1978 prior to submergence by Lake Pukaki.

65 If any damage to the rock lining did occur during any future short-term use of the weir and chute structure, based on the experience of repairing scour holes in 1977, any repairs should be able to be rapidly completed within one to two days at most, as long as the risk mitigation measures outlined in Paragraphs 60 and 63 are followed. The potential for a 6-month outage of Tekapo B Power Station due to damage to the structure is extremely unlikely. The nature of construction of the structure facilitates the rapid implementation of repairs as evidenced by the successful 1977 repairs to two scour holes which formed within the first month of operation.

Dated: 15 April 2026

Murray Grant Webby

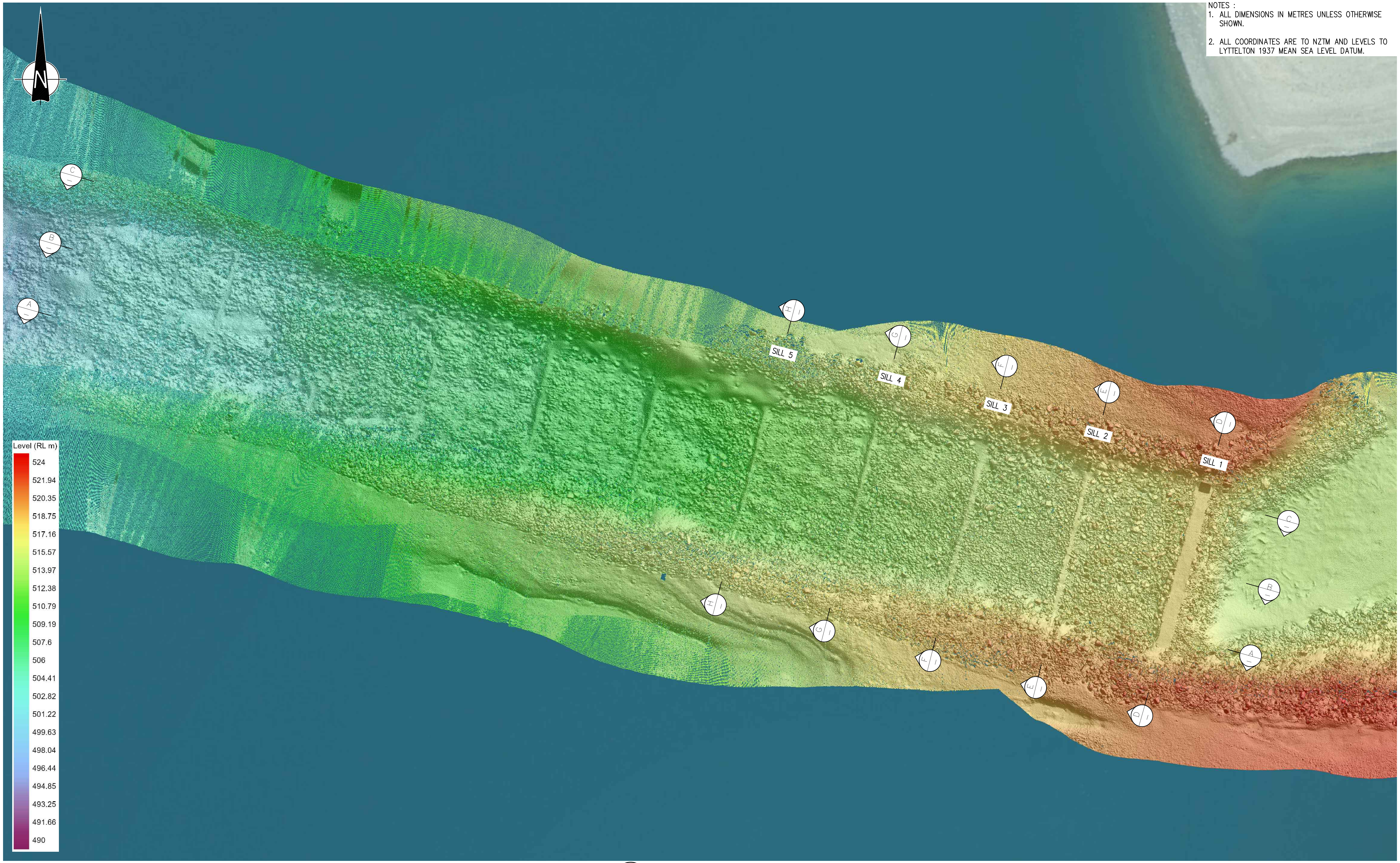
APPENDIX A

TEKAPO B POWER STATION TAILRACE WEIR AND CHUTE: BATHYMETRIC SURVEY DRAWINGS

(reproduced from Appendix C of Damwatch document c – Paragraph 10)

01 02 03 04 05 06 07 08 09 10 11 12 13 14 15 16 17 18 19 20

A B C D E F G H J K L M N



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

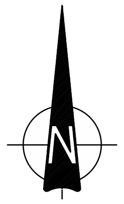
CONDITION ASSESSMENT
LAYOUT PLAN

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

01 02 03 04 05 06 07 08 09 10 11 12 13 14 15 16 17 18 19 20

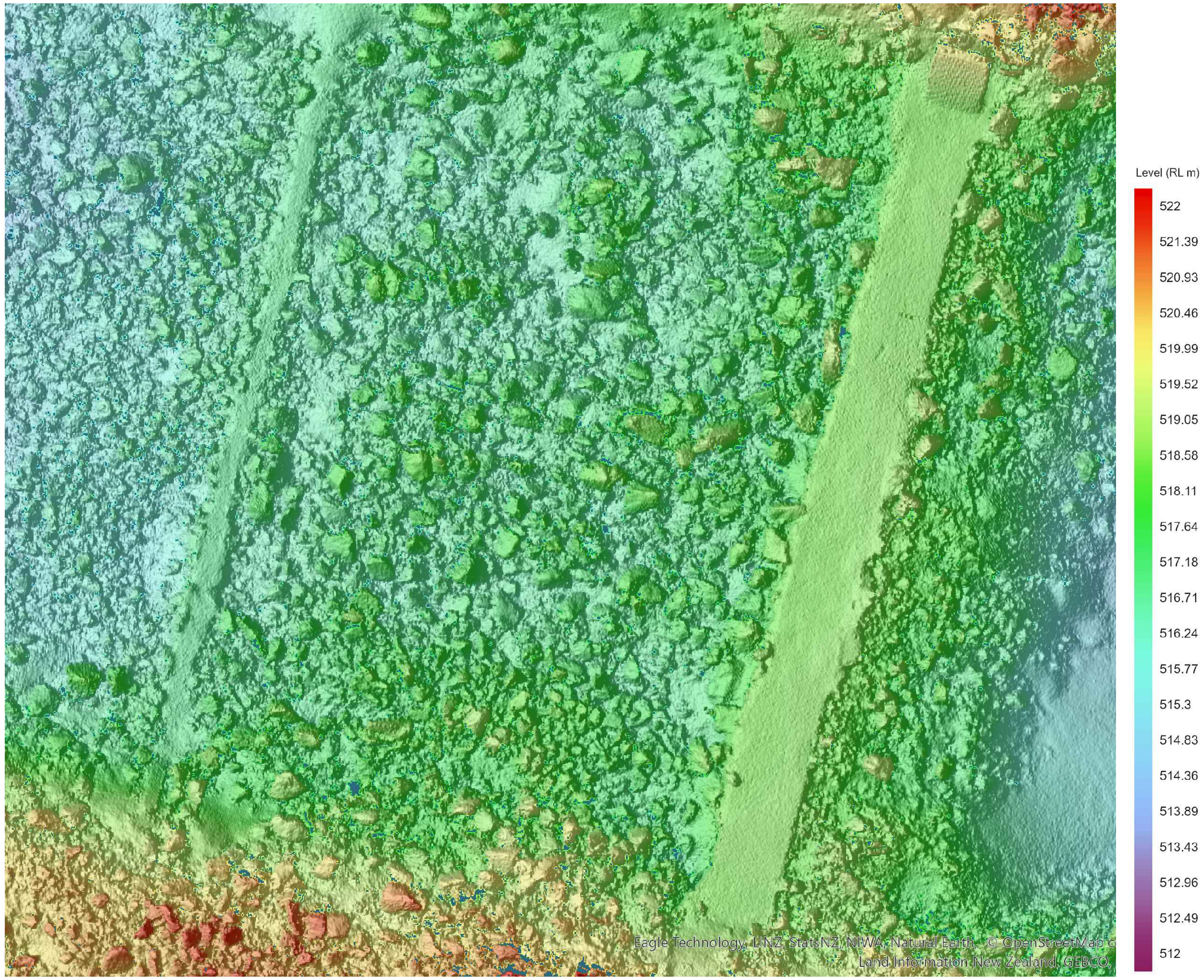
A B C D E F G H J K L M N

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 2

SILL 1



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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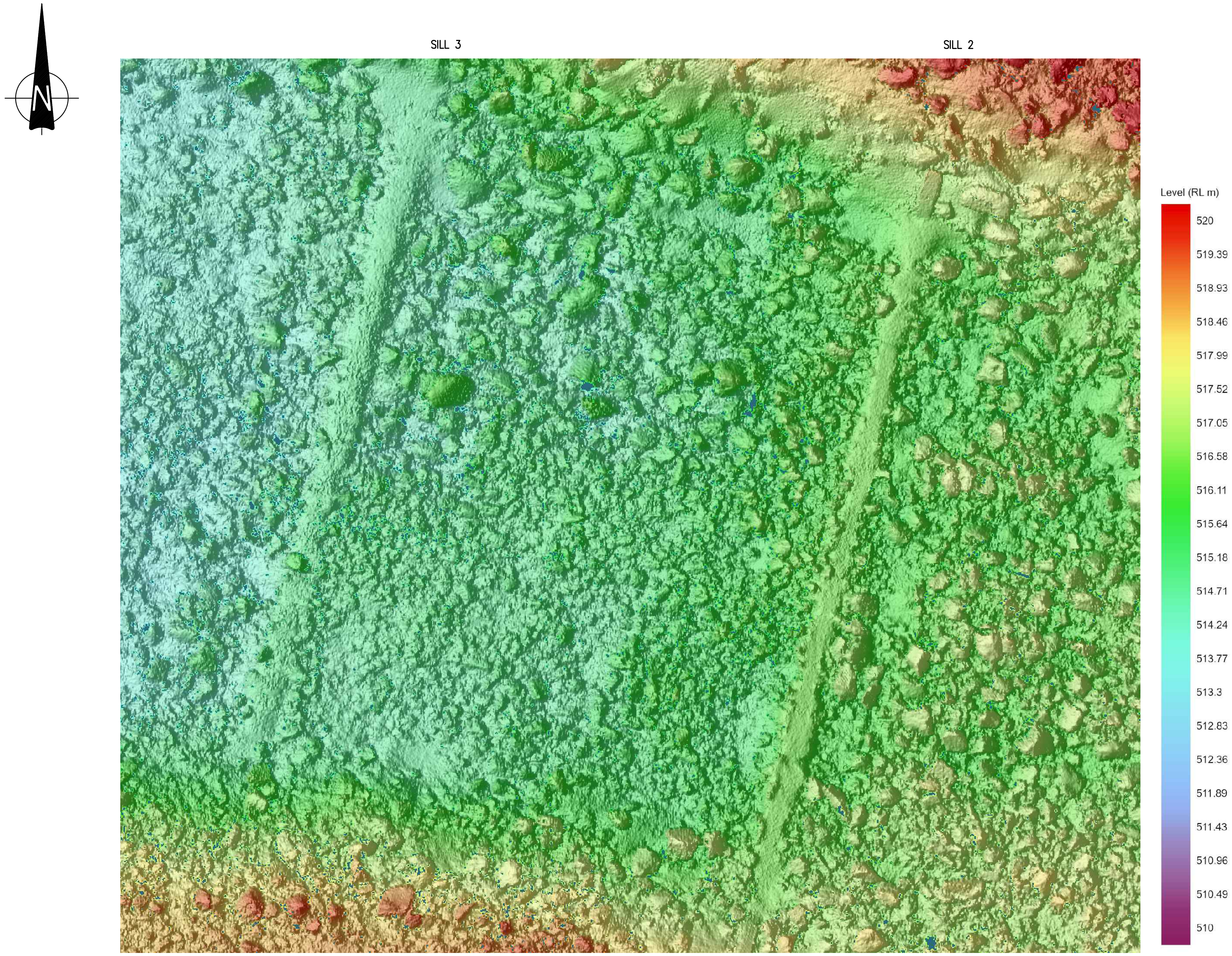
TEKAPO B PS TAILRACE WEIR

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
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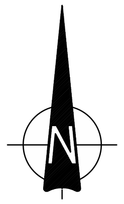
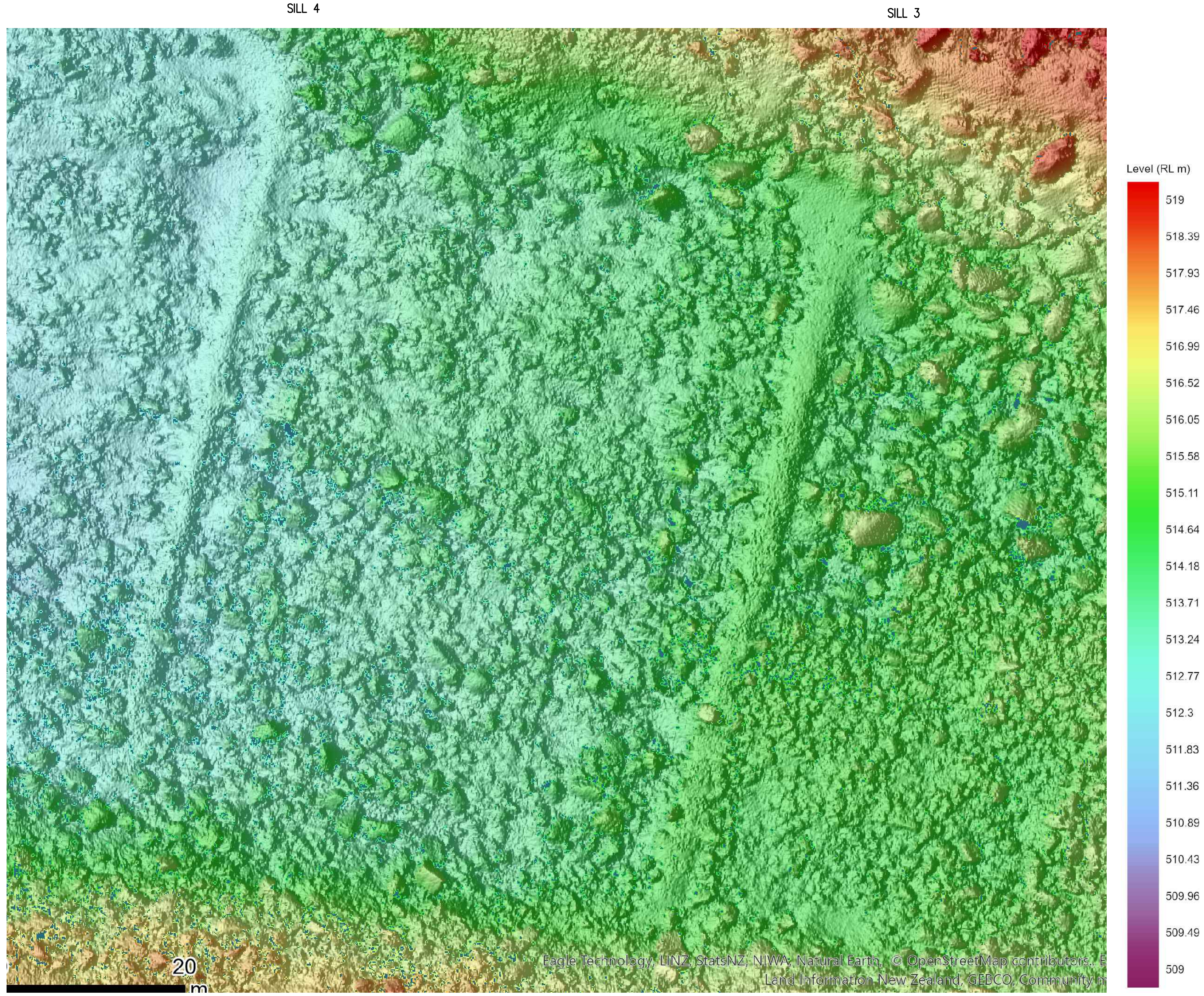
TEKAPO B PS TAILRACE WEIR

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.





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

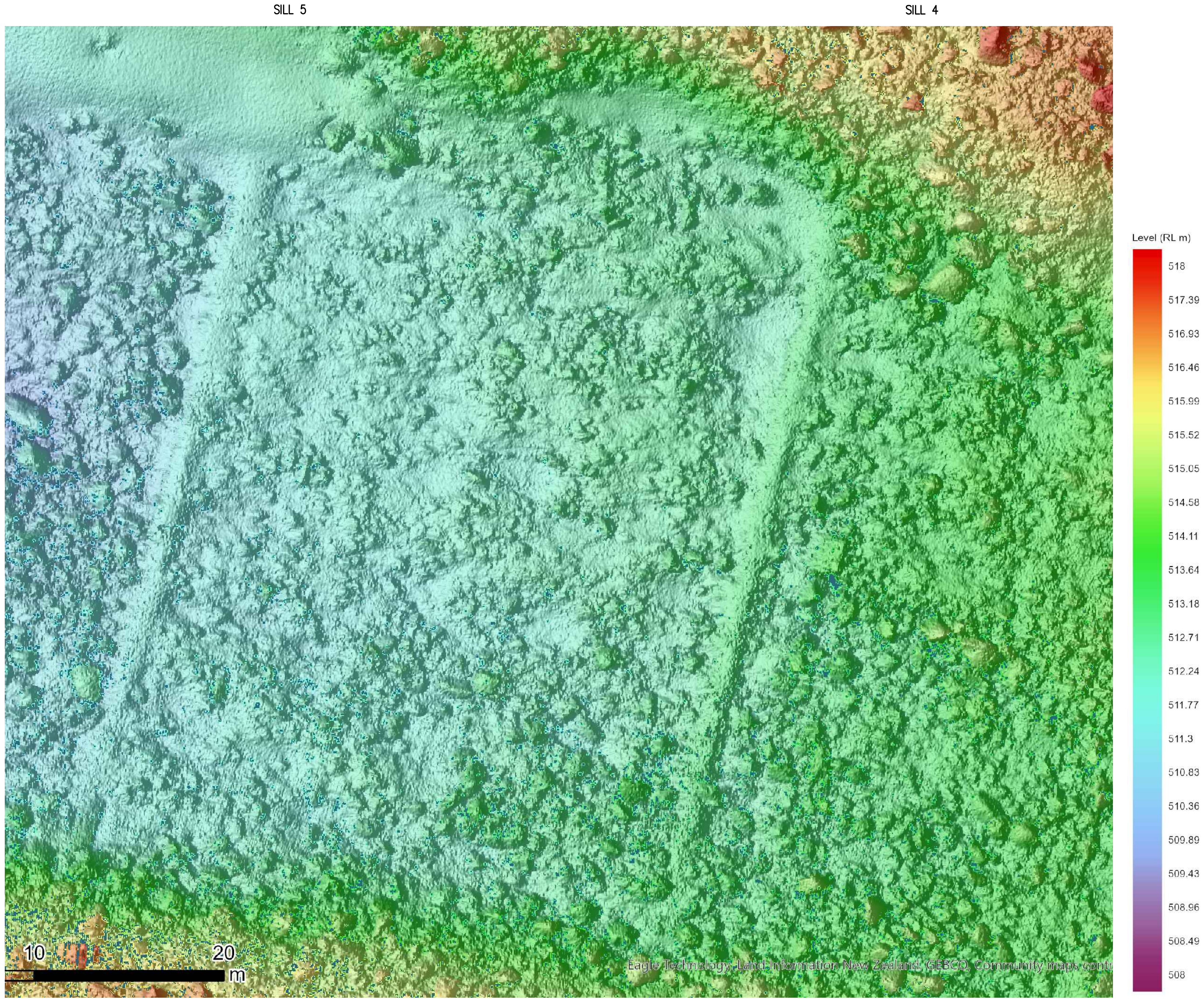
Meridian.

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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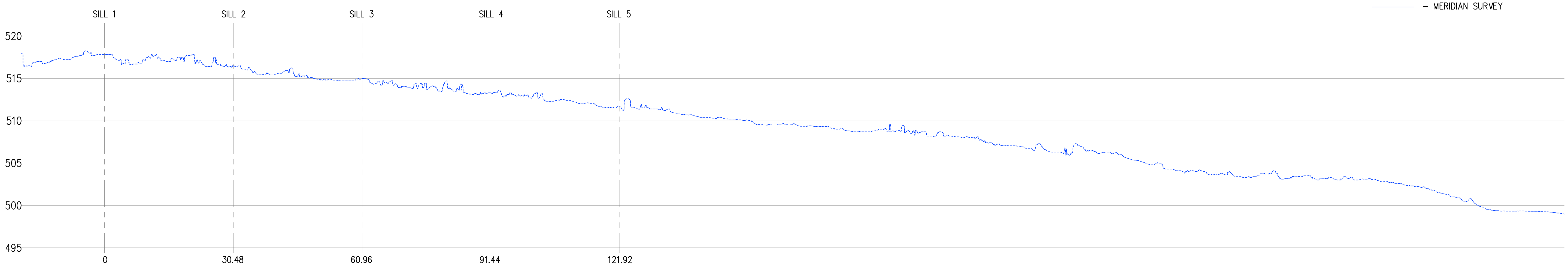
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
BAY 4
PLAN – 4 OF 4

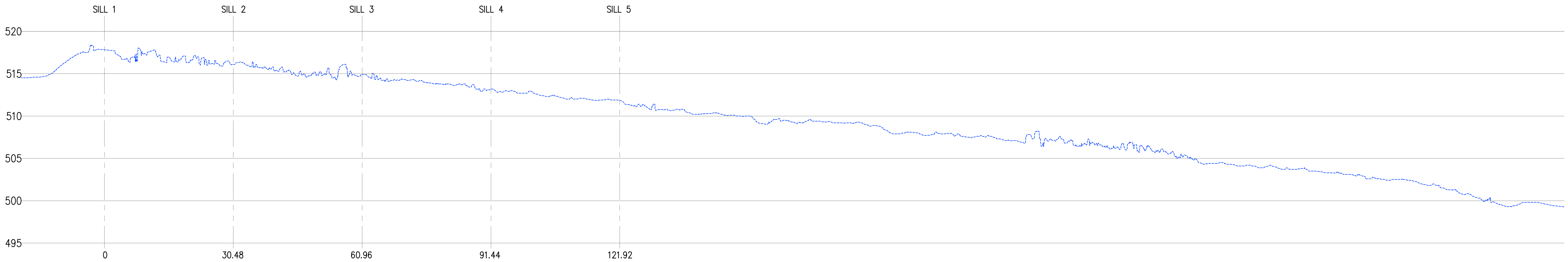
FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK–B\TW\CHT\BAY–4
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

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.

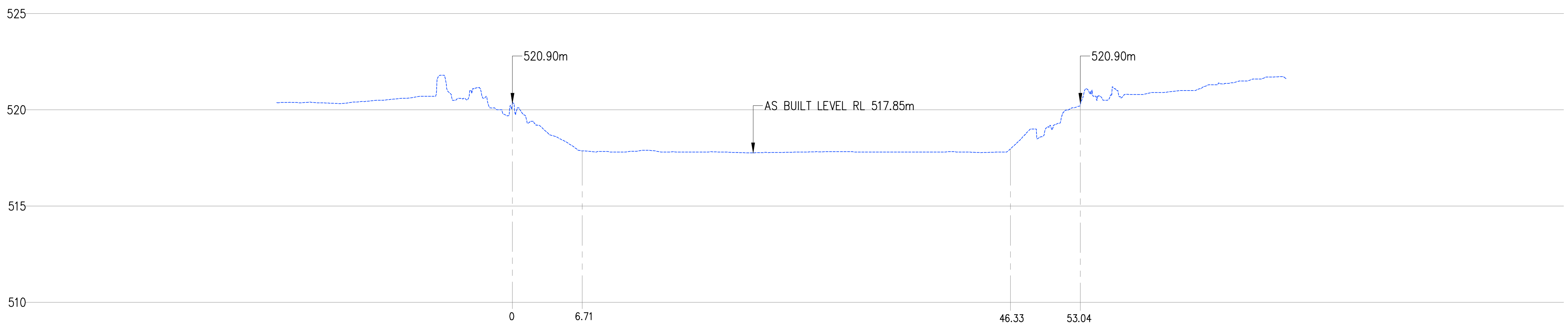
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
LONG-SECTIONS

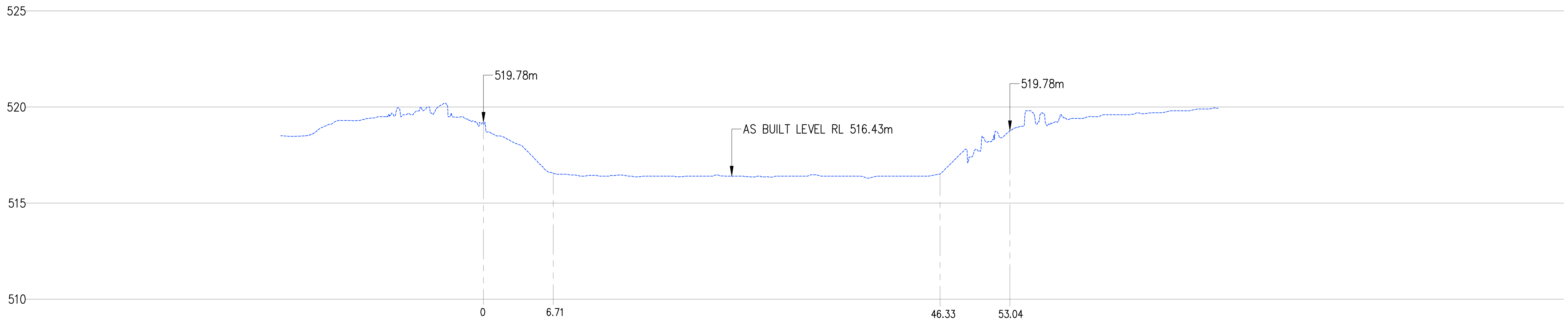
FOLDER:	XXX/XXX
DISTRIBUTION:	–
DRAWING:	TEK-B\TW\CHT\PROFILE SECTION
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

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

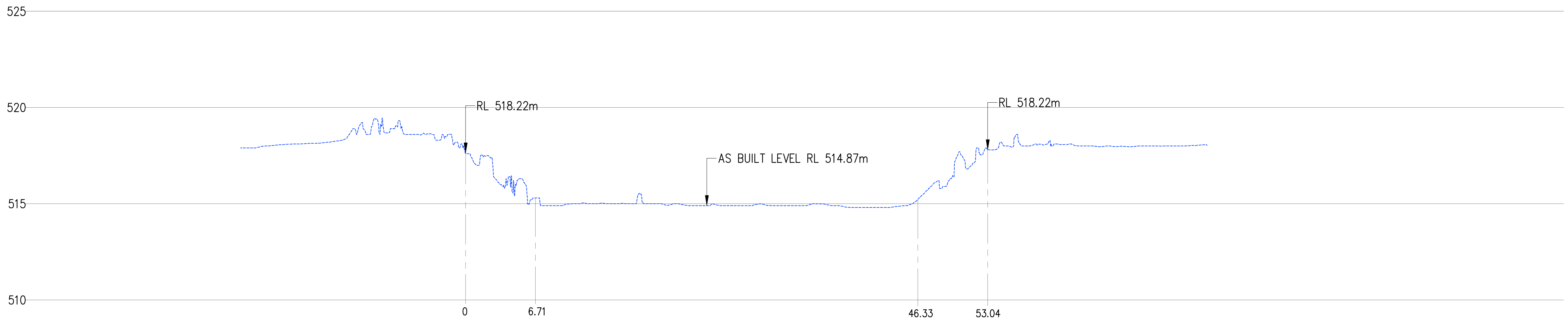
KEY:
— MERIDIAN SURVEY



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.

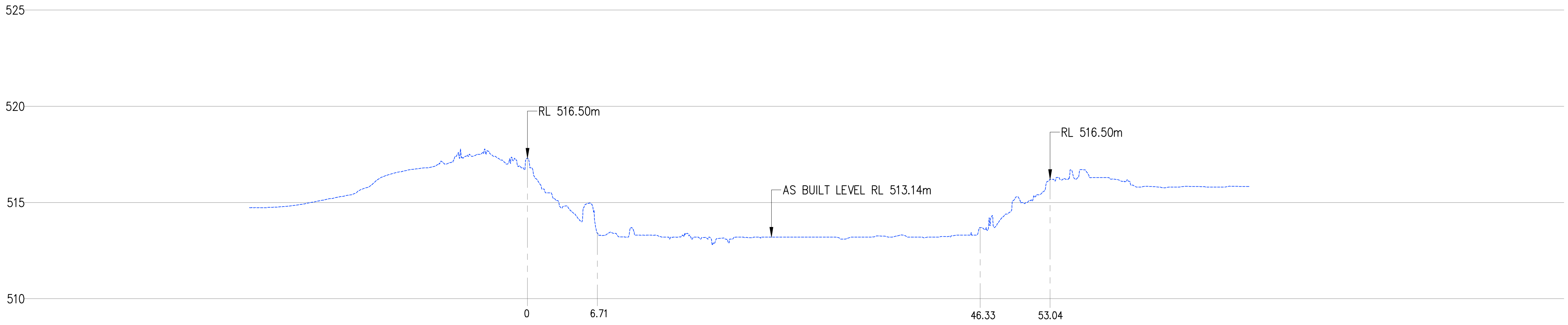
TEKAPO B PS TAILRACE WEIR

CONDITION ASSESSMENT
CROSS-SECTIONS
1 OF 2

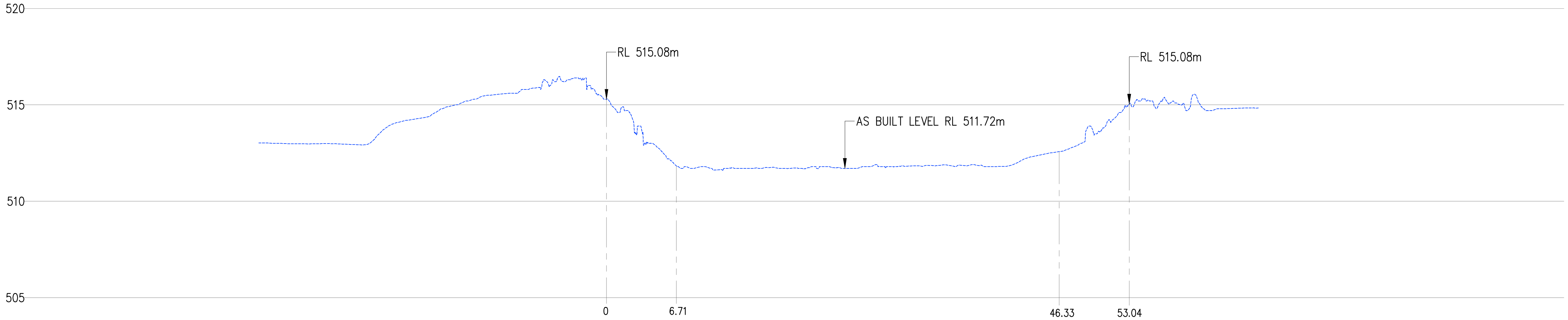
FOLDER:	XXX/XXX
DISTRIBUTION:	-
DRAWING:	TEK-B\TW\CHT\CROSS-SECTION
COMPANY:	DAMWATCH
NUMBER:	E2621
ISSUE:	R0

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