DAMWATCH
ENGINEERING BEYOND THE SURFACE

Maitai Dam

2013 Comprehensive Safety Review

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Prepared for Nelson City Council

Issue 2

P +64 4 381 1300 F +64 4 381 1301
A Level 8, 80 The Terrace, PO Box 1549, Wellington 6140, New Zealand

DAMWATCH ENGINEERING
www.damwatch.co.nz

Damwatch Engineering Ltd
PO Box 1549
Wellington 6140
New Zealand

Telephone: +64 4 381 1300
Facsimile: +64 4 381 1301
info@damwatch.co.nz

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Prepared By	Karina Dahl Dan Forster Brian Benson
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Executive Summary

Nelson City Council (NCC) engaged Damwatch to perform the 2013 Comprehensive Safety Review of Maitai Dam. The purpose of this Comprehensive Safety Review is to provide an independent assessment of the dam's safety status relative to current practice. This five-yearly Safety Review is in accordance with New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines (NZSOLD, 2000).

This is the third Comprehensive Safety Review for Maitai Dam following the NZSOLD Guidelines. Previous reviews were carried out in 1993 (Riley), 1998 (Opus), 2003 (Tonkin and Taylor), and 2008 (Riley), forming the reference points for assessing the current condition, performance and dam safety status of the dam.

The Safety Review dam inspections were made on 16th of January 2014. No testing of the dam low level outlet or scour valves was carried out during the inspection.

Background

Maitai Dam is located on the North Branch of the Maitai River, approximately 18km southeast of Nelson. The dam is owned by Nelson City Council and was constructed in 1986 as a water storage supply for the city of Nelson. Maitai Dam and appurtenant structures consists of a 39m high earthfill embankment with a low level culvert, intake tower, concrete service spillway and auxiliary overland flow (fuse plug) spillway.

Dam Safety Status and Performance

The dam is in good condition and is performing well. There are no indications of any of its failure modes developing. However, the following important dam safety issues are identified:

- The dam includes features that are no longer accepted in standard design practice and present potential vulnerabilities. These vulnerabilities need to be considered when assessing the dam performance. Such features include:
    - construction of a narrow chimney drain
    - a chimney drain of different gradations along the height
    - a culvert interceptor drain constructed of different material (varied permeabilities) and located under high head conditions
    - use of geotextile in embankment drains including the chimney drain
    - installation of drainage lines within the chimney drain and downstream shoulder
    - placement of plywood facing along the upstream culvert bays
    - penetration of culvert geometry within the embankment

  - foundation shaping allowed for benches and sharp corners
  - limited access to intake valves and pipework
- The dam appears to meet acceptability criteria for stability under normal loading conditions. However, this assessment is dependent on the chimney drain performance to act as a filter and drain and maintain unsaturated conditions in the downstream shoulder. Resolution of downstream shoulder pore pressures is required to verify indicated slope stability.
- The stability of the dam under earthquake loading needs to be assessed using site specific ground motions developed using current hazard models. Performance under the SEE needs to evaluate seismic-induced crest settlement and cracking and the potential for overtopping and internal erosion.
- The majority of historical embankment failures are due to piping and internal erosion. The potential for internal erosion within Maitai Dam needs to be assessed using current methodology and gradations of as-placed embankment and drainage materials.

Dam Safety Activities

The Nelson City Council has ongoing dam safety activities for Maitai Dam. A formal Dam Safety Assurance Programme is under development.

Nelson City Council's current dam safety activities cover the majority of Dam Safety Assurance Programme requirements. However, the following items are highlighted to be addressed as part of the current development of a formal Dam Safety Assurance Programme:

- formalisation of ongoing surveillance activities including a process to ensure evaluation, quality assurance and follow up of routine monthly surveillance data collected.
- established deformation survey programme exists, however, this report makes recommendation to improve its evaluation.
- formalisation of procedures for the investigation, assessment and resolution of dam safety deficiencies.

The Maitai Dam Emergency Action Plan (EAP) last issued in August 2010 meets NZSOLD requirements except for the following items:

- incorporation of developed dambreak inundation maps and include flood travel times and depths at key populated areas and evacuation routes
- evacuation planning including prioritised population details, methods of notification, and safe evacuation routes/destinations
- contact list is not complete
- downstream residents contacts list is not complete

- dam dewatering information and procedures are not provided
- sources of equipment and materials details are not complete
- a record of emergency action plan tests is not given

It is recommended that the Maitai Dam Emergency Action Plan is completed, that Nelson City Council staff and emergency agencies become highly familiar with it, and that it is tested for effectiveness and areas identified for improvement addressed.

Recommendations

The following table is a summary of recommendations made in this report relevant to Nelson City Council's dam safety activities for Maitai Dam.

No.	Recommendation	Report Ref.	Priority
1	It is recommended the flood inundation maps include tables of flood travel times and depths.	2.3	B
2	It is recommended documentation of the PIC assessment based on the updated inundation mapping be prepared to fulfil requirements of the Dam Safety Scheme.	2.4	C
3	It is recommended the as-built drawing of the new pipework be included in the drawing record for Maitai Dam.	3.9	D
4	It is recommended a site specific seismic risk study be performed for Maitai Dam.	4.2.3	B
5	It is recommended potential failure modes for Maitai Dam be developed.	5.0	B
6	It is recommended a characteristic model of Maitai Dam be developed for interpretation of surveillance information.	5.0	C
7	It is recommended the rusted pipework be cleaned and repainted.	6.1	C
8	It is recommended that seepage emerging adjacent to the culvert be monitored with documentation on its development.	6.1	A
9	It is recommended the repairs are made to ensure a smooth finish on the chute floor of the service spillway.	6.1	C
10	It is recommended the spillway chute walls be cleared of trees and bushes to facilitate inspection and prevent damage to the wall from root growth.	6.1	C
11	It is recommended that as-built location and installation details for B23 and B27 be confirmed and indicated on a drawing for the purpose of ongoing data evaluation.	7.3.2	B
12a	It is recommended that the purported water pressure in the upper embankment downstream of the chimney drain be thoroughly investigated and resolved. This should include an assessment of the reliability of the instruments and measured data, and consideration given to supplementary monitoring in this location.	7.3.2	A
12b	It is recommended all piezometer gauges should be calibrated to be accurate in their respective normal reading ranges.	7.3.2	B

No.	Recommendation	Report Ref.	Priority
13	It is recommended that piezometer de-airing operations continue at a frequency appropriate to observed accumulation of air in their data plots.	7.3.2	C
14	It is recommended that bottom of hole reduced levels be established for SB1-3 and BH7 to support ongoing data evaluation.	7.3.2	C
15	It is recommended the right exit area drain be checked for damage or blockage in regards to the new seepage observed (ref: Rec-08).	7.3.3	A
16	It is recommended that the spillway drainage system be assessed for condition and performance, and flushed to maintain functionality, as far as is practicable.	7.3.4	C
17(a)	It is recommended that Maitai Dam survey data be consolidated into a full and continuous historical record, managed in one repository, and that an appropriate suite of time-series and spatial plots be developed to allow evaluation.	7.4	C
17(b)	It is recommended that the dam survey mark locations and movement vectors be plotted onto the as-built valley cross sections so that deformations can be evaluated in the context of foundation geometry.	7.4	C
18	It is also recommended that inspection, monitoring and evaluation requirements be reviewed and updated with consideration of the dam's potential failure modes.	7.5	B
19	It is recommended that monthly routine surveillance data is evaluated at the same monthly frequency by a dam safety engineer.	7.5	B
20	It is recommended that NCC dam surveillance data management arrangements be reviewed by an appropriate advisor and improvements made to ensure quality assurance and security of data. Data presentation methods should also be reviewed and improvements implemented to ensure that surveillance evaluation is continuous and effective.	7.5	B
21	It is recommended inspection of the upstream slope area of the auxiliary spillway be performed following unusual high reservoir levels.	8.2.2	C
22	It is recommended that a full slope stability analysis be performed for Maitai Dam following verification of piezometric conditions within the embankment.	8.3.1	B
23	It is recommended assessment of seismic-induced deformations (settlement and cracking) be performed as part of the slope stability analysis of Maitai Dam following development of ground motions from the site specific seismic risk study.	8.3.1	B
24	It is recommended the potential for internal erosion as a result of seismic induced cracking be assessed at Maitai Dam.	8.3.1	B
25	It is recommended an assessment of potential internal erosion be performed for Maitai Dam embankment materials using current methods of practice.	8.3.2	B
26	It is recommended the risks associated with internal erosion and potential overtopping of the crest be assessed for Maitai Dam.	8.3.2	B
27	It is recommended that the performance characteristics of the Maitai Dam scour offtake are understood for the purpose of emergency dam dewatering.	8.5	B

No.	Recommendation	Report Ref.	Priority
28	It is recommended that procedures for ongoing surveillance activities be formalised, including a process to ensure evaluation, quality assurance and follow up of routine monthly surveillance data collected.	9.3.3	B
29	It is recommended that procedures for the investigation, assessment and resolution of dam safety deficiencies be formalised.	9.3.3	B
30	It is recommended that Maitai Dam appurtenant structures and gates and valves that contribute to reservoir safety be formally identified and testing arrangements made.	9.3.3	B
31	It is recommended that the Maitai Dam Emergency Action Plan is completed, that NCC staff and emergency agencies become highly familiar with it, and that it is tested for effectiveness and areas identified for improvement addressed.	9.3.3	A

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1.0 Introduction

1.1 Background

Nelson City Council (NCC) engaged Damwatch to perform the 2013 Comprehensive Safety Review of Maitai Dam. The purpose of this Safety Review is to provide an independent assessment of the dam's safety status relative to current practice. This five-yearly Safety Review is in accordance with New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines (NZSOLD, 2000).

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Maitai Dam is located on the North Branch of the Maitai River, approximately 18km southeast of Nelson. The dam is owned by NCC and was constructed in 1986 as a water storage supply for the city of Nelson. Maitai Dam and appurtenant structures consists of a 39m high earthfill embankment with a low level culvert, intake tower, concrete service spillway and auxiliary overland flow (fuse plug) spillway.

The Safety Review dam inspections were made on 16th of January 2014. No testing of the dam low level outlet or scour valves were carried out during the inspection.

Safety Review Team

The Safety Review team comprised of:

Brian Benson and Karina Dahl of Damwatch Engineering

Howard Schuppan and Alex Miller of Nelson City Council (NCC)

Trevor Ruffell and Richard Kennedy of Fulton Hogan

The Safety Review team gratefully acknowledges the helpful and efficient assistance given by Nelson City Council and Fulton Hogan personnel.

1.2 Review scope

The review of engineering and operation practices relevant to dam safety for the Maitai dam considers:

- Hazards that have the potential to impact on the safety of the dam;
- The potential impact of these hazards on the dam; and
- Consequential safety implications for the downstream community and environment.

Hazards may be natural hazards such as earthquakes, floods and landslides, or hazards relating to the dam such as degradation of dam materials or operational issues.

The activities involved in this Safety Review are:

- Visual inspection of the dam and associated appurtenant structures;
- Assessment of the construction, design, monitoring, physical condition and performance of the structures with reference to current acceptability criteria; and
- Preparation of a report.

No significant new calculation or studies are carried out in this Safety Review. References have been made to existing documentation, records and any work carried out since the last Safety Review in 2008.

1.3 Significant Events and New Information Since 2008 CSR

Significant events and new information since the 2008 CSR include:

1. A duplicate section of the Maitai water supply pipeline was constructed. This included a new pipeline section, bifurcation and discharge valve at the downstream end of the culvert.

2. A Flood Hazard Mapping Modelling Report was prepared (Tonkin & Taylor, 2013). Results of the 2005 dambreak study were used to develop hazard maps for three breach development times for use in emergency planning.

3. A magnitude 6.5 earthquake occurred in Cook Strait on 21 July 2013. This earthquake followed two earlier shakes of magnitude 5.7 and 5.8 in the previous two days. The 6.5 earthquake had a modified Mercalli scale of 8 with damage to buildings on both sides of the Cook Strait. However, ground motions at Maitai Dam were minimal and rated as “weak” on ShakeMap (USGS).

4. On 28th December 2010, a historic high level of R.L. 175.04 m was recorded for the Maitai reservoir. The high reservoir level corresponds to a freeboard of 1.96m to the dam crest and 0.57m to the sand fuse embankment crest although the freeboard to the sill of the auxiliary spillway was only 0.14m. This occurred during a moderate storm event over the entire catchment.

5. On 14th December 2011, a large storm event occurred over the entire catchment, especially the Brook catchment. Reservoir level at Maitai Dam was R.L. 174.51.

2.0 POTENTIAL IMPACT CLASSIFICATION (PIC) REVIEW

2.1 Potential Impact Classification

The Potential Impact Classification (PIC) is a cornerstone of dam safety management in New Zealand. The PIC classification of large dams is intended to align dam safety practice to reflect the potential impact of dam failure on people, property and the environment.

The PIC classification is required under the new Dam Safety Scheme of the Building Act (Department of Building and Housing, 2008), to determine procedures such as Dam Safety Assurance Programmes (DSAP). The PIC is also used in the NZSOLD Dam Safety Guidelines to apply increasing levels of safety (as the consequences of failure increase) to dam development, maintenance and operation.

The Dam Safety Scheme has introduced a new methodology for determining a dam's PIC. The classification system is different from that included in NZSOLD's Dam Safety Guidelines (NZSOLD, 2000). The Dam Safety Scheme is expected to come into operation in March 2015. In the interim the current NZSOLD methodology is the de-facto standard.

2.2 PIC Determinations

Maitai Dam has a High PIC rating based on consequence assessment performed in 2005 (Tonkin and Taylor). Results of a recent dam-break study review (Tonkin & Taylor, 2005) and inundation mapping, performed by Tonkin and Taylor (2013) are consistent with the High PIC rating. However, appropriateness of the inundation mapping presented in the Tonkin and Taylor (2013) report, particularly for dam safety action planning, is addressed below in Section 2.3.

2.3 CSR Review of PIC

The inundation mapping provided in Tonkin and Taylor 2013 report presents:

- A "sunny day" piping initiated dambreak analysis and consequent dambreak outflow inundation;
- Assessment of the flood passage capability of Maitai Dam and finds that the spillway facility has capacity to pass a probable maximum flood (PMF) with greater than 0.5m freeboard;
- Concludes that, as a consequence of the spillway capacity being significantly greater than the PMF, overtopping is not a credible failure mode; and
- Consequently does not consider potential "rainy day" failure of the dam.

The inundation mapping presented in the Tonkin and Taylor (2013) report is for piping initiated peak dambreak outflow corresponding to a breach development times of 0.5, 1 and 2 hours. The dambreak hydrographs for the breach development times were developed as part Dam-Break Study Review (Tonkin & Taylor, 2005). The peak dambreak outflow corresponding to a breach development time of 1 hour is justified by experience with the overtopping failure of Ophua dam in Otago in 1997. For dam safety action planning, (**Rec-01) It is recommended the flood inundation maps include tables of flood travel times and depths.**

2.4 Summary

It is very clear that the Maitai Dam's High PIC is appropriate. Based on the number of houses inundated and consequent population at risk (depth of inundation > 0.5 m), particularly in the reaches closest to the dam, will result in a High PIC being assigned.

The High PIC for the Dam is put into context throughout this CSR, as it pertains to dam performance criteria and ongoing dam safety assurance.

Early next year the Dam Safety scheme (supported by the Dam Safety Regulations) will require documentation to justify classification of Maitai Dam and subsequently a Dam Safety Action Plan. Accordingly, the inundation mapping should be prepared consequent of the peak dambreak outflow based on appropriate breach development time (**Rec-02)** I**t is recommended documentation of the PIC assessment based on the updated inundation mapping be prepared to fulfil requirements of the Dam Safety Scheme**.

3.0 Dam Details

3.1 General

Maitai Dam located on the North Branch of Maitai River just upstream of the confluence with the South Branch in the Bryant Range. Figure 1 shows the dam location and topography. Topography of the catchment above the dams is moderate to steep and covered primarily with native vegetation. Figure 2 shows an aerial view of Maitai Dam. Approximately 7km downstream of the dam the river flows in a relatively narrow gorge which includes Maitai Valley road and the water supply pipeline on a bench upslope. Sharland Creek enters the river at approximately 9.5km downstream of the dam and before reaching the floodplain of Nelson. River flows outlet into the ocean approximately 15.5km downstream of the dam.

Maitai Dam is a 39m high and 160m long semi-zoned earthfill embankment with low level diversion outlet/culvert connected to an intake structure. On the left abutment is a spillway and adjacent emergency spillway (fuse plug). Table 1 provides a summary of the dam details.

The dam was designed by Tonkin and Taylor (1984: rev.1985/86) and constructed by Wilkins & Davies Construction Company Ltd. between 1984 and 1986 (Tonkin & Taylor 1987 construction report) as part of the Maitai Water Supply Project.

Design and construction documentation are available in reports by Tonkin & Taylor (1986, 1897, 1989). Additionally, construction photographs are held by NCC and stored at the dam site along with other dam records and boring core samples. Extensive as-built drawings exist; Appendix A provides a register of the as-built drawings and selected reference drawings. This CSR used this available existing information.

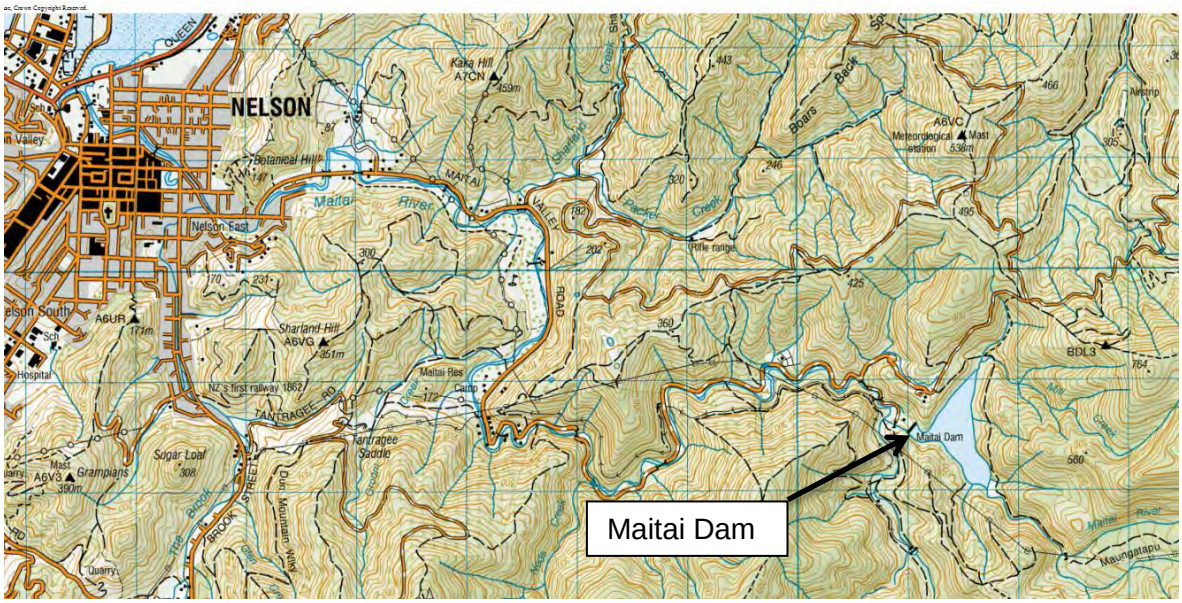

Figure 1: Maitai Dam Location Map

Table 1: Summary of Maitai Dam Details

Parameter	Value
Crest level	R.L. 177.0m
Dam height	39m
Crest width	6.1m
Crest length	approx. 160m
Upstream Slope	2.6H:1V
Downstream Slope	2.1H:1V with 4 m wide berm at R.L. 155m
Chimney level	R.L. 175.0m
Spillway crest level	R.L.173.75m
Spillway crest width	20m
Fuse plug crest level	R.L. 175.61m
Fuse plug sill level	R.L. 175.18m
Fuse plug width	20m
Normal operating range	Average R.L. 173.75m
Reservoir capacity at spillway crest level[1]	4,150,000m³

3.2 Geology

Bedrock in the dam site area consists of mudstone, siltstone, and sandstone of the Permian age, belonging to the Greville Formation of the Maitai group (middle Triassic age). The formation is tightly folded as a result of faulting which occurred early in the geological history of the formation (culvert geology report Tonkin & Taylor 1984). Geology of the Maitai Dam site is fully described in Volume II of the feasibility study report (ref 2 of design report) and shown in Figure 3.

Key aspects of the site geology in design of the dam (design report Tonkin & Taylor rev. 1985/86) include:

- Steep abutment slopes consisting of tightly jointed argillite rock with a thin covering of residually weathered rock and overlying colluvium;
- Valley floors eroded into relatively fresh rock with an overlying thin deposit of alluvial soils;
- Bedrock is more weathered at shallow depths with more open joints, which is considerably more permeable than the argillite rock mass;
- General pattern of bedding and defects are close to vertical and parallel to dam centreline;
- Crushed and sheared zones were encountered in the foundation excavations. This includes the central inspection zone, an approximately 20 to 40m wide (culvert chainage 32 to 73m), zone of weaker or more intensively sheared rock crossing the dam foundation downstream of the chimney drain. Other sheared zones are located further upstream; and

- Inactive Wooden Peak fault passing through reservoir area.

3.3 Foundation Treatment

Treatment of the foundation was specified by two standards of preparation, Grade A and Grade B. Grade A preparation consisted of removal of all surficial materials down to rock or weathered rock including all loose material. Grade A was applied to a 20m wide central inspection zone, which ran parallel to the dam centre line and to the base of downstream collector blanket drain and downstream abutment drains. Grade B preparation required removal of all alluvial material and other surficial material below a specified strength but allowed for loose material to be left on the surface provided it could be incorporated in the placement of the first layer overlying fill. Grade B preparation applied to all other locations of the dam foundation. Majority of abutment areas were excavated down to underlying rock.

Foundation grouting was considered unnecessary and the following local areas of weaker sheared rock were treated with surface blinding concrete:

- Coffer dam area;
- Central inspection zone;
- Immediately alongside the left side of the diversion culvert (Drawing 6516-6ABA) from upstream valve chamber to just downstream of crest except within the interceptor drain;
- Trench along the right hand side of the upstream section of the culvert; and
- Areas on left hand side of Culvert Bays 1 to 5.

Seepage encountered during excavation occurred either at the interface of the rock and the overlying soil or from mechanically loosened upper metre of the rock mass. (culvert foundation mapping) No seepage was observed from any of the defects in intact rock mass. Permeability tests performed during feasibility study indicated rock mass at depth to have very low permeability of1E-06 to 1E-07 m/s.

3.4 Earthfill Embankment

Maitai Dam embankment has slopes at 2.1H:1V along the downstream and at 2.6H:1V along the upstream as shown on Drawing 6516-9AB. In the immediate vicinity of the valve chamber, the upstream slope is steepened locally to 1.5H:1V. The valve chamber has a low headwall on top and tapering along the side walls to retain the embankment fill. Rockfill is placed on top of embankment fill to complete the normal 2.6H:1V upstream slope below R.L. 146m. The downstream shoulder includes a toe buttress with a 4m wide bench at R.L. 155.0m.

The embankment is semi-zoned earthfill with a central chimney drain as shown on Drawing 6516-9AB. Embankment material zones were derived from on-site borrow colluvium and weathered rock. The entire upstream section and upper 17m of downstream section consists of Type I fill. Type IA fill is placed in the lower downstream section. Type IA fill is similar to Type I fill with allowance of coarser material. Type I fill is relatively impervious material with construction permeability results of 10^{-9} to $3.5x10^{-10}$m/s (Tonkin & Talyor, 1987). The downstream toe buttress is constructed out of the Type II fill. The upstream slope is protected by a 0.5m thick layer of rip-rap of 100 to 300 mm sized rock from R.L.165m to crest level. The rip-rap is placed on top of a 150mm thick layer of Type B drainage material.

Figure 2: Aerial View of Maitai Dam (November 2011)

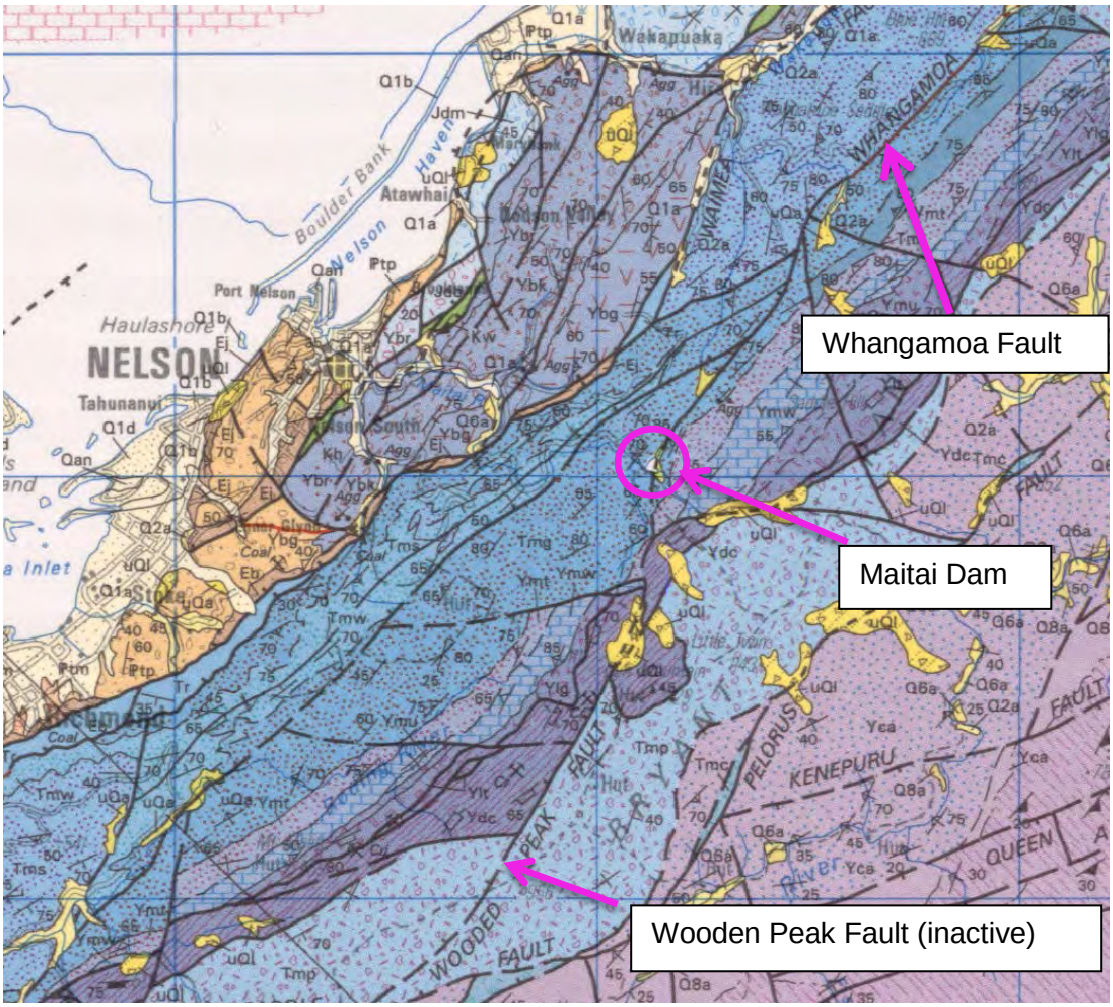

Figure 3: Geologic Map of Nelson (GNS, 1998)

Table 2 lists the specified gradations of material zones Type I, IA and II (Tonkin & Taylor, 1986). The Type I and Type IA fills classify as clayey/silty gravels (GC/GM) with over 20% fines content. The coarse fraction of Type I fill is poorly graded fine to medium gravels while coarse fraction of Type IA fill is a poorly graded fine to coarse gravels. The Type II fill is a well graded to clayey/silty (poorly-graded) gravel derived from spillway excavations of weathered rock. The construction report (Tonkin & Talyor, 1987) states Type I fill placed generally meet the gradational requirements but a small amount (~3 to 4%) of material retained on 26.5mm sieve (coarse gravel) was allowed instead of the specified limit of zero. Type IA fill was occasionally found to just meet the specified 20% minimum fines content.

Table 2: Specified Gradations for Embankment Fill Materials (Tonkin & Taylor, 1986)

Classification	Particle Size (mm)	Percent Passing by Weight		
		Type I	Type IA	Type II
Cobbles	150	100	100	100
	75	100	100	80-100
Cr. Gravel	53	100	-	70-100
	37	100	85+	-
	26.5	100	-	-
Med. Gravel	19	-	70+	40-100
	9.5	80+	60+	-
Fine Gravel	4.75	55+	45+	20-60
Sand	2	30+	30+	10-45
	.6	25+	25+	0-30
Fines	.063	20+	20+	0-20

<u>Chimney Drain</u>

The central chimney drain is designed to intercept and control seepage from the upstream section of the dam from reaching the downstream section. Details of the chimney drain are provided in Drawings 6516-9AB and 6516-11AB. The chimney drain transverses the full width of the dam axis and surrounds the culvert including a 0.75m wide by 0.6m deep trench below. At the base the chimney drain it connects to the central collector drain. The chimney drain was constructed progressively as the dam height increased by trenching into fill placed above protected drain material. Appendix B provides a few construction photographs of the chimney drain construction. The top of the chimney drain is at R.L. 175.0m, which is 2m below the dam crest.

The chimney drain materials and width varies over the height of the dam. The drain is 0.75m wide along the height of the dam and widens to 1.5m in the upper 10m (R.L. 165.0 to 175.0m). Below R.L. 158.5m per Drawing 6516-11 AB or R.L. 157.5m per the construction report (Tonkin & Talyor, 1987), the 0.75m width is constructed of Type C drainage material. Above this level up to the top at R.L. 175.0m, Type A drainage materials were used to construct the 0.75m wide section. The widened section from R.L. 165.0 to 175.0m includes 0.75m of Type B drainage

materials on the downstream side. Filter cloth was provided along upstream and downstream sides between drainage material and embankment fill.

The wider upper section of the chimney drain was designed to provide increased flow capacity and protection in the top section of the dam most vulnerable to cracking and slumping in severe earthquake ground shaking. The downstream 0.75m wide section incorporates a 160mm OD perforated HDPE pipe in the Type B drainage material at R.L. 165 m, which connects to an unperforated HDPE pipe extending to each abutment. These lateral drainage lines were installed to intercept and collect leakage flows as a result earthquake cracking.

Gradations of drainage materials Type A, B and C were based on material compatibility and permeability requirements. Specifications provide gradations for Type A and B drainage materials but were not reviewed as part of this CSR. The construction report (Tonkin & Talyor, 1987) provides a good summary of drainage materials gradation and permeability achieved versus specified values.

Type A drainage materials conforming with specified gradations however had permeability results recorded during construction of 8.6×10^{-5} to 1.2×10^{-4} m/s, which is slightly lower than specified minimum of 10^{-4}m/s value. Gradations of Type B drainage material recorded during construction were outside that specified with typically 1% passing 0.6mm sieve while permeability results of 2.9 to 3.1×10^{-2} m/s being consistent with specified minimum of 10^{-2}m/s value. Gradations of Type C drainage materials consistently conformed to the specified envelope and permeability results of 2.0×10^{-2} m/s being greater than a 7.5×10^{-4}m/s value used in design.

3.5 Drainage

Drainage of seepage flows within the embankment is through the central collector and blanket drains. Drawing 6516-11 AB shows details of the drainage blanket and central collector.

Central collector drain transmits seepage collected by the embankment chimney drain and foundation seepage from the drainage blanket. It generally follows the old river bed on the left side of the culvert and occupies the lower part of the valley. The drain is 2m wide by 0.6m deep trench and consists of a 170 OD perforated HDPE pipe surrounded on the sides and top by densely packed 75-150mm cobbles which are surrounded by Type B drainage materials. The Type B drainage materials are connected with the drainage blanket.

The drainage blanket was placed from the downstream face of the chimney drain along the full width of the left hand side of the culvert to the left abutment (Tonkin & Talyor, 1987) and incorporates the central collector drain. The blanket has a nominal 0.2m thick layer of Type B drainage material covered by filter fabric. Appendix B provides a few construction photographs of the drainage blanket construction. The filter fabric is fixed by timber battens to the side of the culvert and at the abutment side provided with a 0.4m return on the fabric. The blanket tapers to a 2 m wide by 0.8m thick central zone near the outlet. Where sheared faults zones contained very fine material, two layers of filter compatible sands were placed prior to placement of overlying Type B drainage material (Tonkin & Talyor, 1987).

Seepage along the right hand side (R.H.S.) of the downstream culvert bays 12 to 20 is collected by a drain shown on Drawing 6516-8ABA. Seepage is conveyed to the south retaining wall

111mm OD perforated drainage pipe, which exits out in the “culvert exit area flow”. Drawing 6516-15 AB shows details of the pipe layout. The R.H.S. culvert drain was provided in construction and was not part of the original design. The foundation shaping enabled incorporation of such a provision. The drain consists of drainage material and a perforated HDPE pipe. Due to the non-availability of Type A drainage material at the time of work, Type B material was used and wrapped in filter cloth.

Abutment Drains

The left and right abutments are provided with contact blanket drains at the downstream limit of the dam as shown on Drawings 6516-6 ABA and 6516-G4. The abutment drains consist of trenches filled with drainage Type B material, surrounded by filter fabric and with a drainage pipe. Along the steeper section of the abutment the contact drains do not include a drainage pipe. The left and right abutment contact flows are conveyed separately for monitoring at the central collector manhole.

3.6 Culvert

A diversion culvert penetrates through embankment along the right side of the former river channel. At the upstream end the culvert is the valve chamber with pipework connected to the intake tower. The culvert houses the water supply and scour pipelines supported by brackets from the valve chamber to pipework connections near the control building downstream of the dam toe.

Diversion culvert is a 3m high by 2m wide rectangular reinforced concrete structure consisting of 20 bay sections as shown on Drawing 6516-101 AB. The concrete walls and floor are 0.65m thick in the upstream 14 bay sections and 0.5m thick in the downstream 6 bay sections. All joints between adjacent bays were protected with external waterstops .The upstream bay joints (total of 14) were covered by 0.6m wide plywood panels with mastic applied along the upstream concrete/plywood joint.

The culvert was constructed in a shallow bench on the right side of the foundation. The foundation was prepared and blinded using ready mix concrete. Local areas with soft materials were removed and backfilled using ordinary grade blinding concrete.

The culvert has an interceptor drain located 10m upstream of the chimney drain to monitor seepage along the conduit separately. The interceptor drain consists of three 0.5m thick zones of filter sand at the upstream face, Type A drainage material in the middle and Type B drainage material at the downstream face. The drain is surrounded by filter cloth and on the downstream face includes a polythene sheet. Details are shown on Drawing 6516-11AB. Filter sand is well graded from 0.02mm to 2mm size with a D50 between 0.2mm and 0.5mm.

3.7 Reservoir

The impounded reservoir has a surface area of 32 ha and storage volume of 4,150,000 m^3 (Tonkin & Taylor, 1986). The reservoir area includes moderate to steep slopes. Slopes are vegetated with native bush and along the southern rim there is a forested area.

3.8 Spillways

Design for the dam to safely pass flood flows is through the service spillway and an auxiliary spillway. The service spillway is required to operate at flood flows up to 1:100 AEP and in combination with the auxiliary spillway up to the probable maximum flood (PMF).

3.8.1 Service Spillway

The service spillway comprises of a free-overflow ogee crest and ~115 m long lined chute. The chute is 20m wide at the entrance and tapers to 10m wide over a 56.5m distance. At the chute terminus, flows enter a flip bucket with a 17m circular radius. A debris barrier is provided across the spillway forebay to capture logs which might jam in the service spillway. A bridge crosses over the flip bucket to carry the access road for the South Branch.

The flip bucket acts as a stilling basin for flows less than about 20m^3/s. The low flow apron is provided below the flip bucket to convey flows less than 20m^3/s into the plunge pool and prevent erosion at the flip bucket toe. For higher flows the apron is detailed to ensure an air supply to the underside of the jet as the jet starts to break free.

The South Branch and backfeed 0.6m diameter steel pipelines are cast in concrete beneath the downstream end of the service spillway.

Seepage monitoring of the spillway system is arranged to enable independent measurement of each 20m length of underdrainage blanket and each 40m length of chute wall. The drainage blanket beneath the chute is confined between the spillway wall foundations and by transverse cut-off walls spaced at 20m intervals. The seepage flow is directed to the downstream cut-off, then collected in polyethylene discharge pipes with stone and mesh entry filter and taken through the wall foundation to the rear of the right spillway chute wall. At the chute walls, drainage material was placed along the lower portion. Flows are directed to a manhole located along the right side. For the flip bucket two gravel drains are provided, which connect into the outlet drains which run on either side of the chute and discharge onto the apron slab above normal plunge pool levels.

3.8.2 Auxiliary Spillway

The auxiliary spillway is located to the left of the service spillway and separated by some 3m width of rock. A concrete slab protects the bedrock foundation of the fuse plug. The fuse plug consists of an erodible sand embankment with an upstream zone of low permeability material. Triggering of the fuse plug is through three precast troughs connected to loose jointed clay tile pipes embedded in the sand embankment. The troughs have a weir type entry set to provide full pipe flow at crest level. Drainage material in filter cloth is provided against the walls of the service and auxiliary spillways and rock mass.

The concrete inlet weir of the fuse plug directs flow along a 20m wide unlined channel. The majority of the channel is excavated through rock with the final 7 m length routed over an area of bladed fill from construction excavations. The channel has a central “gulley” shape to direct flows away from service spillway located to the right.

3.9 Water Supply Intake and Valve Chamber

The intake structure is steel pipe tower with a working platform at the top reservoir level. There are three main vertical pipe supports that also serve as water intakes (refer Drawing 6516-121AB). The intakes are at R.L. 147.0m (lowest), 167.75m (6m below normal top water level), and R.L. 157.4m (mid-way depth). The scour valve is set as low as possible R.L. 146.0m while allowing some space for permanent siltation below scour level. There are three main steel pipe sections which have flanged and bolted joints. The main pipe sections are supported with laterals beams at 5m spacing over the 30m height between the top of the valve chamber and the base of the tower platform. A spiral staircase is mounted along the centre support for access to the platform.

The intakes are screened bell mouthed valves. Trapdoors are provided in the platform floor through which the screens may be lifted for servicing. Hoisting equipment for the screens consists of a winch system mounted to the tower roof supports.

Cathodic protection was installed in 2008 to provide corrosion protection of the pipework. Bolt corrosion is an on-going issue which is addressed by regular dive inspections and replacement programme. All steel pipes had a shop applied protective coating with an expected life of greater than 30 years and inaccessible pipework has a thicker wall section for corrosion protection.

The valve chamber is an extension of the culvert and connected to the intake tower pipework. It is up to 5.6m wide and 5m high internally to accommodate the intake pipework and valves. The 0.6m diameter scour line has a gate valve while the three intakes lines are provided with a butterfly valve before joining together. The two pipelines are then carried downstream in the culvert.

<u>Pipework at downstream</u>
There is pipework at the exit area of the culvert for the water supply line and a backfeed pipeline to the Treatment Station Platform. Details are provided on Drawing 6516-136 AB. Butterfly valves are provided on the water supply line and backfeed pipeline. The backfeed pipeline is connected to both the water supply pipeline and the scour pipeline. At the end of the scour line is a gate valve for releases. Inside the Treatment Station Platform, the water supply and backfeed pipelines are located next to each other.

The Maitai water supply pipeline is considered to be critical infrastructure for Nelson City. For water security, a duplicate section of Maitai water supply pipeline was recently constructed and completed at the time of the CSR site inspection. The new pipeline was connected to the free discharge end of the original water supply line in the pipework at the end of the culvert. The new pipe section includes a bifurcation with one section being exposed for short distance before being buried and continuing on to the Nelson Waste Water Treatment Plant. The other section has a control valve to allow releases to the outlet channel adjacent to the scour line. (**Rec-03) It is recommended the as-built drawing of the new pipework be included in the drawing record for Maitai Dam.**

4.0 Natural Hazards

4.1 Floods

4.1.1 Precedent Floods

A comprehensive record of precedent floods experienced by the Maitai Dam does not exist. The spillway rating curved (Drawing 92193-6) provided in the SEED review (Riley Consultants, 1993) is used in the following to estimate flood flows based on the reported water level above the spillway crest. The following flood events are cited as part of this CSR:

- 1993 SEED (Riley Consultants, 1993). A large flood occurred in June 1992 with 1m over the spillway crest.
- 2008 CSR (Riley Consultants, 2009), 24th November 2008 a significant flood event occurred in Maitai Dam catchment with approximately 1.2m over the spillway crest for a discharge of approximately 52m^3/s.
- Reservoir level record in this CSR review period (2009 to 2013) shows a maximum R.L. of 175.04m, with 1.29m above the spillway crest, on 28 December 2010. This flood record corresponds to a flow of approximately ~54m^3/s.

The Tasman District Council (TDC) website provides river flow data for the Maitai River at Forks gauge located downstream of Maitai Dam. River flows at the gauge are influenced by outflows from the Maitai Dam. The catchment at the river gauge is 35.7km^2 while Maitai Dam's catchment is 13.5km^2 or about 38% of the river gauge catchment. The 2008 CSR (Riley Consultants, 2009) was able to estimate outflows of Maitai Dam service spillway accounted for about 40% of the measured river flows at this gauge. River flow history between November 1989 and December 2013 for the Maitai River at the Forks gauge (TDC) and estimated at Maitai Dam are summarized in Table 3.

Table 3: River Flow History for Maitai Dam Catchment Area

Return Period (AEP)	Maitai at Forks[a] Flood Flows (m^3/s)	Maitai Dam[b] Flood Flows (m^3/s)
5	115.6	46.3
10	138.9	55.6
20	161.2	64.5
50	190.0	76.0
Extreme Recorded	179.5 (23/02/1995)	71.8

[a] data provided on Tasman District Council website
[b] estimated based on 40% of Maitai River at Forks gauge

The TDC website currently does not provide a flood flows for the 1:100 AEP event. The 2008 CSR reported a flood flow of 208m^3/s for the 1:100 AEP event at the river gauge from the TDC website. The corresponding flow for the 1:100 AEP event would be ~83m^3/s at Maitai Dam

using the 40% contribution. Extrapolation of Table 3 data would approximate the 1:100 AEP event with flows of 225m³/s at the river gauge and 90m³/s contribution at Maitai Dam.

4.1.2 Maitai Catchment Inflows Floods

Maitai catchment inflows have been estimated in a number of reviews with values less than originally used to design the spillway. Table 4 provides a summary of the inflow flood estimates for Maitai Dam. Further commentary and assessment of inflow design floods is made in Section 8 Engineering Assessment.

Table 4: Maitai Dam Inflow Flood Estimates

Study	1:100 AEP	PMF (Outflow)
1984 Adopted for design (Tonkin & Taylor, 1986)	125m^3/s	290m^3/s (280m^3/s)
1994 Maitai River Waterway Upgrading Investigation report (Tonkin & Talyor, 1994)		260m^3/s (246m^3/s)
1998 CSR (indicative) by Opus		277m^3/s
2003 CSR by Tonkin & Taylor	80-100 m^3/s (from 22-yr South Branch record) ~88-112m^3/s (from less reliable 23-year Smiths Ford/Forks record)	

4.2 Earthquakes

4.2.1 Seismic Hazards

Local active seismic sources reported on GNS active fault database for Maitai Dam include:

- Whangamoa fault: ~9km north, dextral type, recurrence interval of 3,500-5,000 years.
- Waimea, Eighty Eight and Heslington faults (combined): ~14km south, reverse type, recurrence interval of 3,500-5,000 years.
- Bishopdale fault: ~7.5km west, dextral type, recurrence interval of 3,500-5,000 years.

Figure 4 shows the area faults identified in study for NCC (GNS, 2013) The Whangamoa Fault was considered in design of the dam (Tonkin & Taylor, 1986). The Waimea fault was considered for earthquake loading in another seismic hazard study for Maitai Dam (GNS, 1993). A fault hazard corridor map was prepared as part of the GNS study for NCC to use for planning purposes. The hazard corridor related to the Whangamoa Fault is shown in Figure 5 along with Maitai Dam. No site specific ground motions were prepared for Maitai Dam in this study.

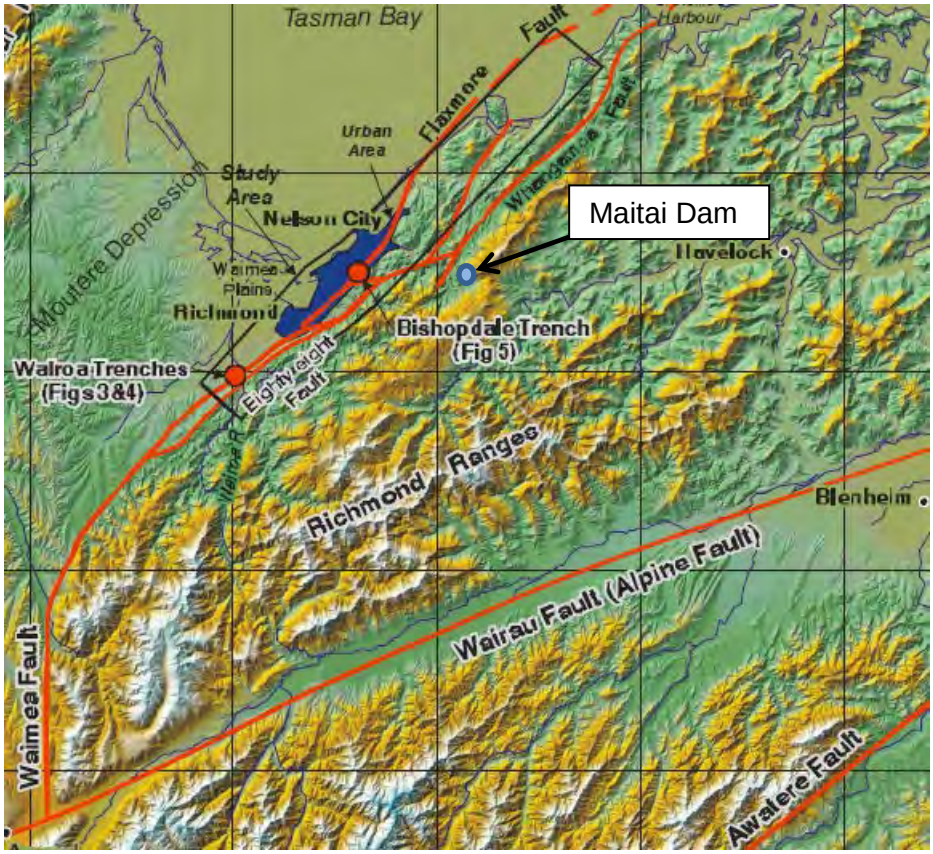

Figure 4: Faults in the Nelson Area (GNS, 2013)

The 2010 National Seismic Hazard Model (Stirling et al. 2010) identifies the Walmea fault (south and north traces) as the major features in the Nelson region. Slip rates are poorly constrained for these faults but are indicated to be low with less than 1mm/yr. The Walmea fault (south and north) is cited to be capable of magnitude 7.0 to 7.4 earthquake with a return interval of 5,570 to 9,610 year, respectively.

4.2.2 Precedent Earthquakes

Maitai Dam or the dam site has not experienced a large earthquake (M>6.5) based on review of GNS Science historical earthquake catalogue. The two largest magnitude earthquakes within the Contractional Northwestern South Island Fault region (Stirling et al. 2012) are:

(1) 1968 Inangahua earthquake M 7.2 on the Lyell and Inangahua faults and

(2) the 1929 Buller earthquake M7.8 on the White Creek fault.

These active fault traces are located approximately over 100km southwest from Maitai Dam.

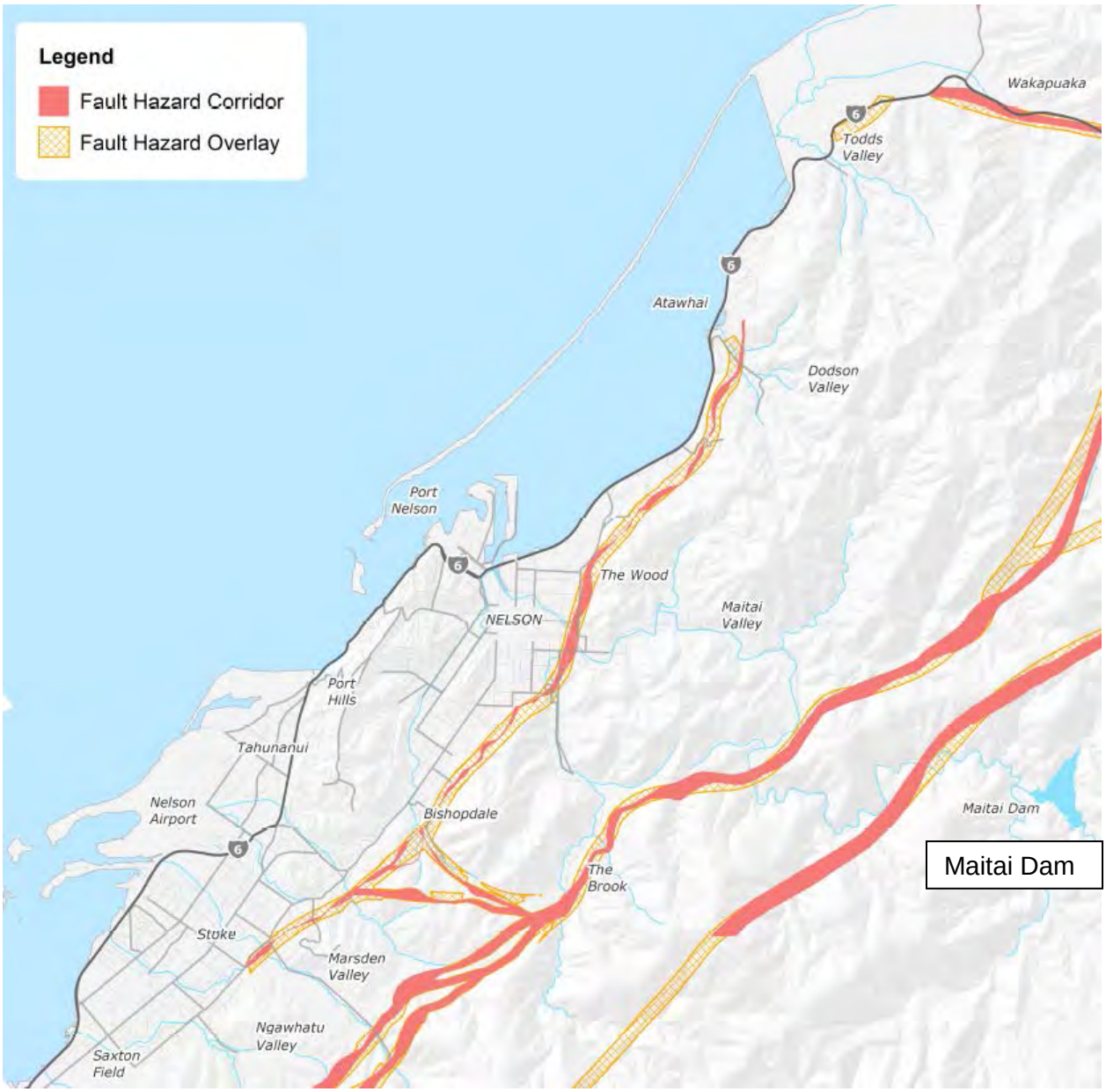

Figure 5: Fault Hazard Corridor Map and Maitai Dam (GNS, 2013)

A magnitude 6.5 earthquake occurred in Cook Strait on 21 July 2013. This earthquake was followed two earlier shakes of magnitude 5.7 and 5.8 in the previous two days. The 6.5 earthquake had a modified Mercalli scale of 8 with damage to building on both sides of the Cook Strait. However, ground motions at Maitai Dam were very low and on ShakeMap described as weak. In Picton approximately 70km east of Maitai Dam, Queen Charlotte College accelerogram recorded only a peak ground acceleration (PGA) of 0.16g.

4.2.3 Ground Motions

Ground motions for Maitai Dam were developed for design and have been reviewed and commented in the past CSRs.

Design of Maitai Dam (Tonkin & Taylor, 1986) was based on a probabilistic design basis earthquake (DBE) and a deterministic maximum credible earthquake (MCE) as follows:

- DBE with a PGA of 0.35g representative of 1:100 AEP event
- MCE with a PGA of 0.75g from magnitude 7.5 earthquake on Whangamoa Fault

It is unclear if deterministic MCE ground motions are based on the 84th percentile (Mejia, 2001) in accordance with standard practice.

A desktop study was performed by GNS in 1993 (GNS, 1993). The study included a review of Maitai Dam's geology, active faulting and strong ground motion. Conclusions of this study are summarized below:

- No active faults are known to cross the Maitai dam site (footprint).
- The recurrence intervals and timing of past movements is unknown.
- Area faults included major northeast-trending Flaxmore, Eight-eight, Waimea and Whangamoa faults.
- Faults in the vicinity of Nelson City include intermittent trace on the Whangamoa and traces on the Bishopdale and Teal faults.
- The MCE for Maitai Dam site re-estimated to be a magnitude 7.8 earthquake on the Waimea fault at 5-14 km from the dam site.
- Re-evaluate the MCE response spectrum using recent spectral acceleration models appears warranted based on low values at all periods.
- Uniform hazard spectra should be reviewed with respect to the standard 150 AEP earthquake (i.e., the Operating Basis Earthquake, OBE).
- It may be important to quantify the probability of occurrence of the MCE by further detailed studies of the active faults in the east Nelson region.

The 2003 CSR (Tonkin & Taylor, 2003) cites an "assessed MCE for the site has a ground acceleration of 0.6g at the dam site". It is not clear if this is based on their review using the at that time current seismic hazard model (Stirling et a. 2000). No response spectrum is provided for the revised ground motion.

<u>NZSOLD Design Earthquake Loads</u>
The NZSOLD Dam Safety Guidelines (NZSOLD, 2000) recommend two levels of earthquake design loads. The Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE). For existing dams the Safety Evaluation Earthquake (SEE) is used instead of the MDE term. The OBE and SEE are the minimum and maximum service level of ground motions to evaluate the design and performance of the dam, respectively. The OBE represents a 1:150 annual exceedance probability (AEP) earthquake. For High PIC, the SEE is either the deterministic Maximum Credible Earthquake (MCE) or the probabilistic 1:10,000 AEP ground motions. Post mainshock static stability as well as aftershock shaking should also be checked. Aftershock earthquake is typically taken as one magnitude less than the SEE.

Updates to the National Seismic Hazard Model

Since the GNS desktop study and the 2003 CSR review there have been recent developments in estimating ground motions:

- The National Seismic Hazard Model (NSHM) was updated in 2010 by Stirling et al. (2012) for performing probabilistic seismic hazard assessment
- Ground Motion Prediction Equations (GMPE) used to estimate PGA and acceleration response spectrum in the NSHM are those by McVerry et al. (2006).
- Updated GMPE are recommended by Bradley (2010).

These recent updates in the NSHM and GMPE incorporates new fault sources, and utilises newly-developed New Zealand-based scaling relationships and methods for the parameterisation of the fault and subduction interface sources. The distributed seismicity model has also been updated to include new seismicity data, a new seismicity regionalisation, and improved methodology for calculation of the seismicity parameters. The new seismic maps show a decrease in the estimated hazard for the Nelson region from hazards maps based on the 2002 NSHM (references Stirling et al. 2012).

Maitai Dam Ground Motions

NZSOLD Guidelines recommended a site specific seismic risk study with response spectra be developed for High PIC dams because of the hazards they pose. Given Maitai Dam is a High PIC dam and there have been significant changes to seismic understanding, (**Rec-04) It is recommended a site specific seismic risk study be performed for Maitai Dam.**

An understanding of ground motions for Maitai Dam using the 2002 NSHM provides some basis for determining structural earthquake design loads under New Zealand Standard (NZS) 1170.5:2004, Structural Design Actions, Part 5: Earthquake actions (New Zealand Standards, 2004). The current edition will be revised to incorporate the updated NSHM. Table 5 provides PGA using the current standard for reference. Ground motions for dam assessments however should be based on a site specific of the ground motions study for High PIC dam. Additionally, the new Dam Safety Scheme as part of the Building (Dam Safety) Regulations (2008) will require dams in earthquake zones to meet criteria at the 1:500 AEP event.

Table 5: Earthquake Peak Ground horizontal Accelerations for Maitai Dam Assuming Rock Site (New Zealand Standards, 2004)

Return Period (years)	Peak Ground Acceleration, PGA (g)
50	0.09
100	0.14
150	0.16
500	0.27

4.3 Landslides

4.3.1 Hazards of Concern

In determining the hazard that landslides pose to a dam safety there are two key considerations:

- Slope failures that are large enough to block the reservoir (creating an upstream dam) or the dam's spillway (impeding safe flood passage).
- A rapid slope failure generating seiche (impulse) waves in the reservoir that overtop the dam.

Landslides may occur under everyday ('sunny day') conditions, but are usually associated with heavy rainfall and infiltration, earthquakes or in the case of dams due to the presence of a reservoir that did not exist previously. The first filling and ongoing operation of a reservoir can mobilise landslides (historic or new) around the reservoir rim, as slopes, and in particular their toes, become submerged and saturated. Rapid drawdown of a reservoir can also cause landslides due to the effect of hydrostatic load being removed from the toe of a saturated slope.

Geology of the area as described in Section 3 includes moderate to steep topography in Bryant Range. Hillsides are covered primarily with native vegetation. Bedrock consists of mudstone, siltstone, and sandstone which are tightly jointed, and form part of the Greville Formation of the Maitai group. The region has a number of dormant and active north east trending faults (reference).

Regionally, landslides are common in the Nelson area. For example the December 2011 severe rainstorm resulted in extensive flooding and landslips in the Nelson City and Tasman District. This rainstorm equated to a 1: 200 AEP storm event. Along the Maitai pipeline, there were a number of smaller and larger landslips along the alignment (Ellis, 2013). Slope failure of slaking and slabbing also occurred in the rock cliffs along Rocks Road section of State Highway 6 along the coast (Stewart, 2013).

4.3.2 Reservoir Rim Stability

Reservoir rim landslides should be anticipated under high earthquake shaking, such as that produced by an MCE event. With respect to the Maitai reservoir, two key considerations for slopes that fail in an earthquake are:

- Would there be sufficient volume to block the reservoir, and
- Would the generated impulse wave be safely passed by the Maitai service spillway and not fuse the auxiliary spillway, or at worst cause a short period of overtopping.

A cursory review of Google aerial photography indicates no previous or existing reservoir rim instability. In part this is supported by no incidents of reservoir rim slope instability in previous CSRs. Documented slips along the reservoir rim are historically stable with minor movement. Vegetated slopes provide some resistance to slip failures in saturated soils due to heavy rainfalls. The forest plantation near the inlet of North Branch the on south slopes maintains a buffer along the reservoir rim. Borrow site for dam construction along the east side is a visibly gently sloped area along the reservoir rim.

5.0 POTENTIAL FAILURE MODES AND CHARACTERISTIC BEHAVIOUR

The understanding of a dam's credible potential failure modes and characteristic behaviour provides the basis for comprehensive assessment of its ongoing safe performance. Potential failure modes are the most likely credible ways that a dam will fail, if it was to fail, and therefore do not imply that failure is imminent by any of the identified modes. Understanding a dam's potential failure modes and characteristic behaviour allows operation, surveillance and maintenance activities to be targeted directly at the areas of importance and thus provide best prevention, or at worst, early detection of the initiation of a dam failure mechanism. The same applies to safety reviews of dams in ensuring that issues relevant to the safety of the dam are not overlooked.

Failure modes have not been formally developed for Maitai Dam. Typically failure modes are developed by an independent process using technical experts. Historical information on design and construction and performance of the dam under normal and unusual conditions are valuable inputs to form a collective judgment of the dam's credible failure modes. (**Rec-05) It is recommended potential failure modes for Maitai Dam be developed.**

The characteristic model of the dam forms a reference in determining whether observed behaviour is expected and acceptable within appropriate performance limits. Expected behaviour is best drawn from a characteristic model that combines the dam's historical performance with an engineering appreciation of the nature of the dam, its foundation, its operational context and the interaction between. Surveillance information of observations and instrumentation data can then be interpreted based on expected behaviour. (**Rec-06) It is recommended a characteristic model of Maitai Dam be developed for interpretation of surveillance information**.

6.0 Site Inspection Detail

The safety review dam inspection was made on 16th January 2014. No testing of the low level dewatering structures was carried out.

6.1 Safety Review Team

The Safety Review team comprised of :

Brian Benson	Lead Examiner & Author	Principal Geotechnical Engineer Damwatch Engineering Ltd.
Karina Dahl	Support Examiner & Author	Senior Geotechnical Engineer, Damwatch Engineering Ltd.

The dam inspection was attended by:

Howard Schuppan	Team Leader Utilities	Nelson City Council
Alex Miller	Technician	Nelson City Council

During the site inspections the review team was also assisted by:

Trevor Ruffell	Dam Caretaker	Fulton Hogan
Richard Kennedy	Water Treatment Plant Manager	Fulton Hogan

<u>Summary of Key Observations</u>
Maitai Dam appears to be in satisfactory condition. No major signs of deformation or increased risk were identified. The key issues identified during the inspection of Maitai Dam are:

1. Pipework inside the valve chamber including the scour pipeline has significant rusting which needs maintenance. (**Rec-07) It is recommended the rusted pipework be cleaned and repainted.**
2. New seepage was observed emerging from a step at the interface with the south retaining wall on the right hand side of the culvert downstream end. (**Rec-08) It is recommended that seepage emerging adjacent to the culvert be monitored with documentation on its development.**
3. The chute floor on the service spillway shows turbulent flow over surface cavities and level differences at some of the floor joints. **(Rec-09) It is recommended the repairs are made to ensure a smooth finish on the chute floor of the service spillway.**
4. Significant vegetation along the spillway chute walls need to be cleared. (**Rec-10) It is recommended the spillway chute walls be cleared of trees and bushes to facilitate inspection and prevent damage to the wall from root growth.**

Table 6: CSR Site Inspection Overview

Date/Time	16/01/2013 10:30am to 3:30pm Pre-inspection briefing ~9:30 to 10:30am
Inspection Personnel	Howard Schuppan and Alex Miller of NCC Trevor Ruffell and Richard Kennedy of Fulton Hogan Brian Benson and Karina Dahl of Damwatch
Safety Briefing	Damwatch Inspectors were under the supervision of NCC & Dam operator personnel (Fulton Hogan) at all time.
Weather	Fine
Reservoir Level	R.L173.75 (Spillway Crest Level)
Photographs	Photographs taken during the dam inspection are in Appendix C of this CSR

Table 7: CSR Site Inspection Details

Item	Comments/Action Required
1. Dam Embankment Inspect: - Upstream slope and abutment contacts for missing rip-rap, slips, surface erosion.	Upstream slope above waterline is in satisfactory condition. No evidence of new/recent damage or movement in the rip-rap surface. Replacement of stones previously removed evident. Overall in good condition (Photos 1-4).
- Crest for deformation, sink holes, slumps, settlement, cracking, slips, surficial erosion	Crest is in satisfactory condition with a minimum grass cover. No cracks or dips were observed. All wooden slats were in place along the crest parapet wall (Photo 5).
- Downstream slopes and abutment contacts for seepage, damp areas, cracking, slips, surficial erosion, settlement, unusual vegetation.	Downstream slope is in satisfactory condition. Vegetation control is managed by mowing and sheep grazing for clear visual inspection of slope. No observed seeps, wet areas or slips were observed (Photos 6-9).
- Right Abutment for deformations, slips, surficial erosion.	Right Abutment –Slip along natural ground observed with no evidence of recent movement and start of grass growth. Rock drainage along groin. No seepage or wet areas observed (Photos 10 and 1).
- Left Abutment for deformations, slips, surficial erosion.	Left Abutment – scattered reeds observed along the slope groin area and at the berm interface. This is attributed to collection of rain runoff. No evidence of slipping or other indicators of instability (Photos 12-16).

Item	Comments/Action Required
2. Culvert Inspect: - Bay joints and joint filler condition	Continued leakage observed at historical points primarily in degraded joints along right hand side of culvert but also affecting pipe bracing on left hand side (Photos 17-19).
- Structural concrete for damage, cracking, deformation, abrasion and spalling	Precipitation of minerals and bacteria growth observed, this bacteria growth poses a Health & Safety (H&S) and proper air ventilation must be maintained. Concrete otherwise in good condition with minor shrinkage cracks and spalling. Aggregates are exposed on floor surface from flood flows experienced during construction.
Valve Chamber Structural concrete for damage, cracking, deformation, abrasion and spalling	Minor cracking in concrete walls.
Pipes and valves – steel condition, rust,	Areas along pipes especially at contact with chamber wall observed covered in rust. Photo 20 shows rust on scour inlet and adjacent wall. (**Rec-07) It is recommended the rusted pipework be cleaned and repainted.** Valves were not operated.

Item	Comments/Action Required
3. Monitoring Facilities Inspect: - Embankment seepage: Left and Right Abutments Contact Drains	Observed low clear seepage flow from left abutments contact drain. Gas detector alarm sounded in manhole, repeat previous recommendation for H&S to change method of seepage flow monitoring (Photo 21).
Central collector Drain exit area - total flow, - clear flow measured, - Check seepage water quality	Seepage from collector outlet was measured to be 20sec for 46L (2.3l/s), which is within normal range.
Embankment and abutment standpipe piezometers	Surface seal and caps intact in embankment and abutment standpipe piezometers(Photo 22).
Hydraulic Piezometers	Hydraulic piezometers de-aired in October 2013, well maintained condition of gauges in control room.
Chimney Drain (R.H.S. & L.H.S.)	No seepage observed emerging from chimney drain, PVC pipe outlet clear of blockage (Photo 23).
Culvert Interceptor Drain – low flow, consistent with expected	Low and normal clear flow observed from culvert interceptor drain (Photo 24).
Left and right culvert exits area drains	No seepage observed from either the right or left drains (Photo 25).Seepage was observed emerging from corner of first step on right and retaining wall; this is a new seepage. Leakage may be due to blockage of drain or damage at pipe joint (Drawing 6516-15 AB) (Photos 25-26). (**Rec-08) It is recommended that seepage emerging adjacent to the culvert be monitored with documentation on its development.**
Spillway seepage manholes (1-3) - Check seepage water quality	Nil with moist to dry conditions observed.
- Deformation Survey marks. - Deformation pins	Survey markers and pins in good condition.

Item	Comments/Action Required
5. Service Spillway	
Inspect: - Joints and joint filler condition	Joints on invert have some level differences and need infilling at some locations (Photo 27).
- Structural concrete for damage, cracking, deformation, abrasion and spalling	The spillway concrete invert includes areas needing patching (Photo 28). **(Rec-09) It is recommended the repairs are made to ensure a smooth finish on the chute floor of the service spillway.**
- Spillway channel surface for unusual flow paths, ridges.	Unable to inspect spillway floor and drains due to flow. Reservoir at spillway crest at time of inspection, wind caused low intermediate flow. A preferential flow along the right side observed (Photo 29).
- Spillway channel sidewalls for tilting -	No significant movement across chute wall joints observed and monitored pins found to be intact. Significant vegetation was observed along the spillway chute right wall and some along the left wall (Photos 30 and 31). (**Rec-10) It is recommended the spillway chute walls be cleared of trees and bushes to facilitate inspection and prevent damage to the wall from root growth.**
- Upstream for blockage.	Debris boom intact and retaining small tree branches at time of inspection
- Flip Bucket	Flip bucket contained water, concrete observed to be intact. Seepage observed emerging from apron drains (Photo 32). The dive inspection performed in August 2013 included clean out of the drain and inspection of rip-rap with no sign of erosion. Concrete on apron has minor cracks.
Plunge Pool	Plunge pool filled and slopes protected by with rip-rap and vegetation, no erosion or slips observed.

Item	Comments/Action Required
6. Auxiliary Spillway (Fuse Plug) Inspect: - Upstream concrete for damage, cracking and spall	Upstream concrete face and walls in good condition, minor cracking. (Photo 33).
-Upstream slope, drainage gravel	Upstream and crest drainage gravel intact with no deformations or erosion observed. (Photo 33).
- Crest and Fuse Triggering Troughs for damage, blockage, cracking,	Fuse triggering troughs intact with no damage, minor rust on cover plates.
- Downstream slope and outlets for vegetation, soil over-compaction, damage, blockage	Downstream sand embankment is clean and maintained in a satisfactory consistency to collapse under fusing (Photos 34 and 35). The V-shaped channel was clear of debris for no blockage during potential fuse plug flows and includes exposed bedrock on left side (Photo 36). Observed normal seepage conditions at downstream toe of sand embankment (Photo 37)
7. Reservoir Area and Perimeter: - Inspect shorelines for slips, slope movement, erosion	Forestry plantation observed along southern rim with vegetation buffer provided (Photos 38 and 39). The shoreline includes three historic relatively stable slips with only minor sloughing of colluvium observed (Photos 40 and 41).
8. Water Supply Intake Tower/General - Steel pipes (above water level).	Steel pipe observed above water in good condition and maintained with paint coverage (Photos 42 and 43).
-Screen hoisting equipment	Screen hoisting wench missing at one intake (Photo 44).
- Condition of cathodic protection	Recent cathodic protection maintenance providing readings within expected levels compared to 2013 intermediate inspection. Current readings were a voltage reading of ~29 V and current meter reading 2amps (Photo 45).

Item	Comments/Action Required
8. Dam Safety Critical Plant - Water supply line valve, check valve operations over full range - Bypass scour valve, check valve operations over full range	No operation of water supply or scour valves were performed as part of the CSR inspection. New duplicate water supply line observed at downstream end of culvert area. New section includes capability to make temporary releases to the stream channel (Photos 46 and 47). Rip-rap protection in channel in good condition and recently upgraded with larger stone (Photo 47)
9. Document Review Review: - Monthly Inspection Sheets - Deformation Survey Data - Previous Annual Inspection Report	 Monthly sheets from last 12 months sighted Deformation survey data provided Annual report from 2013 sighted Surveillance data provided for interpretation

7.0 SURVEILLANCE REVIEW

Surveillance and monitoring of Maitai Dam includes the following aspects:

- Continuous lake level monitoring by sensor and SCADA.
- Monthly routine visual inspections and instrument reading by the dam Caretaker of:
    - Embankment and foundation hydraulic piezometers (27),
    - Embankment standpipe piezometers (2),
    - Abutment standpipe piezometers (4),
    - Abutment drains, central chimney drain, and culvert interceptor drain with time flow seepage measurement, and
    - Spillway underdrain with time flow seepage measurement.
- Monthly processing and archiving of inspection and instrument records by NCC Utilities.
- Intermediate (Annual) Dam Safety Reviews by an External Consultant.
- Annual Dam Deformation Survey.

7.1 Reservoir Level

7.1.1 Purpose

Reservoir level is an important dam safety monitoring parameter in that it provides a direct measure of the dam's normal loading condition. A measure of loading condition is essential to assess dam performance by correlation with other observations (visual and measured). Records of reservoir level also identify periods of operation outside normal operating limits and exceedance of precedent reservoir levels. At higher reservoir levels than precedent the dam is subject to pressure and seepage behaviour never before experienced. This may impact on the dam's integrity and need to be monitored with increased vigilance.

7.1.2 Measurement

Maitai reservoir is measured by an automated water level sensor and archived on Nelson City Council's (NCC) Historian database. Daily readings for the period 1/3/2009 to 1/3/2014 have been provided for this CSR. A staff gauge is available at the spillway entry to measure water level manually during emergency inspections and calibration checks of the automated sensor.

The full reservoir level record contains monthly readings commenced in 1986 and automated half-hourly readings commenced in 2004.

7.1.3 Analysis

In the analysis period for this review (2009 to 2014) Maitai reservoir ranged between R.L. 171.80m and R.L. 175.04m (refer Figure 6). The latter is the historic high level on record for Maitai reservoir (occurred on 28/12/10), corresponding to 1.29m depth of spill over the service spillway crest.

The historic high reservoir level (R.L. 175.04 m) was 0.04m above the top of the chimney drain. Freeboard to the sand fuse embankment crest was 0.57m (although only 0.14m to the sill of the auxiliary spillway). Freeboard to the dam crest was 1.96m.

7.2 Visual Observations

Visual observations are a key performance indicator for Maitai Dam. Detailed inspections are completed monthly by the on-site dam Caretaker, who has been inspecting the dam since construction, has a high level of knowledge of its performance behaviours and is competent in performing duties for operations and maintenance of the dam. Inspections are completed using detailed checksheets.

Scanned records of the completed checksheets were provided for the CSR review period. No reports of significant abnormal behaviour have been made in the record sheets since the last CSR.

Inspections of Maitai Dam are performed at the following frequency:

- monthly routine visual inspections and measurements
- annual visual inspections and measurements including annual deformation surveys
- five-yearly inspections as part of comprehensive safety review

7.3 Seepage Control

7.3.1 Features and Design Philosophy

Maitai is a nearly homogeneous embankment dam with a central chimney drain and downstream abutment and foundation drainage. The chimney drain connects to the foundation blanket drain and a central collector drain. The culvert has an interceptor drain 10m upstream of the dam centreline. On the R.H.S. of the culvert downstream bays there is a drain to convey seepage to the culvert exit area. Seepage on the left side of the culvert is conveyed by the foundation drainage blanket. The left and right abutments are provided with contact blanket drains.

The dam's designers expected that seepage through the embankment would be minimal (0.8 litres per second), and the chimney drain would allow the downstream embankment to remain dry. The design report (Tonkin & Taylor, 1986) states, regarding design flow net assumptions, "*phreatic line on downstream side of chimney drain permitted to rise to R.L. 144m (but no higher to maintain high downstream slope stability and reduce leakage potential at culvert joints*) *to assist base drainage*".

Manholes 1-3 collect seepage from the spillway underdrains beneath the chute slabs and behind the chute walls.

Expected seepage flows and piezometric levels are based on values cited in the Commissioning Report (Tonkin & Taylor, 1989).

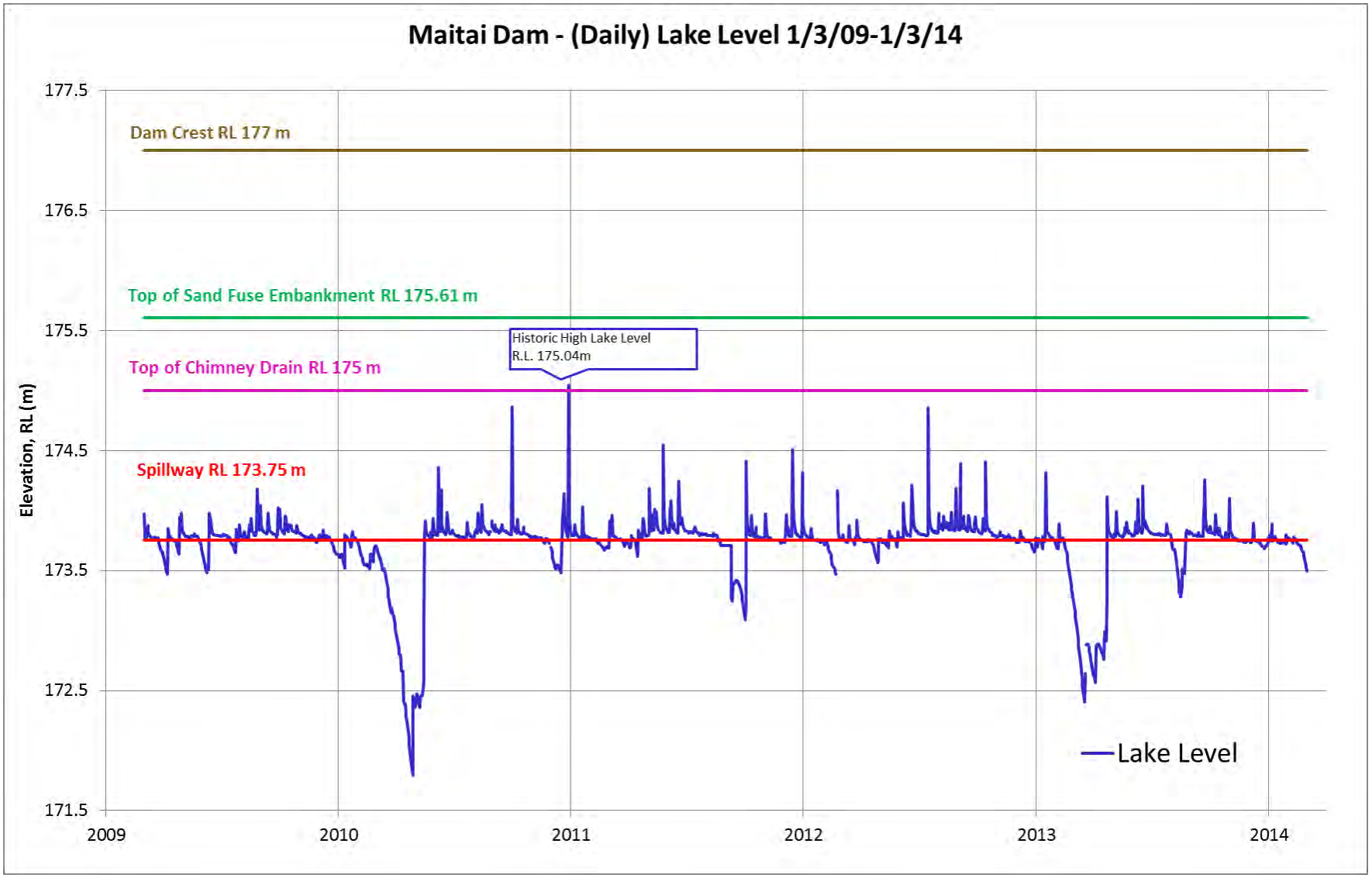

Figure 6: Maitai Dam (Daily) Lake Levels 1986-2014

7.3.2 Piezometric Pressure Monitoring

Piezometric pressure is a key performance indicator for Maitai Dam and foundation.

Foundation and Embankment
There are a total of 27 hydraulic piezometers to monitor water pressure in the foundation (12) and embankment (15). They are located on three dam cross-sections named Instrumentation Line 1, 2 and 3 (refer Instrumentation Layout and Details Drawing 6516-12AB and Geological Plan of Foundations Drawing 6516-G4). The piezometers are read monthly via gauges in the Control building on the dam toe. The piezometers were last de-aired in October 2013 [refer (Tonkin & Taylor, 2014) in Appendix D]. Piezometer gauges were not calibrated.

Standpipe piezometers B23 and B27 were installed around 1990, in an effort to verify data from hydraulic piezometers P23 and P27, located in the upper embankment downstream of the chimney drain (Instrument Lines 1 and 3 respectively). There are no design or as-built drawings available for B23 and B27. (**Rec-11) It is recommended that as-built location and installation details for B23 and B27 be confirmed and indicated on a drawing for the purpose of ongoing data evaluation.**

Piezometer readings from 2/1/14 are presented spatially on the respective three dam cross-sections (Figure 7). Time-series data for the hydraulic and standpipe piezometers are presented in Figures 8 to 15.

Foundation and Embankment Observed Behaviour
Instrument Lines 1, 2 and 3 piezometer expected and observed behaviours are presented in Tables 8, 9 and 10 respectively.

Table 8: Instrument Line 1 (Maximum Section) Piezometers (U/S to D/S order)

Name	Location	Expected Behaviour	Observed Behaviour
P1	**5m in foundation** near upstream toe 78m u/s of chimney drain	Expect responsive to reservoir due to fault zone	**Closely responsive to reservoir (within 1m)**
P2	**12m in foundation** 36m u/s of chimney drain	Expect responsive to reservoir due to fissile rock foundation	**Closely responsive to reservoir (2m below)**
P8	In embankment 37m u/s of chimney drain	Expect responsive to reservoir	Closely responsive to reservoir (within 0.5m)
P3	In embankment approx. 5m above foundation 26m u/s of chimney drain	Expect may have some response to reservoir but significant head loss through fill	Some response to reservoir (reads approx. 13m below). Pressure steadily decreasing with time.
P22	High in embankment 11m u/s of chimney drain	Expect responsive to reservoir	Closely responsive to reservoir (essentially reads reservoir level)
P23	High in embankment **7m d/s of chimney drain**	Expect dry due to chimney drainage	**Piezometer tip appears to measure pressure**
B23	High in embankment **near P23 d/s of chimney drain.** Exact location unknown.	Expect dry due to chimney drainage	**Standpipe appears to measure water level**
P4	In embankment 7m d/s of chimney drain approx. 5m above foundation	Expect dry due to chimney drainage	Piezometer tip appears to not have pressure on it
P9	In embankment 20m d/s of chimney drain in Zone 1 just above Zone 1A	Expect dry due to chimney drainage and permeability of Zone 1A below	Piezometer tip appears to not have pressure on it
P5	2m in foundation 21m d/s of chimney drain	Expect relatively low pressure although possibly some drainage blanket influence	Piezometer reads 1m above tip (approx. top of blanket drain). Piezometer under negative pressure. **Plot indicates air in the lines (removed during periodic de-airing)**
P6	In embankment 52m d/s of chimney drain in Zone 2 approx. 5m above foundation	Expect dry due to blanket drainage and permeability of Zone 2	Piezometer tip appears to not have pressure on it
P7	2m in foundation 90m d/s of chimney drain	Expect low pressure although possibly some drainage blanket influence	Piezometer reads approx. 2m above tip (approx. foundation level). Piezometer under negative pressure. **Plot indicates air in the lines (removed during periodic de-airing)**

Table 9: Instrument Line 2 (Right Abutment) Piezometers (U/S to D/S order)

Name	Location	Expected Behaviour	Observed Behaviour
P10	**4m in foundation** near upstream toe 28m u/s of chimney drain	Expect relatively low pressure in competent rock	**Closely responsive to reservoir (essentially reads reservoir pressure)**
P11	**On foundation** 19m u/s of chimney drain	Expect some response to reservoir although significant head loss through fill or along foundation	**Closely responsive to reservoir (approx. 9m below)**
P24	High in embankment 11m u/s of chimney drain	Expect responsive to reservoir	Responsive to reservoir (within 2m) with lag
P25	High in embankment **8m d/s of chimney drain**	Expect dry due to chimney drainage	**Piezometer unreliable**
P12	3m in foundation 11m d/s of chimney drain	Expect relatively low pressure due to height on valley	Piezometer reads approx. at tip. **Plot indicates air in the lines (removed during periodic de-airing)**
P13	In embankment 29m d/s of chimney drain in Zone 1A approx. 5m above foundation	Expect dry due to chimney drainage and permeability of Zone 1A	Piezometer tip appears to not have pressure on it
P14	In embankment 51m d/s of chimney drain in Zone 2 approx. 5m above foundation	Expect dry due to abutment drainage and permeability of Zone 2	Piezometer reads approximately at tip
P15	2m in foundation 61m d/s of chimney drain	Expect low pressure	Piezometer reads below tip

Table 10: Instrument Line 3 (Left Abutment) Piezometers (U/S to D/S order)

Name	Location	Expected Behaviour	Observed Behaviour
P16	**5m in foundation** 27m u/s of chimney drain	Expect low pressure in competent rock	**Closely responsive to reservoir (reads 3m below)**
P17	**On foundation** 14m u/s of chimney drain	Expect some response to reservoir although significant head loss through fill or along foundation	Responsive to reservoir (approx. 13m below)
P26	High in embankment 8m u/s of chimney drain	Expect responsive to reservoir	Responsive to reservoir (within 3m)
P27	High in embankment **9m d/s of chimney drain**	Expect dry due to chimney drainage	**Piezometer appears to measure pressure and respond to reservoir (approx. 4m below) with lag**
B27	High in embankment **near P27 d/s of chimney drain.** Exact location unknown.	Expect dry due to chimney drainage	**Standpipe appears to measure water level and respond to reservoir (approx. 5m below) with lag**
P18	1m in foundation 5m d/s of chimney drain	Expect relatively low pressure due to height on valley	Piezometer responsive to reservoir (approx. 20m below)
P19	In embankment 27m d/s of chimney drain in Zone 1A approx. 5m above foundation	Expect dry due to chimney drainage and permeability of Zone 1A	Piezometer tip appears to not have pressure on it
P20	In embankment 57m d/s of chimney drain in Zone 2 approx. 2m above foundation	Expect dry due to abutment drainage and permeability of Zone 2	Piezometer reads approximately at tip
P21	On foundation 70m d/s of chimney drain	Expect low pressure	Piezometer reads below tip. **Plot indicates significant air in the lines (removed during periodic de-airing)**

<u>Foundation and Embankment Interpretation</u>

Piezometric behaviour is generally as expected, with embankment pressures responsive to reservoir level upstream of the chimney drain, and generally dry conditions downstream of the chimney drain. However, some exceptions are observed and interpreted as follows.

Foundation pressures beneath the upstream half of the embankment are high at P1, P2, P10, P11, P16 and P17. This is likely to be due to fault zones and fissile rock observed in the foundation during construction. High foundation pressures do not occur beneath the downstream half of the embankment.

Based on readings at P23 and B23 (maximum dam section), and P27 and B27 (over left abutment), there appears to be localised water pressure high in the embankment downstream of the chimney drain. P25 (over right abutment) is unreliable. Such observations purport a condition of phreatic surface downstream of the chimney drain, which departs from the design intent and may represent an adverse condition. (**Rec-12a) It is recommended that the purported water pressure in the upper embankment downstream of the chimney drain be thoroughly investigated and resolved. This should include an assessment of the reliability of the instruments and measured data, and consideration given to supplementary monitoring in this location.**

A number of piezometers read pressure close to their tips. However, taking into account gauge calibration, real tip height (accounting for embankment settlement since installation), and possible thermal effects of water in the leads (near downstream face of dam) it cannot be certain that this is real water pressure or just the head of water between the leads and gauge house. The recommended instrument reliability assessment **(Rec-12a)** should aim to address these factors. (**Rec-12b) It is recommended all piezometer gauges should be calibrated to be accurate in their respective normal reading ranges**.

Piezometers P5, P7, P12 and P21 accumulate air in the leads as evidenced by their signature increasing head with time and (temporary) resumption to normal behaviour following de-airing. (**Rec-13) It is recommended that piezometer de-airing operations continue at a frequency appropriate to observed accumulation of air in their data plots.**

<u>Abutment Standpipe Piezometers</u>
Standpipe piezometers SB1-3 allow water level measurement in the left abutment ridge between the spillway and the South Branch Maitai River, while BH7 provides the same on the right abutment downstream of the dam centreline (refer Seepage Measurement and Standpipe Locations Drawing 12446). Top of hole reduced levels are provided, however, bottom of hole reduced levels are not stated. (**Rec-14) It is recommended that bottom of hole reduced levels be established for SB1-3 and BH7 to support ongoing data evaluation**.

Time-series data for the abutment standpipe piezometers are presented in Figures 16 and 17.

Abutment Standpipe Piezometers Observed Behaviour

Abutment standpipe piezometer expected and observed behaviours are presented in Table 11.

Table 11: Abutment Standpipe Piezometers

Name	Location	Expected Behaviour	Observed Behaviour
SB1	On left abutment ridge above spillway with cap R.L. 182.4m	Expect response to reservoir. Also expect influence by rainfall infiltration and natural groundwater.	Responds to reservoir (approx. 6m below) and may be influenced by rainfall.
SB2	On south side of left abutment ridge with cap R.L. 165.2m		Responds to reservoir (approx. 14m below) and may be influenced by rainfall.
SB3	On south side of left abutment ridge with cap R.L. 152.77m		Responds to reservoir (approx. 26m below) and appears influenced by rainfall.
BH7	On right abutment d/s of dam centreline with cap R.L. 180.25		Responds to reservoir (approx. 6m below) and appears influenced by rainfall.

Abutment Standpipe Piezometers Interpretation

Piezometric observations in the left and right abutments are as expected and no anomalous trends are identified. Hydraulic gradients appear to be satisfactory.

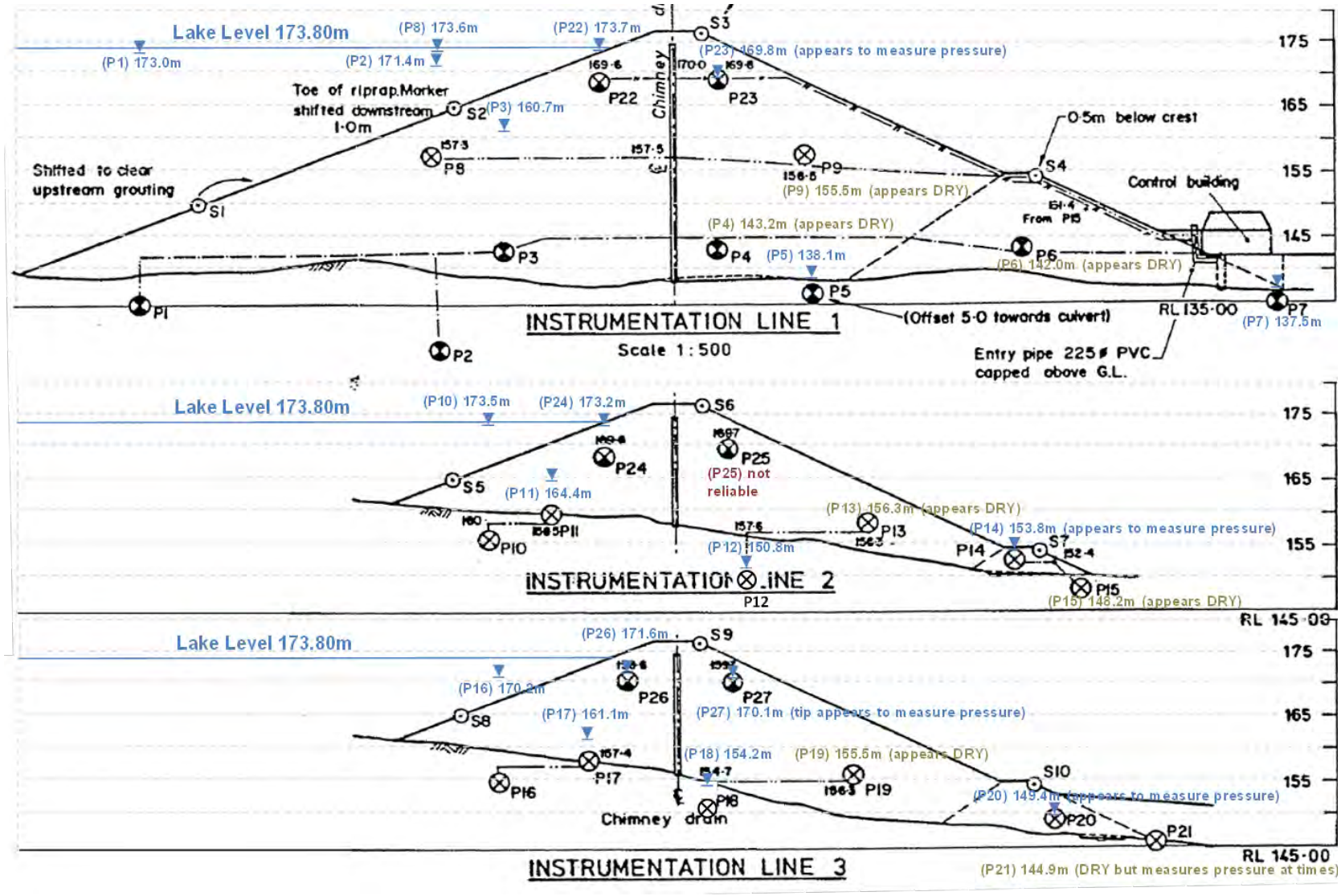

Figure 7: Maitai Dam – Piezometer Instrument Lines 1, 2 and 3 Cross Sections with 2/1/14 Readings

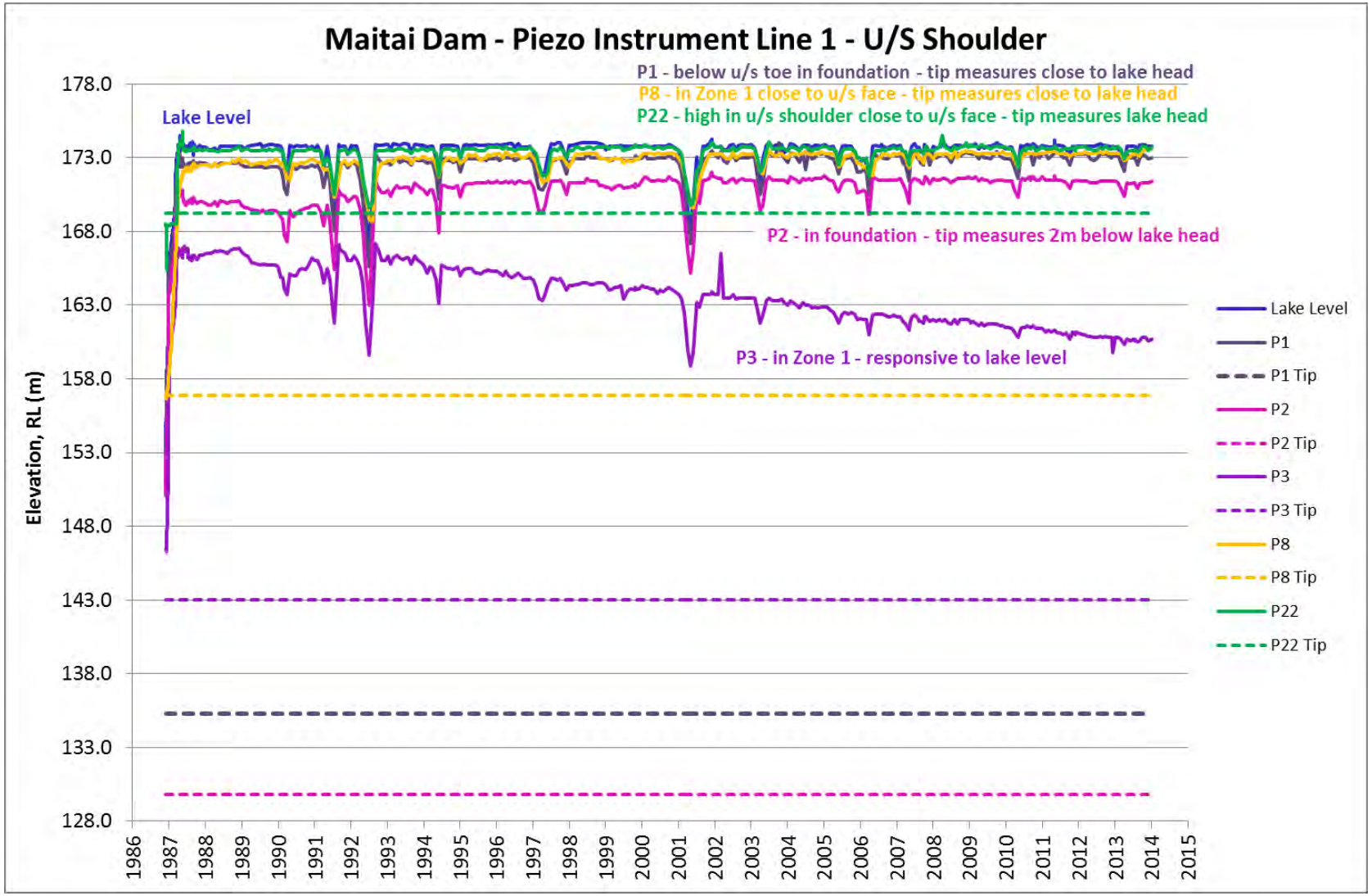

Figure 8: Maitai Dam – Piezometer Instrument Line 1 – Upstream Embankment and Foundation (1986-2014)

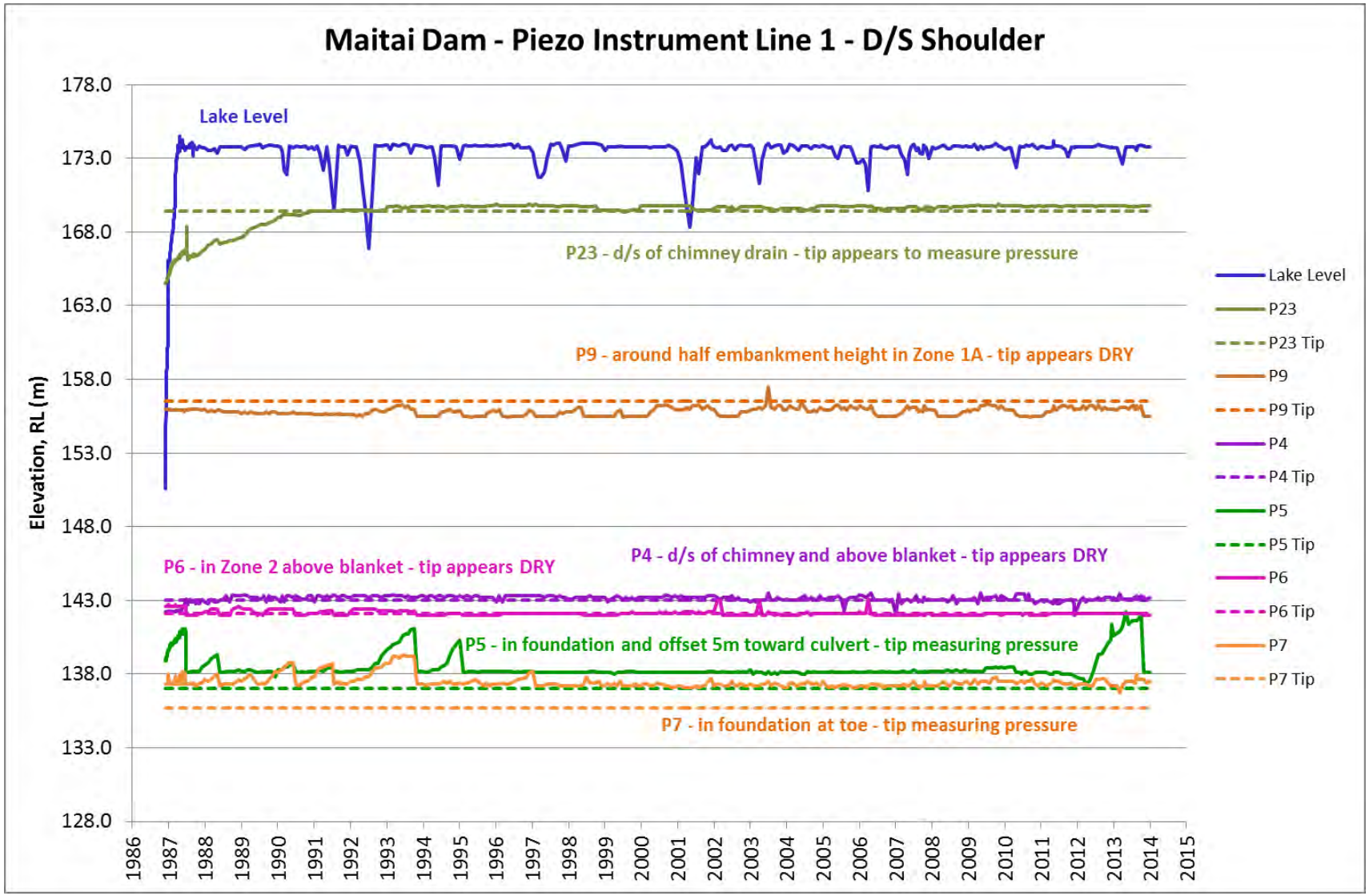

Figure 9: Maitai Dam – Piezometer Instrument Line 1 – Downstream Embankment and Foundation (1986-2014)

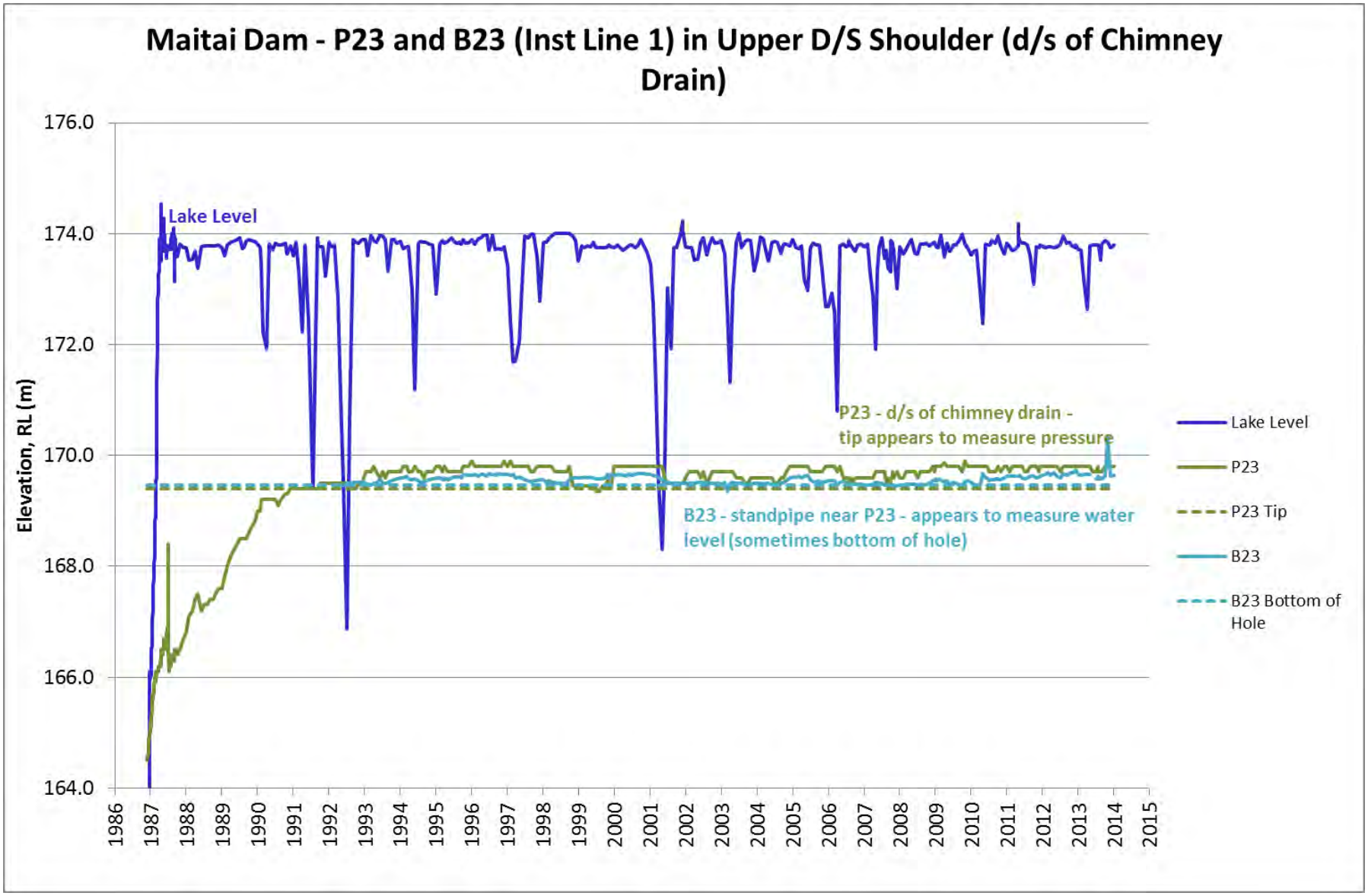

Figure 10: Maitai Dam – Piezometer Instrument Line 1 – P23 and B23 Upper Downstream Embankment (1986-2014)

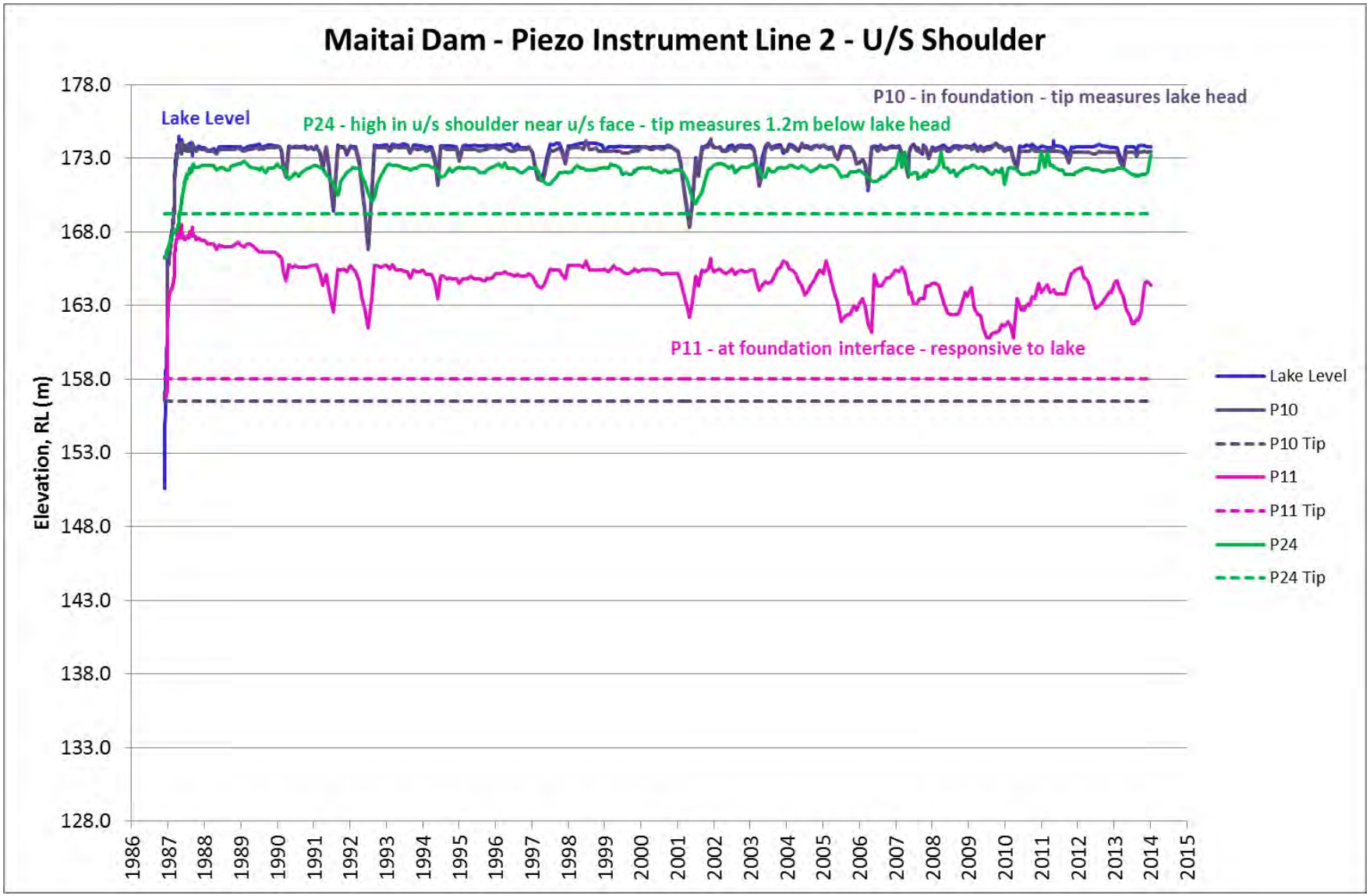

Figure 11: Maitai Dam – Piezometer Instrument Line 2 – Upstream Embankment and Foundation (1986-2014)

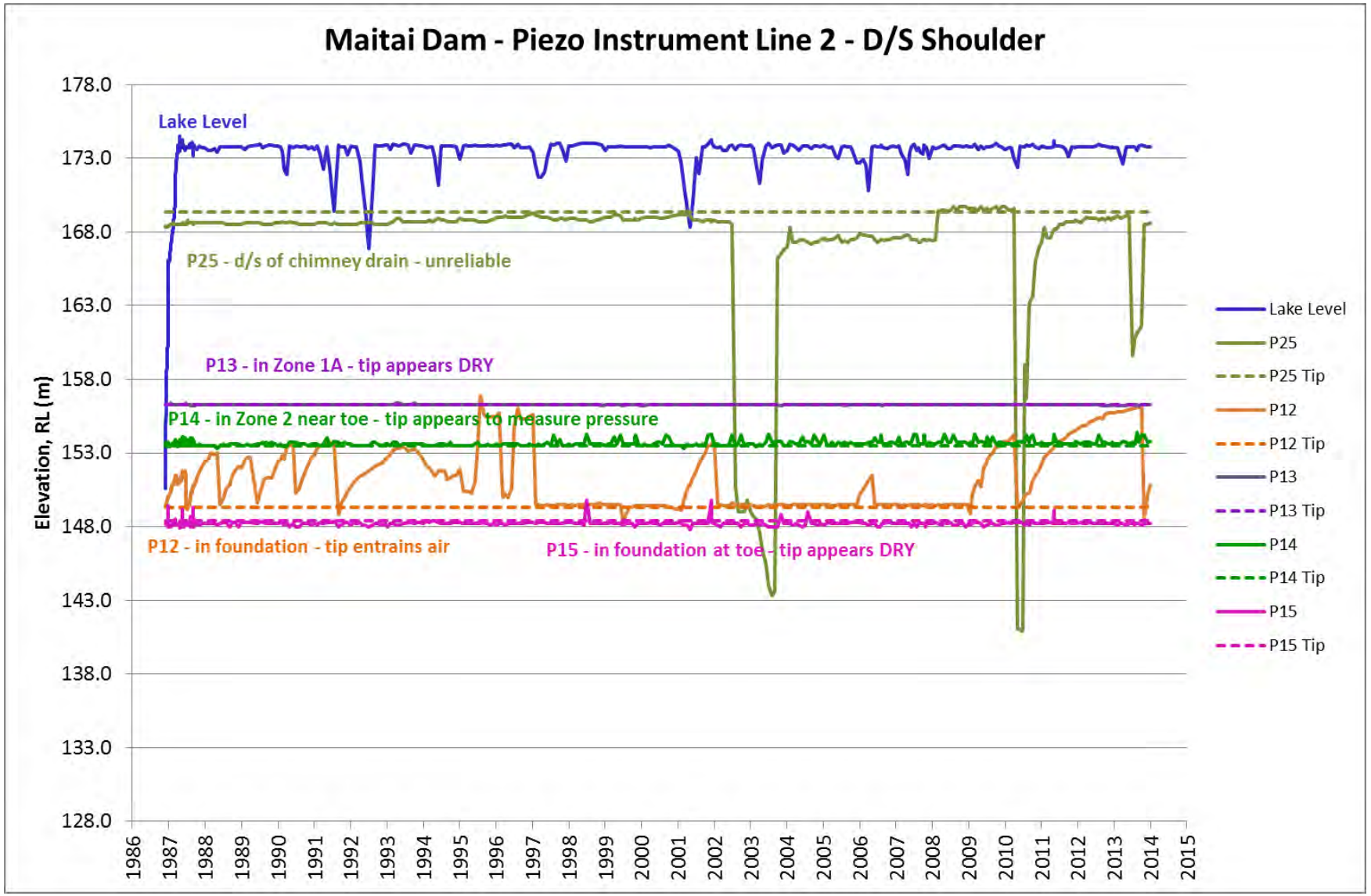

Figure 12: Maitai Dam – Piezometer Instrument Line 2 – Downstream Embankment and Foundation (1986-2014)

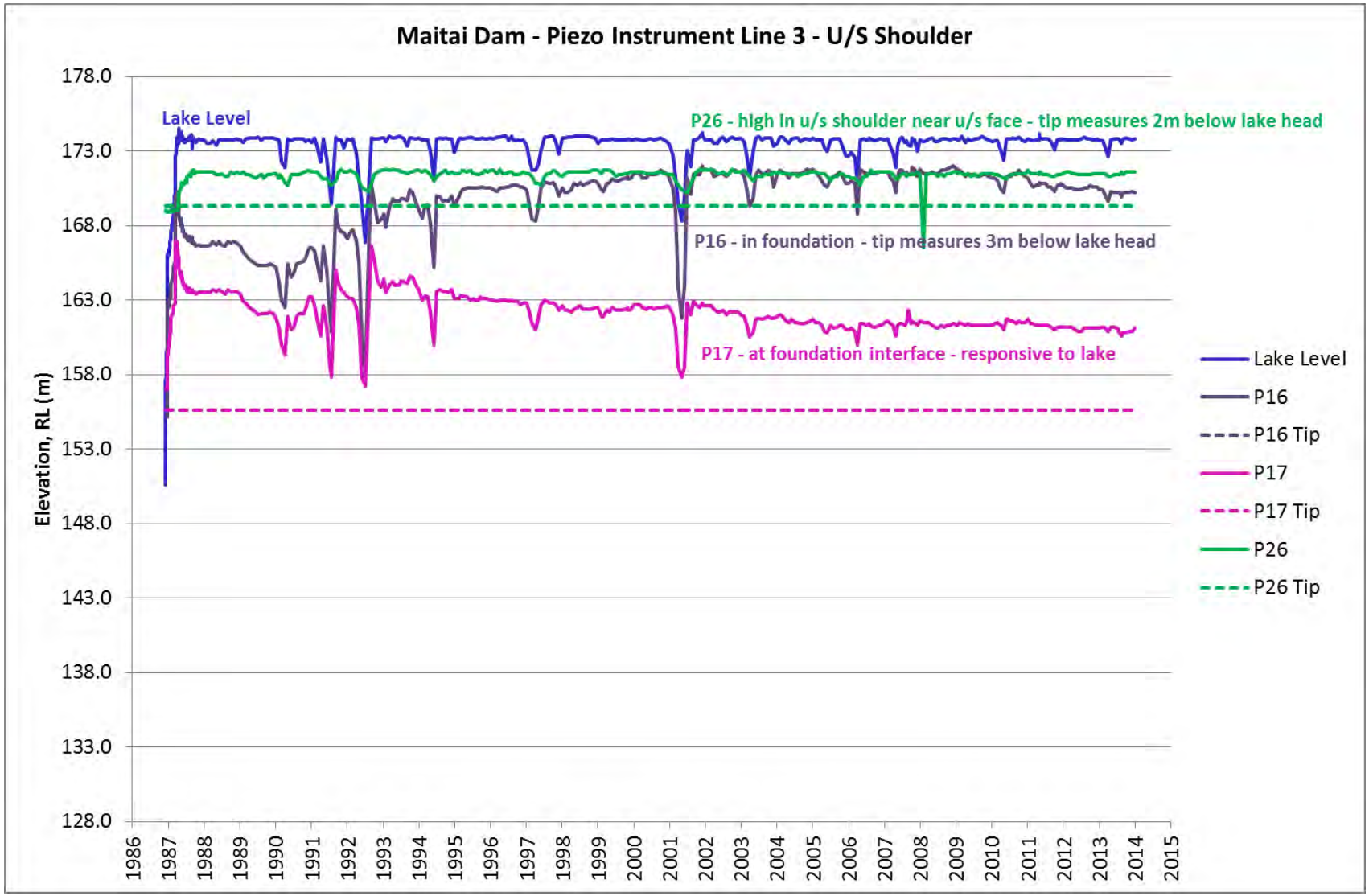

Figure 13: Maitai Dam – Piezometer Instrument Line 3 – Upstream Embankment and Foundation (1986-2014)

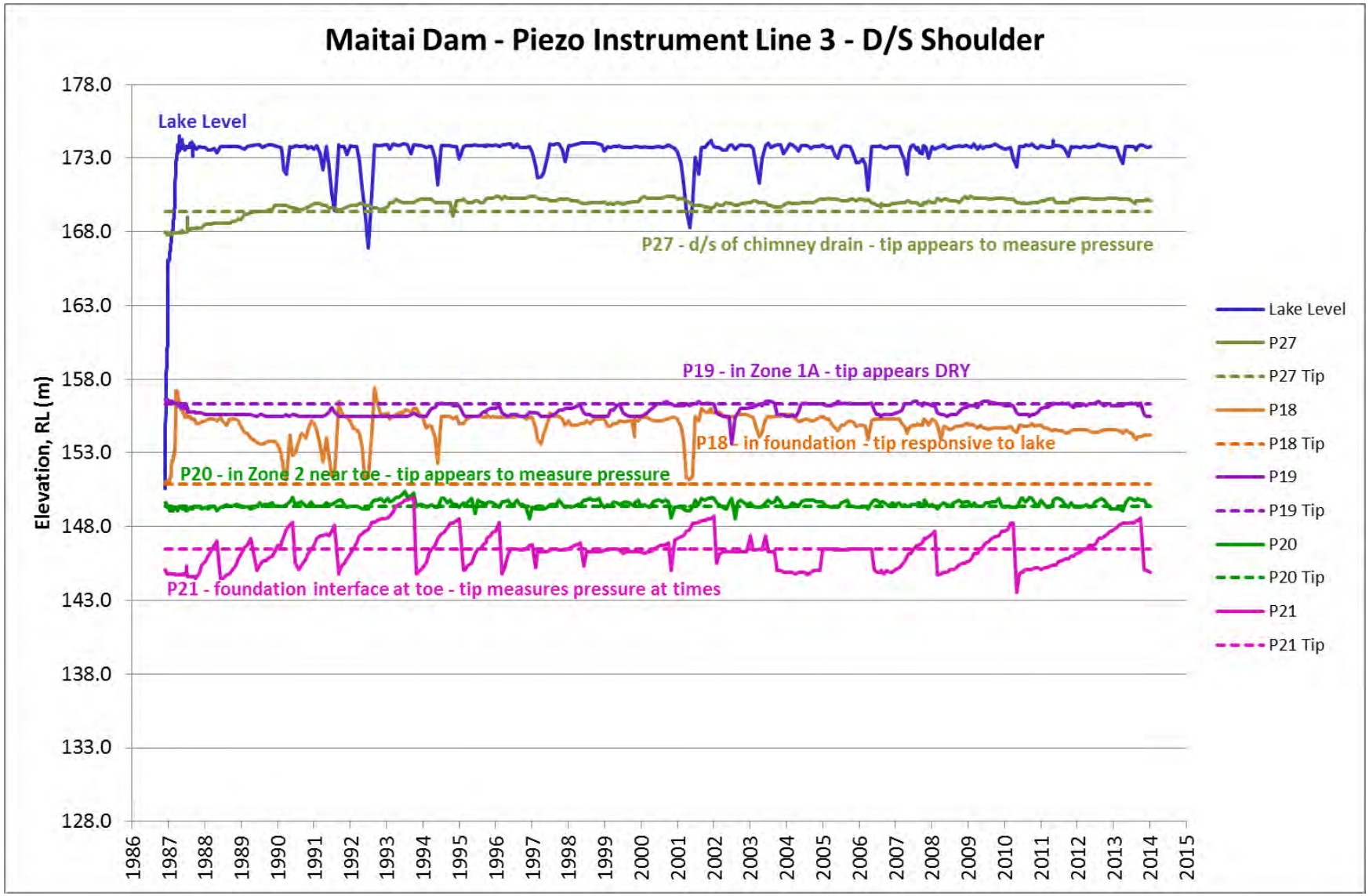

Figure 14: Maitai Dam – Piezometer Instrument Line 3 – Downstream Embankment and Foundation (1986-2014)

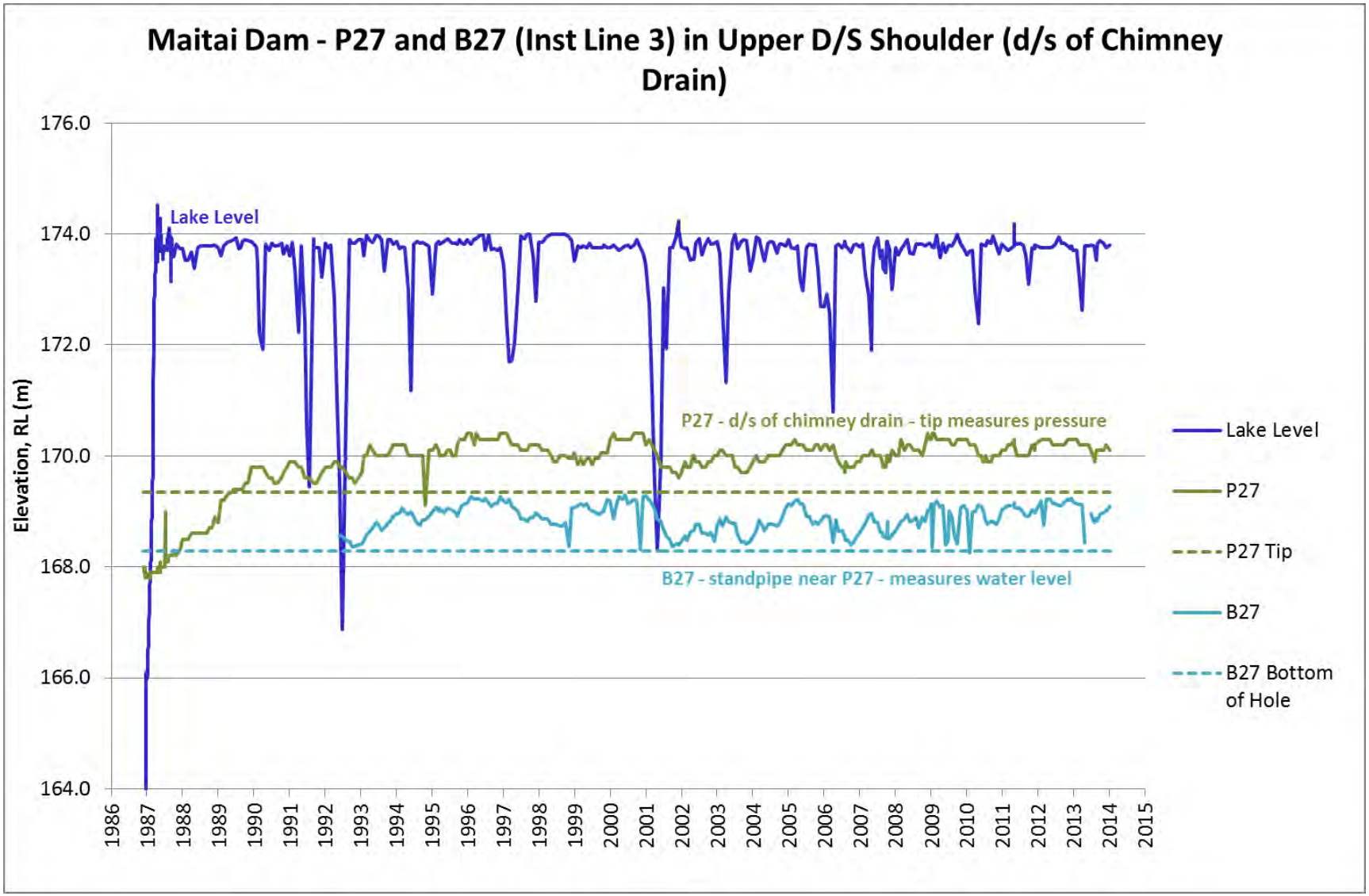

Figure 15: Maitai Dam – Piezometer Instrument Line 3 – P27 and B27 Upper Downstream Embankment (1986-2014)

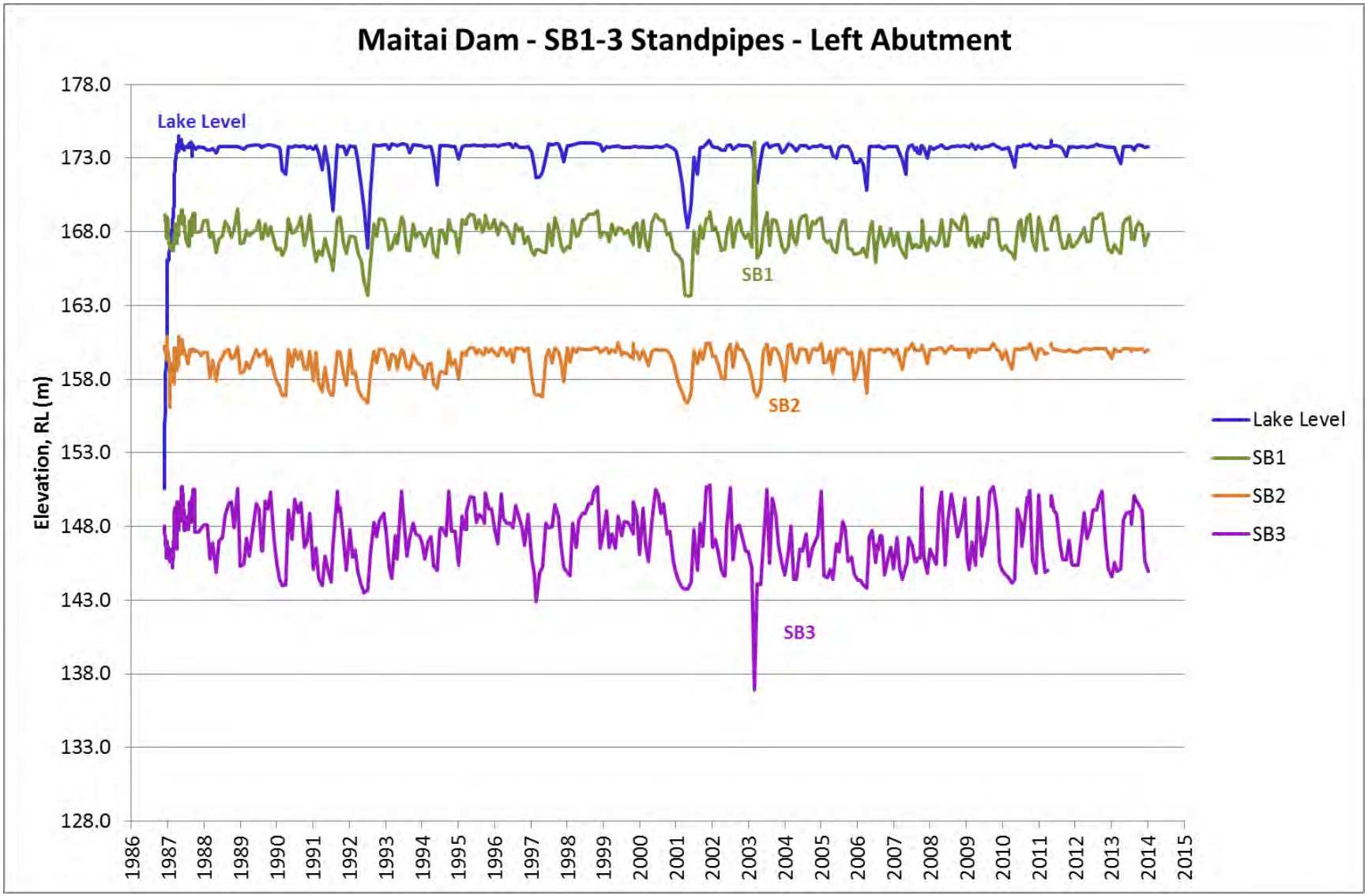

Figure 16: Maitai Dam – Standpipe Piezometers – SB1-3 Left Abutment Ridge (1986-2014)

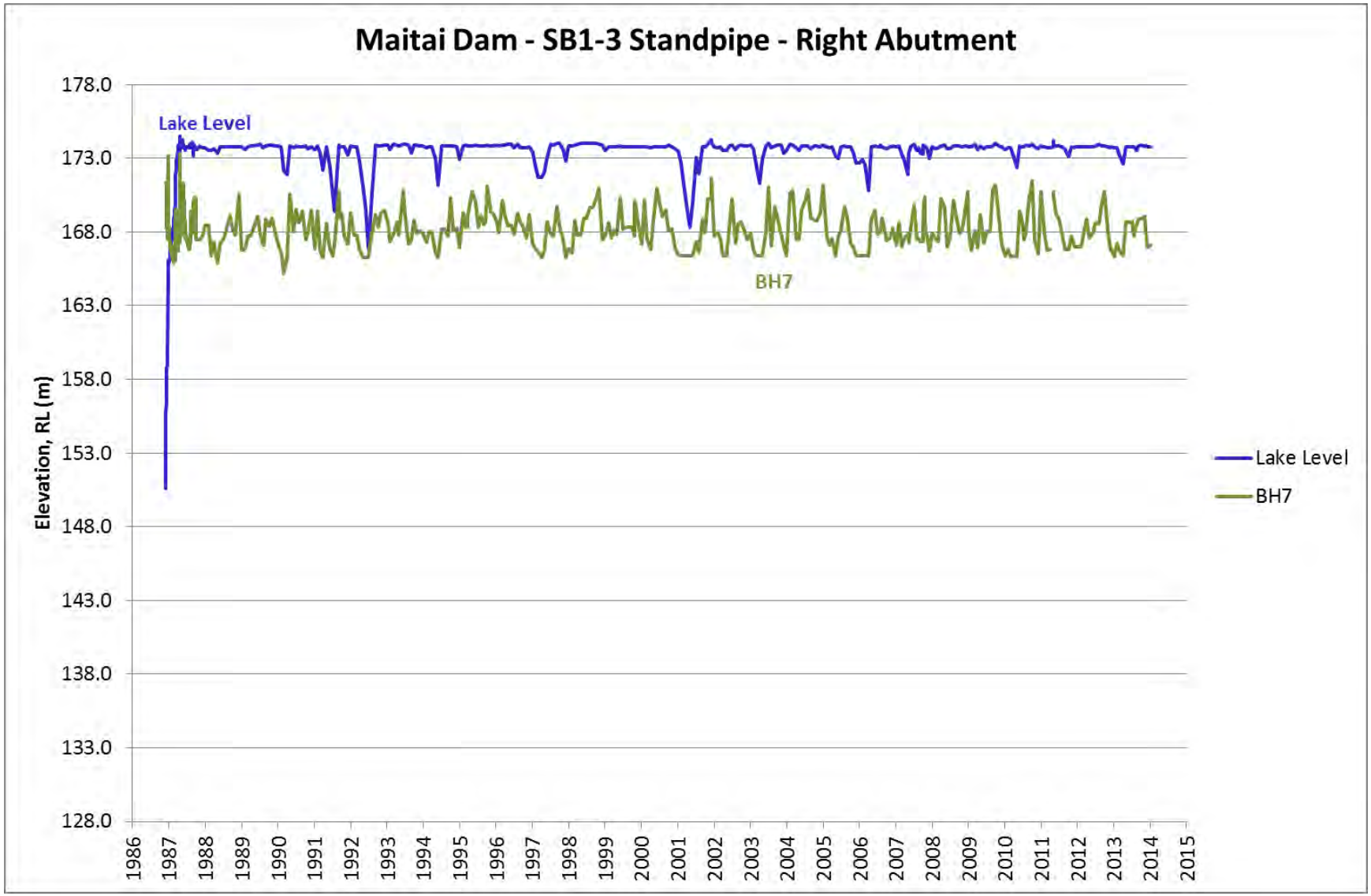

Figure 17: Maitai Dam – Standpipe Piezometer – BH7 Right Abutment D/S of Dam Centreline (1986-2014)

7.3.3 Dam Seepage Flow Monitoring

Dam seepage flows are a key performance indicator for Maitai Dam.

Dam seepage is measured at the respective drainage outlets at the dam toe (refer Seepage Measurement and Standpipe Locations Drawing 12446). Total outlet flow, left and right abutment drain flows and culvert interceptor flows are measured directly. Central collector drain flow (chimney drain and foundation blanket drain) is calculated by subtracting the left and right abutment drain flows from the total outlet flow. Time-series seepage flow data are presented in Figures 18 to 21.

The downstream culvert area includes left and right exit area drains. The right exit area drain conveys seepage from the R.H.S. of the downstream bays of the culvert and any seepage collected behind the south retaining wall. The left culvert exit area drain conveys any seepage from the left side of the culvert along the south retaining wall.

Observed Behaviour
Dam seepage flow expected and observed behaviours are presented in Table 12.

Table 12: Dam Seepage Flows

Name/Location	Expected Behaviour	Observed Behaviour
Central Collector (chimney and blanket drains)	Commissioning acceptable limit was 14 l/s. Design expectation was 0.8 l/s. Expect response to reservoir.	Flow around 1 l/s. Decreasing trend with time. Appears some response to reservoir and possible rainfall infiltration of embankment and runoff from abutments.
Culvert Interceptor Drain	No significant measurable flow expected.	Flow around 0.05 l/s. Decreasing trend with time. Appears some response to reservoir.
Left Abutment Contact Drain	Commissioning acceptable limit was 2 l/s. Expect heavily influenced by rainfall runoff from abutment.	Little apparent base drainage flow. Peak flows appear to be due to rainfall runoff from the abutment.
Right Abutment Contact Drain	Commissioning acceptable limit was 2 l/s. Expect heavily influenced by rainfall runoff from abutment.	Little apparent base drainage flow. Peak flows appear to be due to rainfall runoff from the abutment.
Right Culvert Exit Area Drain	Commissioning acceptable limit was 5.0l/s Expect culvert flows to be influenced by reservoir level and drainage along wall to be influenced from rainfall runoff along the retaining wall	Decreasing trend with time and periodic spikes. Commissioning experienced highest flow of 3.64l/s. Over operational period flows are nil to 1.0 and during this CSR review period has a max 0.87l/s on 2/10/2010 and average flow of 0.073l/s.
Left Culvert Exit Area Drain	Commissioning acceptable limit was 0.5l/s Expect heavily influenced by rainfall runoff along the retaining wall	Commissioning and operation: Nil.

Interpretation

Observed dam seepage flows are satisfactory. Dam designers expected that seepage would be minimal and this is confirmed. There is a slow decreasing trend in central collector and culvert interceptor drainage, which may be indicative of slow siltation staunching of the reservoir. The left and right abutment contact drains are heavily influenced by rainfall runoff and significant base flows are not apparent.

No seepage was observed from the R.H.S. culvert exit area. Over the review period, flows were typically at 0.73l/s with one high flow of 0.870l/s on 2 October 2010. Over 2013 minimal flows were recorded. During the CSR site inspection, leakage was however observed emerging from the right side at the corner of the first step and the south retaining wall (refer Appendix C). This is near the pipe joint of the R.H.S culvert drain to the retaining wall 110mm diameter drainage pipe, as shown on Drawing 6516-15 AB. The low leakage observed in 2013 (average of 0.031l/s) and observed leakage may indicate blockage or damage to the drain outlet, possibly during construction of the secondary water supply line. (**Rec-15) It is recommended the right exit area drain be checked for damage or blockage in regards to the new seepage observed (ref: Rec-08).**

7.3.4 Spillway Drain Monitoring

Spillway underdrains are provided beneath the chute slabs and behind the chute walls, with outlets in Manholes 1-3, and downstream of, and adjacent to the flip bucket.

Drain outlet flows are recorded using a visual rating system and measured by timed flow as necessary. Results indicate that drains are largely dry with some trickling or producing low flow. No adverse conditions are identified.

However, it is not known whether the spillway drainage system has been recently assessed for condition and performance, or flushed to maintain function. The blocked flip bucket outlet drains were recommended to be cleaned out in the 2013 Intermediate Inspection (Tonkin & Talyor, 2013). (**Rec-16) It is recommended that the spillway drainage system be assessed for condition and performance, and flushed to maintain functionality, as far as is practicable.**

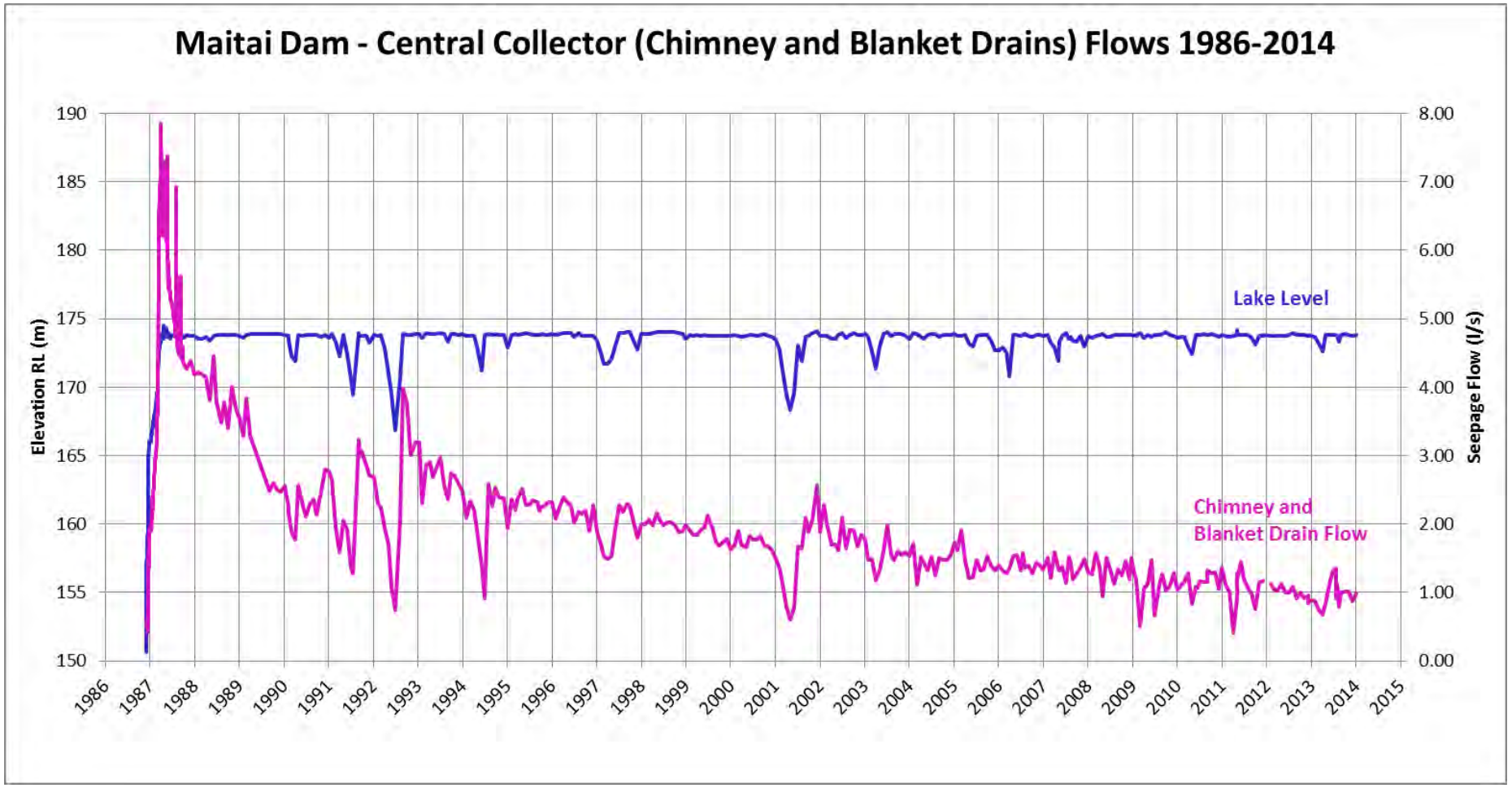

Figure 18: Maitai Dam – Central Collector Seepage Flows (Chimney and Blanket Drains) (1986-2014)

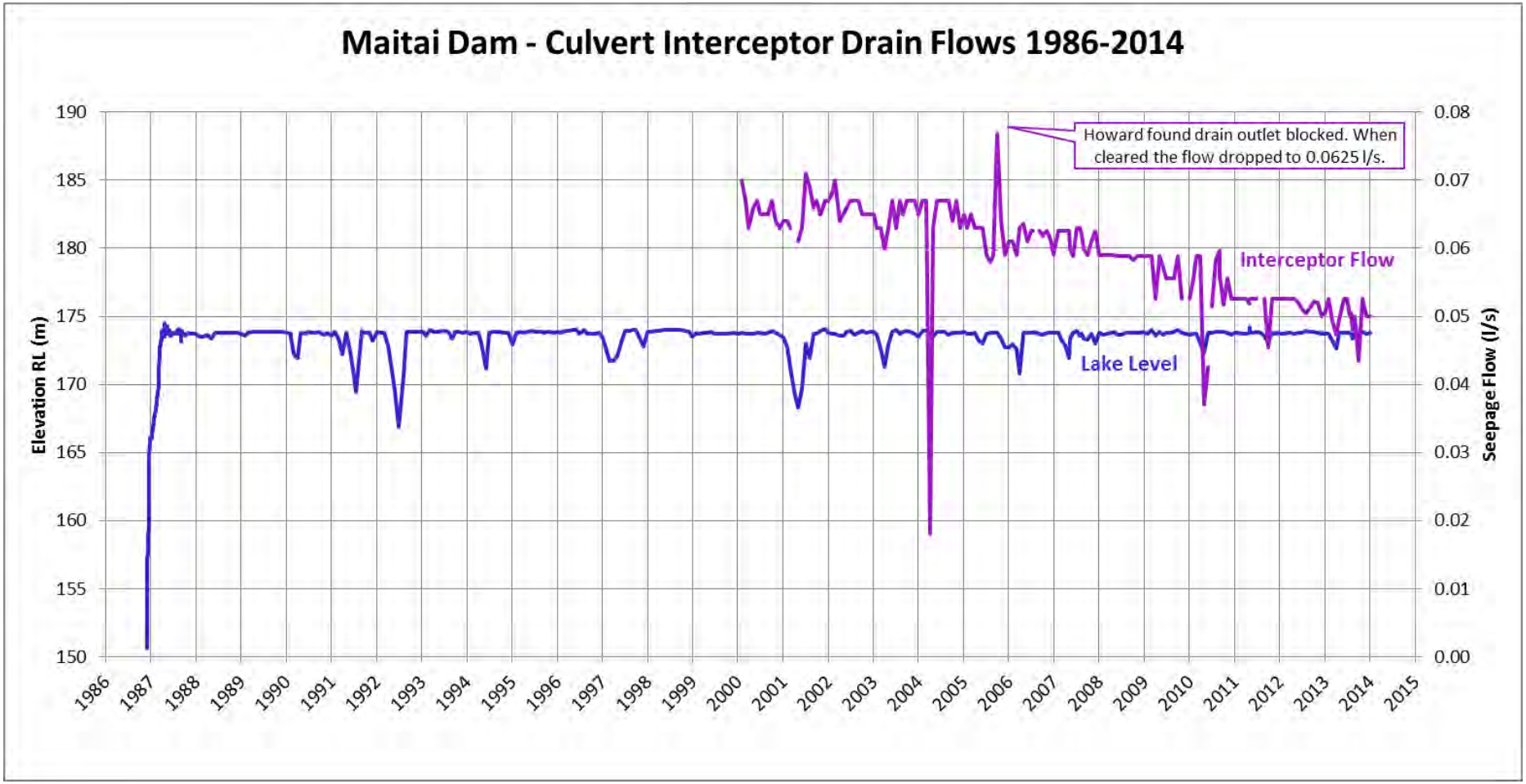

Figure 19: Maitai Dam – Culvert Interceptor Drain Flow (1986-2014)

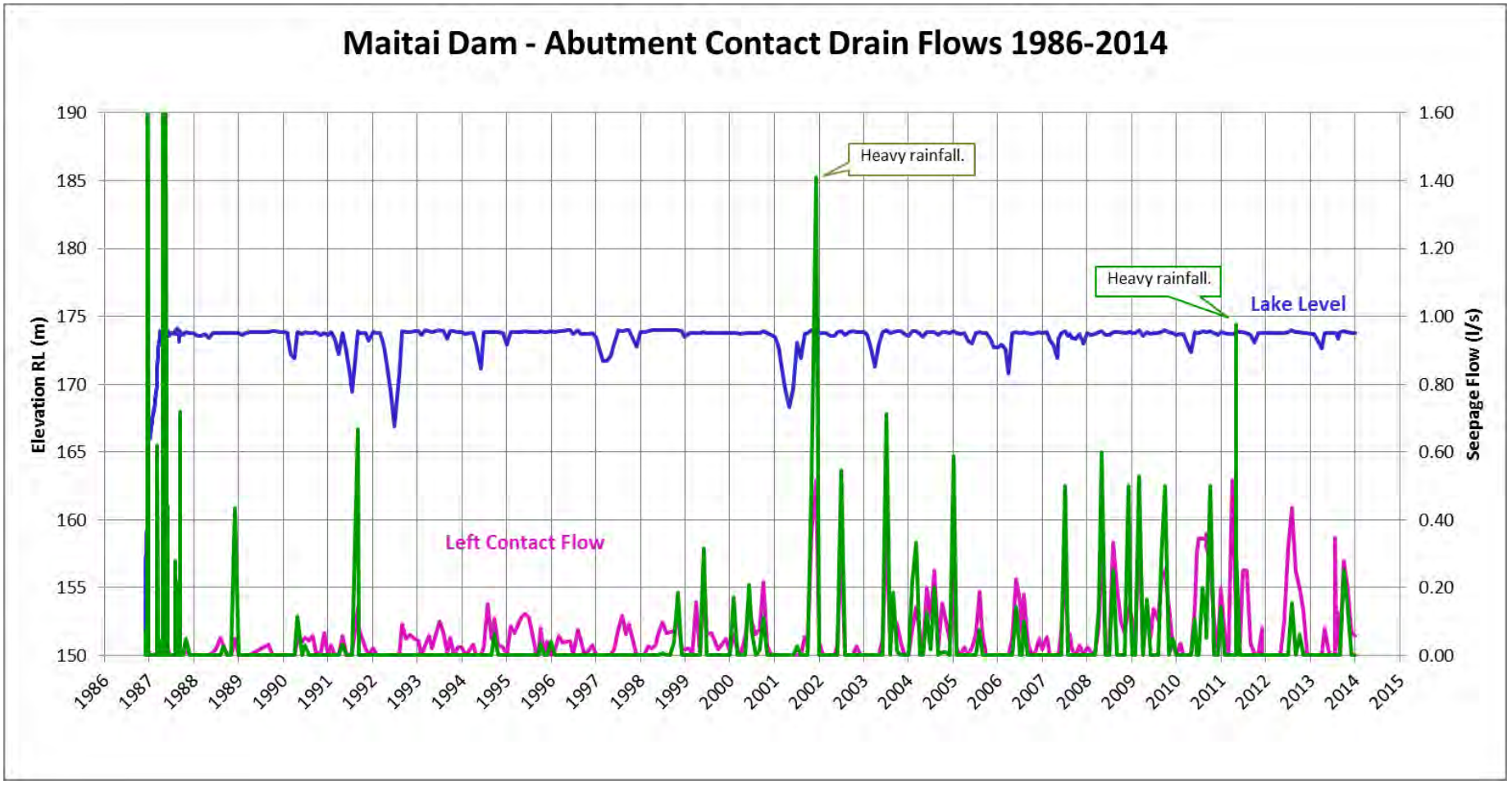

Figure 20: Maitai Dam – Abutment Drain Flows (Left and Right) (1986-2014)

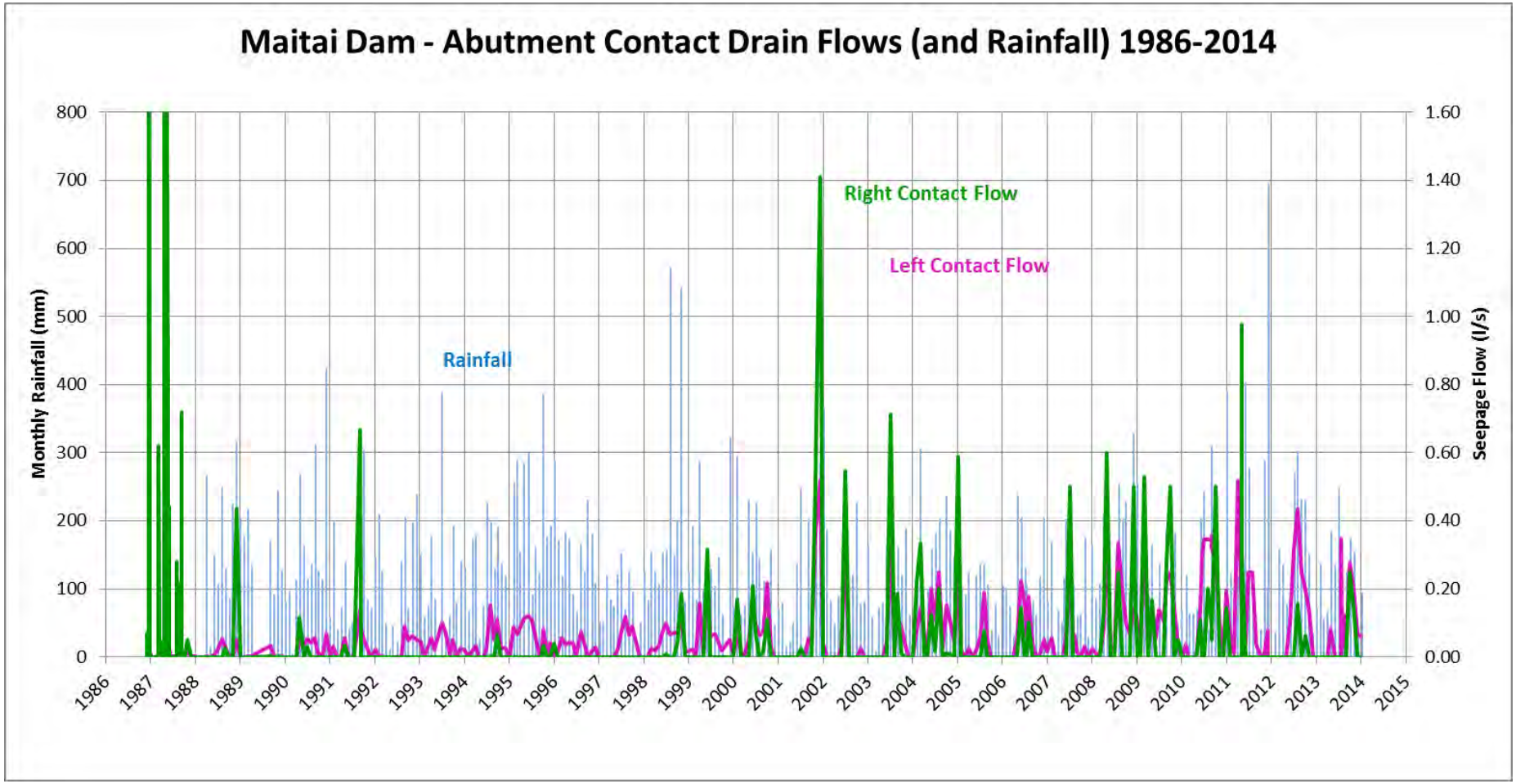

Figure 21: Maitai Dam – Contact Drain Flows (Left and Right) – with Rainfall (1986-2014)

7.4 Deformation Survey

Deformation survey is a key performance indicator for the Maitai Dam, particularly after a significant earthquake, where earthquake-induced settlement or structural damage may occur.

Survey Network
The dam survey network includes control pillars BM6-10 and dam targets as follows (refer John West Survey Drawing JWS304):

- C1-7 crest marks surveyed for vertical position
- S3-4, S6-7 and S9-10 downstream face marks surveyed for horizontal and vertical position
- SY1L/U to SY6L/U joint marks in the centre of the spillway chute surveyed for vertical position and slope distance between pins measured by stainless steel ruler (horizontal distance calculated)
- W1L/U to W6L/U joint marks on the spillway right wall surveyed for horizontal and vertical position
- Nails 1-11 in a timber beam across the auxiliary spillway crest surveyed for vertical position

Surveys Completed
Maitai Dam deformation surveys have been completed in 1986/1987 (monthly surveys during commissioning), 1988, 1989, 1990, 1991, 1993, 1995, 1998, 1999, 2000, 2001, 2002, 2003, 2006, 2008, 2011 and 2013. Horizontal measurements are made using a Topcon Total Station 225 and coordinates adjusted using Snap least squares adjustment. Levelling is carried out using a Leica DNA03 Digital Precise Level and Bar Coded Invar Precise Stave.

Survey Data Management
Maitai Dam deformation surveys records do not appear to contain the full history of surveys in one repository. Current data comparisons of mark movement by John West Surveys are relative to the 1995 survey rather than the commissioning surveys in 1986/87. Time-series and spatial presentations of survey data do not appear to exist. This is inadequate for a High potential impact dam that attracts the highest level of surveillance in the NZSOLD Dam Safety Guidelines (NZSOLD, 2000). **(Rec-17a) It is recommended that Maitai Dam survey data be consolidated into a full and continuous historical record, managed in one repository, and that an appropriate suite of time-series and spatial plots be developed to allow evaluation.**

Deformation Survey Evaluation
In order for evaluation of dam deformations to be completed a considerable suite of time-series and spatial plots are required. The 2008 Comprehensive Safety Review (Riley, 2008) provided examples of such plots and drew conclusions on the dam's deformation performance since construction. The dam has settled less than 50mm in its 28 years since construction. This is satisfactory performance for a 39m embankment dam (0.12%) and consistent with expected values for wide core embankments (Hunter, 2003).

Based on the 2008 data presentation it appears that the left abutment crest has settled more than the centre crest and right abutment crest. Evaluations of dam deformation should be made in the context of the dam's foundation geometry and features. Differential settlement cracking can be an issue where rapid changes in geometry occur. **(Rec-17b) It is recommended that the dam survey mark locations and movement vectors be plotted**

onto the as-built valley cross sections so that deformations can be evaluated in the context of foundation geometry.

7.5 Adequacy of Surveillance

Maitai Dam is High potential impact and therefore attracts the highest level of surveillance for dams by the NZSOLD Dam Safety Guidelines (NZSOLD, 2000). For an earthfill dam, the key performance indicators that characterise the behaviour of the dam and monitor for onset of potential failure modes are visual observations, piezometric conditions, seepage flows and measurement of deformations. Current routine dam inspections include visual observations and seepage and piezometric data collection as monitoring for confirmation of ongoing satisfactory dam performance. The check-sheets used in the field and the competence and knowledge of NCC's surveillance inspectors are suitable for this.

Monthly routine inspections and annual deformation surveys meet the frequency requirements of the Guidelines.

Potential Failure Modes Monitoring

Maitai Dam does not have formally developed potential failure modes (PFM's). Understanding a dam's potential failure modes, as well as its characteristic behaviour, allows operation, surveillance and maintenance activities to be targeted directly at the areas of importance and thus provide best prevention, or at worst, early detection of the initiation of a dam failure mechanism. The same applies to safety reviews of dams in ensuring that issues relevant to the safety of the dam are not overlooked. **(Rec-5 repeat): Considering Maitai Dam's high potential impacts of failure it is recommended that potential failure modes be formally developed. (Rec-18) It is also recommended that inspection, monitoring and evaluation requirements be reviewed and updated with consideration of the dam's potential failure modes.**

Piezometer Monitoring

Maitai Dam is well monitored for piezometric pressure which is one of its key performance indicators. As stated earlier there are some anomalous observations in the upper embankment downstream of the chimney drain that need to be resolved. It is essential that the data collected and presented is reliable and interpreted in the context of the dam's performance expectations. There are many aspects that contribute to this including accurate documentation of the instrument installation details, ongoing performance characteristics, and maintenance and calibration activities. Appropriate specialist inputs should be sought to provide NCC with assurance that the monitoring data is reliable and dam safety status can therefore be correctly judged.

Deformation Surveys

The absence of continuous deformation survey data records and presentation methods is a serious shortcoming of the current deformation survey programme. Deformation surveys are a key performance indicator for Maitai Dam, particularly after a significant earthquake event, and warrant a level of quality and evaluation commensurate with the dam potential impacts. This is subject to preceding recommendations.

Surveillance Data Evaluation and Management

The record of the routine field surveillance inspections are captured in hard copy and filed for monthly check by NCC staff, and evaluation by dam safety engineers during annual and 5-yearly dam safety reviews. For a High potential impact dam it would be appropriate for surveillance data to be evaluated by a dam safety engineer at the time it is collected**. (Rec-19) It is recommended that monthly routine surveillance data is evaluated at the same monthly frequency by a dam safety engineer.**

NCC's surveillance data is largely stored on a monitoring spreadsheet, with the exception of deformation survey data, which appears to be managed by the surveyor. During this review a number of data errors and miscalculations were found in the spreadsheet, which highlights the pitfalls of spreadsheets as a dam surveillance data management tool. **(Rec-20) It is recommended that NCC dam surveillance data management arrangements be reviewed by an appropriate advisor and improvements made to ensure quality assurance and security of data. Data presentation methods should also be reviewed and improvements implemented to ensure that surveillance evaluation is continuous and effective.**

8.0 ENGINEERING ASSESSMENT

General design of embankment and appurtenant structures are assessed against modern practice for loading conditions.

8.1 Assessment Criteria

General

The over-riding principles applied by the NZSOLD Dam Safety Guidelines (NZSOLD, 2000) are the performance criteria that the dam should;

a) be able to safely pass the Inflow Design Flood, and

b) be able to withstand the Maximum Design Earthquake (or SEE for existing dams) without uncontrolled release of the reservoir.

Flood Passage
Flood passage criteria are given in the Guidelines based on a dam's PIC. As discussed earlier, Maitai Dam has a High potential impact classification (PIC). The following is therefore relevant;

"For High potential impact dams the minimum inflow design flood (IDF) is usually between 1 in 10,000 AEP and the PMF. PMF is usually selected if a large number of fatalities would result from failure of the dam."

By definition under the new Dam Safety Scheme, a High PIC is assigned to a dam "if it is highly likely that two or more lives will be lost" as a result of its failure. Maitai Dam is clearly a High PIC dam based on the population at risk from a potential dam break. Following the Dam Safety Guidelines would therefore indicate that the Maitai Dam is required to safely pass the PMF without risk of dam failure.

The spillway structure must also meet requirements for stability under the design loading conditions.

Embankment Stability
Embankment slope stability is evaluated under normal (static) and earthquake loading. The stability of the dam also depends on measures to prevent internal erosion and control seepage under all loading conditions.

For normal static loading, embankment performance is usually assessed in terms of minimum factors of safety for the following cases:

- Downstream slope a have a minimum factor of safety of 1.5 under steady state seepage with maximum storage pool.
- Upstream slopes have a minimum factor of safety of 1.2 to 1.3 under full or partial rapid drawdown.

For earthquake loading, embankment performance is assessed using the following criteria recommended by NZSOLD Dam Safety Guidelines (NZSOLD, 2000):

- Only minor damage to occur under the OBE loading.

- Impounding capacity of the dam to be maintained under the SEE loading.

8.2 Flood Passage Assessment

The design of Maitai Dam for safe passage of the flood included the use of a service spillway and an auxiliary spillway. Assessment of the service and auxiliary spillway capacity to safely pass the PMF flood and structural stability are discussed in the following sections based on review of available documents.

8.2.1 Spillway Capacity

The service spillway was designed to pass a maximum flow of 125 m^3/s (1:100 AEP flood event). The auxiliary spillway is to operate (by fusing) at higher floods flows. The combined capacity of the auxiliary and service spillway is to safely pass flood flows of 290 m^3/s (PMF event). The peak PMF outflows correspond to 280m^3/s and the maximum reservoir level R.L. 176.75m. Under the PMF, the estimated peak flows were 221m^3/s for the service spillway and 59m^3/s for the auxiliary spillway.

The flood flows used in design are more conservative than estimated in recent studies for Maitai Dam catchment as outlined in Table 4 in Section 4.0. The 1:100 AEP flood was estimated in the 2003 CSR (Tonkin & Taylor, 2003) to range between 80 to 112m^3/s, which is consistent with 90m^3/s estimated from Maitai catchment river flow data (TDC). The 1994 Upgrading Design (Tonkin & Talyor, 1994) updated the PMF using methods developed by the New Zealand Meteorological Service. The updated PMF resulted in inflows of 260m^3/s and peak outflows of 246m^3/s with a maximum reservoir level of R.L. 176.53m. The estimated outflows were 195m^3/s for the service spillway and 51m^3/s for the auxiliary spillway. The 1998 CSR provided a slightly higher indicative value of 277m^3/s for the PMF, which is consistent with design and updated values.

The service and auxiliary spillway combined capacity is sufficient to safely pass design flood criteria for a High PIC structure in terms of the NZSOLD Guidelines. The PMF flood flows used in design are conservative and derived using appropriate methodologies.

The capacity of the auxiliary spillway will not produce a surge flood wave as a result of the fuse plug operating. The auxiliary spillway provides flood passage capacity of 59m^3/s or21% of the peak PMF outflows. This contribution is relatively small compared to total peak outflows of 280m^3/s and thus, does not change the downstream flood hazard levels.

The 2003 CSR (Tonkin & Taylor, 2003) concluded that;

- the design flood inflows for the 1:100AEP and PMF were considered to be conservative.
- having assessed the service spillway performance using updated PMF inflows and outflows
    - the chute walls appear to have sufficient freeboard.
    - the estimated trajectory of spill flows was comparable to the original design estimate of 50m throw and the 80m long plunge pool would readily accommodate spillway flow trajectory.
    - there is potential for the spill flows to skip across the surface of the plunge pool at high velocity due to marginal entry angle but competent rock at the plunge pool river

bank is expected to be able to resist erosion from any jet wash; colluvium present would erode.

8.2.2 Structural Assessment

Structural performance of the service and auxiliary spillway was assessed as part of this CSR. Stability of the structures under design load conditions of normal, flood and earthquake were considered.

Service Spillway

The service spillway block dimensions suggest a conservative design, and it's stability under the design earthquake accelerations is acceptable. The stability analysis was performed for the concrete-rock interface using a conservative interface angle of friction of 42º with zero cohesion and presuming the foundation rock is stable. Sliding mechanisms along rock joints were not considered.

The spillway chute side walls are stable with acceptable performance under the DBE with a PGA of 0.35g. The structural performance under MCE (0.75g) will require detailed analysis. Preliminary calculations suggest yielding of the wall base under MCE. This is consistent with the design report (Tonkin & Taylor, 1986) that some damage to the spillway may be acceptable under the MCE loading.

It is pertinent to mention that transverse reinforcement for spillway wall base as shown in as-built Drawing 6516-215 AB appears to be inconsistent with general industry practice. The wall sections indicate less reinforcement in the transverse direction (across flow) compared to the longitudinal direction (along flow). The forces acting on the wall require main reinforcement in the transverse direction and only nominal reinforcement in the longitudinal direction. This could be a drafting mistake, but design (Tonkin & Taylor, 1986) and construction (Tonkin & Talyor, 1987) reports do not provide clarity on it. The calculations were based on the reinforcement shown in the drawing.

The chute floor has not been specifically analysed, but appears to be conservatively designed. Photographs of the chute floor show turbulent flow over surface cavities and level differences at some floor joints (Photos 27 and 28 in Appendix C). The surface cavities should be repaired to ensure a smooth finish on the chute floor **(repeat Rec-09)**.

The flip-bucket at the end of the chute is anchored to the foundation rock and has been conservatively designed for expected uplift forces.

Auxiliary Spillway

The auxiliary spillway is stable against potential undermining below sill level during fuse operation. The fuse plug triggering troughs are mounted on a lightly reinforced concrete sill. The concrete sill was cast in-place on excavated foundation bedrock. The concrete and foundation bedrock provide appropriate protection and resistance against erosional forces under fuse operation to prevent the potential loss of the sill and embankment leading to uncontrolled release of reservoir contains.

The proper operation of the auxiliary spillway fuse plug depends on the gravel facing to protect and maintain the upstream and crest clay core intact. The auxiliary spillway concrete sill is at

R.L. 175.18m, which is overlain by clay core and then drainage gravels. The gravels provide protection against wave erosion and the environment effects (i.e., drying, rainfall). The clay core prevents premature fusing during high lake levels above the concrete sill level and below the auxiliary spillway crest at R.L. 175.61m triggering level. As discussed in Section 4.1, past flood events were estimated to have flows up to 1.29m above the spillway crest level, which corresponds to a minimum freeboard of 0.14m to the auxiliary spillway sill. These events highlight the potential for premature fusing to occur as a result of undermining of upstream gravel and clay core under wave action. Maintaining the auxiliary spillway slope materials is important to the auxiliary spillway operation. **(Rec-21) It is recommended inspection of the upstream slope area of the auxiliary spillway be performed following unusual high reservoir levels.**

Blockage of the service and auxiliary spillway during flood events is prevented by the log boom in place. The catchment includes bush cover so some debris should be expected. The service spillway weir width is 20m and the auxiliary spillway width is 20m.

8.3 Dam Stability Assessment

Dam assessments include evaluation of embankment slope stability and material compatibility and stability against internal erosion. NZSOLD Dam Safety Guidelines (NZSOLD, 2000) recommend these assessments be performed under normal (static) and earthquake loading conditions. The design of Maitai Dam embankment slopes for stability considered the following principal factors (Tonkin & Taylor, 1986):

- crest width at 6 m wide and elevation for wave freeboard above the maximum flood level,
- possible future storage level at R.L. 175m with corresponding steady seepage gradients,
- seepage control measures,
- rapid drawdown from R.L. 175 to 159m,
- design basis and maximum credible earthquakes with reservoir full, and
- dynamic characteristics of fill materials determined by test.

The following provides an assessment of embankment slope stability and internal erosion based on available information and previous studies.

8.3.1 Embankment Slope Stability

The slope stability analysis of Maitai Dam embankment slopes were assessed as part of the dam design (Tonkin & Taylor, 1986). Bishop simplified methods of limit equilibrium were used to analyse slope stability. Preliminary checks have been made in previous safety reviews. However, there has been no formal stability analysis of the embankment slopes to account for behaviour represented in the surveillance data over its operation life. For example, design stability analysis considered the chimney drain fully effective and no pore pressures within the downstream shoulder.

<u>Normal Loading Conditions</u>
Normal loading conditions are related to normal operation of the dam and reservoir contents. There are two primary dam slope stability assessments performed as follows:

- Embankment downstream slope with steady state seepage and
- Embankment upstream slope with rapid draw down of reservoir contents

Embankment shear strengths with an effective friction angle of 37° and apparent cohesion of 15 kPa was used based on laboratory testing. Table 13 provide a summary of design stability results compared to recommended minimum factors of safety (NZSOLD, 2000).

Table 13: Summary of Maitai Dam Slope Stability under Normal Loading Conditions

Condition	Factor of Safety	
	Design	NZSOLD Recommended
Downstream slope - Steady State Seepage	>2.0	1.5
Upstream slope – Rapid Draw Down	>1.3[a]	1.2 to 1.3[b]

[a]Drawdown between R.L. 173.75m and R.L.159m with steeper slope at 2.5H: 1V than constructed
[b] Higher factors of safety may be required if drawdown occurs relatively frequently during normal operation

Stability of the downstream slope is highly dependent on pore pressures conditions. Pore pressure conditions in the downstream are monitored by piezometers installed within the embankment during construction. Section 8 .3 provides a discussion on piezometric readings and interpretation. Observations purport some pore pressures within the upper levels of the downstream shoulder. Refer to recommendations **Rec-12a and 12b** to resolve this issue. Given there is uncertainty in the embankment pore pressures, assessment of the downstream slope stability under steady state seepage cannot reliably be made. **(Rec-22) It is recommended that a full slope stability analysis be performed for Maitai Dam following verification of piezometric conditions within the embankment**.

Stability of the upstream slope under rapid draw down is considered reasonable given the analysis provided:

- No drainage within the upstream shoulder,
- Reduced apparent cohesion to 7.5kPa,
- Evaluated steeper slope at 2.5H:1V than as constructed at 2.6H:1V, and
- Reservoir draw down levels to approximately half the embankment height.

<u>Earthquake Loading Conditions</u>
Stability of the embankment was evaluated using pseudo-static methods for the design basis earthquake (DBE) with a PGA of 0.35g and maximum credible earthquake (MCE) with a PGA of 0.75g. Figure 22 shows the critical slip surfaces evaluated for Maitai Dam. It is unclear if the pore pressure conditions were applied in these analyses and most likely used unsaturated condition within the downstream shoulder (refer to **Rec-11a, 11b and 20**). Table 14 provides a summary of results presented as yield acceleration, which corresponds to a factor of safety equal to one. The table also includes a preliminary check performed during the 1993 SEED review on downstream slope using Utexas software.

Embankment dynamic shear strengths of an apparent cohesion of 175 to 190 kPa and friction angle of zero were used. These values are consistent with 80% reduction of the average static

shear strength (Idriss, 2008) depending on pore pressures conditions. Limited cyclic anisotropically consolidated triaxial tests were performed but it is unclear how they were interpreted and accuracy of results due to unequalised pore pressures.

Earthquake performance of the Maitai Dam should be evaluated against criteria recommended by Dam Safety Guidelines (NZSOLD, 2000) for a High PIC dam. Site specific ground motions need to be developed for Maitai Dam **(Rec-4)** to evaluate earthquake performance at the OBE at 1:150 AEP and at the SEE of either the 1:10,000 AEP or MCE. The new Dam Safety regulations will also require dams such as Maitai Dam located in earthquake zones to meet criteria at the 1:500 AEP event.

Given there are no site specific ground motions developed using current methodology for Maitai Dam, Table 5 (Section 4.2) provide some guidance relative to the current NZ Building Code Standard. It is noted site specific ground motions may be higher or lower than those estimated using current NZ Building Code Standard.

The embankment slopes are considered to be stable with 'No to only minor repairable damage' under the OBE loading at 1:150 AEP with a PGA of 0.16g. This is based on a higher minimum yield acceleration of 0.35g. The yield acceleration estimated during design is based on results of a pseudo-static stability analysis with no pore pressures applied within the downstream shoulder. Pore pressure conditions were discussed in more detail in Section 7. A lower yield acceleration is expected if pore pressures exist within the downstream slope are present.

The minimum yield acceleration of 0.35g indicates seismic-induced deformations are expected under the SEE strong ground shaking. There is uncertainty in the level of the SEE ground motions and stability results with no pore pressures within the downstream slope. The critical slip surface is the downstream slope. Results of the upstream slope also indicate seismic-induced deformations are expected under the SEE loading.

Table 14: Summary of Potential Failure Surface Under Sesimic Loading

Slope	Slip Surface	Design Yield Acceleration	1993 SEED	Average Dynamic Shear Strength
Upstream	1	0.37g		190kPa
	2	0.38g		185kPa
	3	0.67g		175kPa
Downstream	4	0.35g	0.46g	180kPa

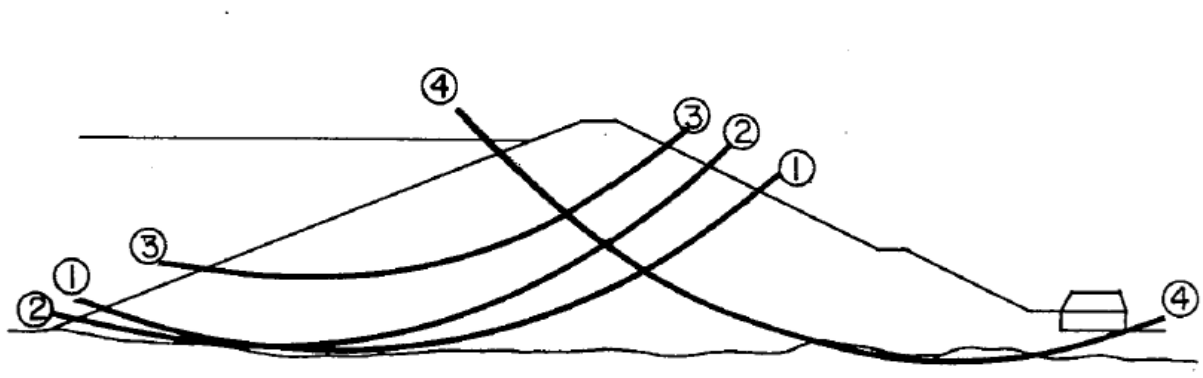

Figure 22: Potential Failure Surfaces Analysed for Seismic Loading of Maitai Dam (Tonkin and Talyor 1986)

Seismic-Induced Deformations

Seismic-induced deformations within embankments exhibit as crest settlement and cracking. The amount of deformations depends on the level of ground shaking and embankment stability. Limit equilibrium methods in pseudo-static slope stability analyses provide yield accelerations, which can be used to estimate the relative magnitude of crest settlement from Newmark-type models. Empirical charts by Pells and Fell (2002, 2003) can be used to estimate both crest settlement and cracking based on case histories of damage to dams under strong ground motions. More detailed dynamic analyses are required to better understand embankment deformations and areas susceptible to stress concentrations and cracking.

Newmark-type methods provide an estimate of crest settlement magnitude and not an absolute value. The design of Maitai Dam employed Newmark-type methods recommended by Makdisi and Seed (1978 and 1979) to estimate embankment crest settlements of 0.5m under the MCE loading. The 1993 CSR estimated deformation of 750mm using Newmark methods by Jansen (1990) and Bureau (1997).

Various Newmark-type models are available with the most current methodology by Bray and Travasarou (Bray, 2007), which is best suited for analysis of earthfill embankment dams. Bray and Travasarou method is an update to Makdisi and Seed approach. Both of these methods utilise response spectra and yield acceleration to estimate crest settlements. A benefit of Bray and Travasarou's method is the model was developed using a comprehensive data base of 688 pairs of ground motions records to estimate deformations.

Empirical charts by Pells and Fell (2002, 2003) provide guidance on seismic-induced deformations of crest settlement and cracking. The earthquake magnitude and PGA are used in a chart to define a damage class. Based on the damage class, crest settlements and longitudinal cracking are estimated. The 2008 CSR (Riley Consultants, 2009) used Pells and Fell chart to estimate crest settlement of 600mm to 2m based on settlement normalized by embankment height at 1.5 to 5.0%.They used design parameters for the MCE with an

earthquake magnitude of 7.5 and PGA of 0.75g. It is important to note this level of ground shaking corresponds to a damage class 4 described as 'Severe'. Additionally, longitudinal cracking with a maximum width of 150-500mm are also expected to occur under this level of strong shaking.

Seismic-induced deformations include longitudinal and transverse cracking (Pells, 2002). Longitudinal cracking is more common than transverse cracking. Visible longitudinal cracks occur in earthfill dams that experience an earthquake magnitude (M) of 6.5 or greater and PGA greater than 0.15g. Dams which experience a damage class 2 or greater are highly likely to experience transverse cracking. Transverse cracking and longitudinal cracking occur together for M of 6. 3 with PGA greater than 0.3g and M of 7.5 with a PGA greater 0.12g. Transverse cracking is more severe to embankment performance because they provide a direct path for leakage of reservoir water to erode embankment materials which may lead to piping failure. Based on design ground motions and a damage class of 4, transverse cracking is expected to occur.

Transverse cracking is more likely to occur as a result of cross valley differential settlement (Pells, 2002) and from material stiffness differences at the abutments contacts. Mechanisms for differential settlement from both static and seismic loading are related to changes in geometry or strength and compressibility of embankment materials or foundation. For Maitai Dam features potentially resulting in differential settlement are illustrated in Figure 23 and include:

- Sudden change in depths of embankment material along the foundation due to benches or irregularities within the left and right abutment profiles.
- Existence of highly stressed zones within the embankment from the culvert penetration and geometry.

Maitai dam has bench profiles in the foundation along the left and right abutment. Drawings 6516-230 AB, 231 AB and 232 AB provide the profile of the foundation and abutment. The width of the bench profiles varies from upstream to downstream. For the left abutment, the bench or irregularities in foundation profile are most prominent over sections from 20 to 80m and from 120 to 140m. For the right abutment, bench or irregularities in foundation are present from 20 to 60m and from 100 to 140m. Features such as these may result in differential settlement and transverse cracking under normal or earthquake loading. For example, Matahina Dam experience of internal erosion due to a bench feature in the abutment foundation as evident in investigations performed following magnitude 6.3 earthquake in 1987. As discussed in Section 7.4, the left abutment crest region has exhibited more settlement compared to the centre and right abutment.

The culvert penetration and its geometry within the embankment results in areas of high and low stress concentrations for differential settlement and cracking to likely occur. The culvert geometry from upstream to downstream is shown on the foundation profile drawings. The sharp corners of the culvert create areas of high stress concentrations while the corners at the culvert base result in areas of low stresses. The potential for differential settlement and transverse cracking also depends on compaction obtained along the culvert. Construction documents state fill material along the culvert was well compacted using hand equipment. The upstream 14 culvert bays, however, include plywood panels placed on the sides. The long-term implications of the plywood on the contact of fill against the culvert needs to considered when evaluating ongoing performance of the dam for cracking and internal erosion.

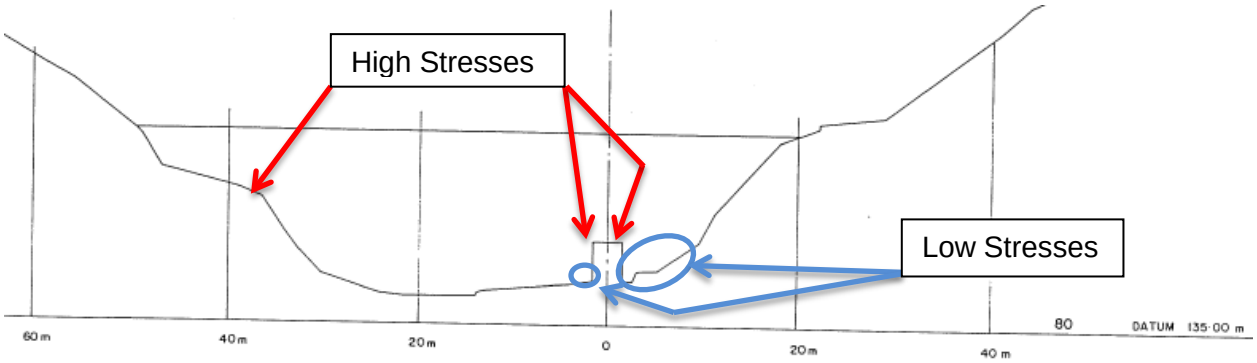

Figure 23: Areas of High and Low Stresses along Maitai Dam Foundation, Section 80

Performance of the dam under the earthquake loading cannot be assessed with certainty due to outdated ground motions parameters. Methods used to estimate deformation during design are no long valid. There is a better understanding of embankment performance under earthquake loading since design of the dam. Current methods to estimate seismic-induced crest settlements and cracking particularly transverse should be used to assess the risk and potential for overtopping and internal erosion resulting in failure. **(Rec-23) It is recommended assessment of seismic-induced deformations (settlement and cracking) be performed as part of the slope stability analysis of Maitai Dam following development of ground motions from the site specific seismic risk study.**

(Rec-24) It is recommended the potential for internal erosion as a result of seismic induced cracking be assessed at Maitai Dam.

8.3.2 Embankment Internal Erosion

The potential for internal erosion depends on material compatibility and internal stability of Maitai Dam's embankment and fill materials under seepage flow. The chimney drain materials have not been assessed using current standards of practice for internal erosion. Current methodology includes Foster (2001) revised criteria to assess filters of existing dams for material compatibility, which supersede recommended criteria by Sherard used in the 1993 SEED review (Riley Consultants, 1993). Recent methods by Li and Fannin (2008) may be used to assess internal stability (suffusion) of the fill materials, which has not been performed for Maitai Dam. **(Rec-25) It is recommended an assessment of potential internal erosion be performed for Maitai Dam embankment materials using current practice method.** Based on potential pore pressures within the downstream shoulder as discussed in Section 7.3.2 the performance of the chimney drain to act as a filter and a drain needs to be included in the internal erosion assessment.

The assessment of internal erosion should be based gradations of as-placed materials. A thorough record search should be performed to obtain embankment fill and drainage material gradations. Variability in the all material gradations, preferably across the embankment zones, needs be captured to understand the uncertainty in the assessment and potential risks.

Gradations of Type C drainage material could not be located following the 1998 CSR recommendations (Tonkin & Taylor, 2003). Uncertainty of Type C materials should be considered in the internal erosion assessment. Appendix B provides construction photographs of the chimney filter material. This material is most likely Type C drainage stone based on construction date of 22 November 1985, which is the time first lift placement of chimney drain occurred (Tonkin & Talyor, 1987).

Embankment Fill

This review of Type I and Type IA embankment fill indicates they are generally a gap graded materials with significant fines based on permeability results. Gap graded material are known to be prone to suffusion but this depends on the plasticity of the fines. No Atterberg Limits for borrow materials are cited in the design and construction documents. The plasticity and fines content of as-placed materials need to be defined as part of the internal erosion assessment to evaluate the potential for internal instability of the material.

Chimney Drain and Culvert Interceptor Drain

The chimney drain is a critical dam safety feature of Maitai Dam against internal erosion. It was designed to serve as a filter and drain. Over the vertical height and horizontally, materials used to construct the chimney drain vary, which may lead to material incompatibility with embankment fill, potential defects or unintended seepage paths. For example, the upper portion (R.L. 175.0 to 158.5/157.5m) of the drain consists of the finer Type A drainage material and lower portion (below R.L.158.5/157.5m) of the drain consists of the coarser Type C drainage. Type C drainage material is coarse based on Photo in Appendix B2 and may not act as a filter within the lower part of the dam. Horizontally, the upper 10m of the drain consist of an upstream 0.75m wide section of Type A and adjacent downstream 0.75m wide section of Type B, which has a drainage capacity between Type A and B. Any defect or high reservoir level conditions above the top of the chimney drain may result in seepage bypassing the upstream Type A section into the Type B section. The geotextile placed along the sides of the chimney drain may include defects and has the potential to clog over time. All these factors affect the chimney drain performance to act as a filter against internal erosion. The chimney drain purpose to act as a drain is related to maintaining the downstream shoulder unsaturated for embankment stability.

Similar to the chimney drain, construction of the culvert interceptor drain includes narrow 0.5m wide sections of three different filter/drainage materials. There is the potential for seepage to bypass the upstream section and lead to internal erosion due to construction defects from this configuration and the use of geotextile fabric and polythene sheet. The location of the culvert interceptor drain is also in the upstream shoulder where there are higher hydraulic head conditions for erosion process to occur.

The chimney drain is also of limited width (0.75m) below R.L. 165.0m. The narrow section may be susceptible to arching across and thus reduce the ability of the filter to self-heal. Additionally, any defect may limit the chimney drains performance. The construction report also documents during trenching of the chimney drain " if the Contractor allowed sections of the trench to become deeper than intended some small transverse tension crack could be observed adjacent to the trench. No such problems occurred when the 1.5m maximum depth was adhered to." There is no additional documentation on any remedial action taken during construction concerning the observed "small transverse tension cracks".

The chimney drain performance under strong ground shaking and expected seismic-induced deformations of cracking needs to be assessed **(Rec-22 repeat)**. The lateral drainage pipelines provided at R.L. 165.0m in the downstream section of the upper chimney drain should not be considered operable under earthquake loading and most likely will be sheared due to the PVC age and limited thickness.

The crest of Maitai Dam is not fully protected during high reservoir levels above the top of the chimney drain at R.L. 175.0m. As cited in Section 7.1, the historic high reservoir level was 0.04m above the top of the chimney drain. Long periods of such precedent reservoir levels have the potential for seepage induce internal erosion of embankment crest. **(Rec-26) It is recommended the risks associated with internal erosion and potential overtopping of the crest be assessed for Maitai Dam.**

8.4 Reservoir Rim Stability

The reservoir rim is in good and stable condition. Historic small slips do not exhibit gross slope instability characteristics. The current vegetated reservoir slopes provide some resistance to slope failure. The buffer zone between the reservoir and forestry activities should be maintained. Operations need to consider the reservoir drawdown rate to prevent rim instability.

Reservoir slope instability is more likely to occur as a result of strong ground shaking under a large earthquake such as the MCE. Slope failure into the reservoir may generate an impulse wave. The impulse wave depends on the volume of material displaced and relative direction of the wave. The potential impacts of the resulting wave may include a short period of overtopping, fusing of the auxiliary spillway or blocking of the spillway inlet area. The expected volume of material displaced by the slip surface is expected to be small for Maitai reservoir. The reservoir slopes consist of a thin covering of residually weathered rock and colluvium overlying bedrock. Mobilisation of the thin overlying soils in an earthquake would generate an impulse wave proportional to the failure surface area. A deep slip surface into the bedrock mudstone and sandstones is less likely to occur. A detailed study would be required to define these parameters.

8.5 Emergency Dewatering

Maitai Dam has a designated scour pipeline along with the water supply line, (refer Drawing 6516-101 AB and 6516-136 AB). The scour level is at R.L. 146.0m and has a gate valve located downstream of the culvert. The lowest intake level is at R.L. 147.0m. This would allow significant capacity to lower the reservoir in an emergency. Additionally, the discharge valve in the valve tower may be opened to maximise the drawdown rate. Risks associated with access to the valve chamber through the culvert would need to be clearly defined. The maximum design drawoff is ~5000l/s (NZSOLD, 1989).

A dewatering capability is not mandatory for a dam in New Zealand. However, it is considered a worthwhile risk reduction measure if in place. It is also encouraged in dams which have an extremely high vulnerability to earthquake damage. Maitai Dam performance under the SEE is expected to result in deformations, which need to be evaluated **(Rec-21)**. Significant damage to the upstream face could impact the connection of the intake tower to the valve chamber. It could lead to significant leakage into the downstream shoulder.

(Rec-27) It is recommended that the performance characteristics of the Maitai Dam scour offtake are understood for the purpose of emergency dam dewatering. The outlet must be maintained and tested regularly to ensure ongoing reliable function. Operating instructions must be provided in the dam O,M&S manual. Reliable operation in the event of a large earthquake must be assured.

Intake Tower

The pipe truss column that comprises the intake tower will require detailed analysis to gain a realistic assessment of its performance. The design report (section 12.4) suggests that it was conservatively designed, but the construction report (section 3.3.2) suggests that the fabrication and erection was not quite what was envisaged. The most recent tower dive inspection was performed in August 2013 (CDC). It includes details for inspection and repair/replacement as necessary of bolts, nuts, flanges, and intake gates and screens.

It is important to ensure that the strength of pipes assumed in the design is not compromised. The cathodic protection to steel should therefore be regularly inspected and minor defects made good so that section properties are available as assumed in the design.

Valve Chamber

The valve chamber structure has not been analysed, but appears to have been conservatively designed for severe uplift and earthquake loads, and its floor is anchored to rock with tensioned rock bolts (high strength Dywidag bars, with double corrosion protection).

Culvert

The culvert has not been analysed. Some photographs indicate leaching of salts through concrete joints. This does not appear to pose a structural problem, but a chemical analysis to determine quality of water seeping may be useful in future.

9.0 DAM SAFETY STATUS

In this final section, conclusions are drawn on dam safety status by bringing together the key components of the review. This is a process that links understanding of the dam structures and an assessment of their performance (through observation, analysis and interpretation). Section 7 Surveillance Review examines visual and instrumented observations in the context of both engineering design and performance expectations. Engineering criteria and assessments are provided in Section 8.

Potential failure modes and a characteristic behaviour model are recommended to be developed for Maitai Dam. An understanding of the failure modes and expected behaviour will be useful in targeting areas of importance in operation, surveillance and maintenance of the dam.

9.1 Assessment of Performance

The Maitai Dam has performed well over its 28 year life, based on surveillance data and current observations. Continued performance is also based on obtaining reliable surveillance data and understanding changes in observed behaviour. The CSR provides recommendations in surveillance instrumentation and data management to ensure reliable data, interpretation of data and timely reviews of performance.

The dam service and auxiliary spillway appears to meet acceptability criteria for flood passage.

The design of Maitai Dam includes some features that are no longer accepted in standard practice and present potential vulnerabilities. These features include:

- Construction of a narrow chimney drain below R.L. 165.0m, marginal width above and not extending above likely high water storm events.
- Placement of coarse drainage material within lower part of the chimney drain.
- Construction of the culvert interceptor drain using different material and placement under high head conditions
- Use of geotextile in majority of the drains including the chimney drain, R.H.S. culvert drain, foundation blanket drain.
- Installation of drainage lines within downstream side of the chimney drain which extend and daylight near the abutments.
- Placement of plywood facing along the upstream culvert bays.
- Penetration of culvert geometry within the embankment.
- Foundation shaping allowed for benches and sharp corners.
- Limited access to intake valves and pipework.

Assessments of the Maitai Dam need to consider these features when evaluating the potential risks and uncertainties in embankment performance.

Stability of the embankment is dependent on the chimney drain performance to act as a filter and drain and maintain unsaturated conditions in the downstream shoulder. Resolution of downstream shoulder pore pressures is required to verify the indicated acceptable slope stability under normal loading conditions.

New methodologies should be used to assess the potential for internal erosion within the embankment. Gradations of the as-placed materials should be obtained to perform this assessment. The potential for transverse cracking due to foundation shaping and culvert geometry should be considered.

This CSR makes the recommendation **(Rec-4)** to perform a site specific seismic risk study to update the earthquake loads for Maitai Dam given recent advances in ground motions hazard modelling. Assessment of the embankment under earthquake loading should include evaluation of seismic-induced deformations (crest settlement and cracking) and the potential for internal erosion.

9.2 Dam Safety Status

The dam is in good condition and is performing well. However, on-going performance and assessment of surveillance data need to consider design features which present potential vulnerabilities with regard to potential failure modes.

The dam's stability needs to be analysed following confirmation of downstream shoulder pore pressure conditions. Performance of the embankment under earthquake loading should be assessed using updated site specific ground motions and include estimation of seismic-induced deformations of cracking and the potential for internal erosion.

Assessment of the embankment fill and drainage materials for internal erosion should be performed using current methodology for existing dams based on as-placed gradations.

9.3 Dam Safety Assurance Programme (DSAP)

This section discusses Nelson City Council's dam safety assurance programme (DSAP) in the context of good industry practice and the impending Building (Dam Safety) Regulations.

9.3.1 NZSOLD Dam Safety Guidelines and the Building (Dam Safety) Regulations

The NZSOLD Dam Safety Guidelines (NZSOLD, 2000) remains the leading reference in New Zealand on industry practice for dam safety. In 2008, the Department of Building and Housing issued the Building (Dam Safety) Regulations and supporting Dam Safety Scheme: Guidance for Regional Authorities and Owners of Large Dams (Department of Building and Housing, 2008). The dam safety scheme is a risk-management regulatory regime for dams in New Zealand. The Dam Safety Scheme information updates have been provided by the now Ministry of Business Innovation and Employment (MBIE) in the form of proposed amendments to the Building (Dam Safety) Regulations (MBIE 2013, and MBIE, 2014). Whilst not yet formalised or enforced, the Dam Safety Scheme provides the most current indication of what will be formalised by March 2015. It is expected revisions of the NZSOLD Dam Safety Guidelines (NZSOLD, 2000) will be revised to reflect the requirements of the Dam Safety Scheme and subsequent amendments.

9.3.2 DSAP Requirements

The Dam Safety Scheme requires that High and Medium PIC dams have a Dam Safety Assurance Programme (DSAP), and that the DSAP activities are fulfilled (assessed annually), as approved by a 'Category A Recognised Engineer'.

Regulation 8 states the following criteria and standards for the DSAP.

The DSAP must:

- be consistent with the dam safety management principles related to operation, maintenance, surveillance, and emergency action planning as provided in the NZSOLD Dam Safety Guidelines (NZSOLD, 2000).
- be appropriate to the type and size of the dam and dam classification (PIC) given to the dam.

A DSAP must contain the following:

(a) requirements for and frequency of, routine visual inspections, instrument monitoring, data evaluation, and reporting to the dam owner,

(b) requirements for annual dam safety reviews,

(c) requirements for comprehensive dam safety reviews,

(d) details of an emergency action plan (EAP),

(e) requirements for inspection of appurtenant structures, including testing of gates and valves that contribute to reservoir safety, and

(f) procedures for the investigation, assessment and resolution of dam safety deficiencies.

Note 1: A deficiency is where a dam failure scenario is possible under certain circumstances, such that the assumed safety condition or criteria of one aspect of the dam may not be met (a potential deficiency), or, is not met (a confirmed deficiency).

9.3.3 Assessment of Maitai DSAP

The Nelson City Council (NCC) has ongoing dam safety activities for Maitai Dam. A formal Dam Safety Assurance Programme is under development. Items to be addressed are summarised in this section.

<u>Dam Safety Activities</u>
NCC's dam safety activities include monthly visual site observations and reading of instruments by surveillance inspectors. Dam safety engineering evaluation at a matching interval is not yet in place. Surveillance data is currently only evaluated at annual intervals as part of Intermediate Dam Safety Reviews (IDSR). **(Rec-28) It is recommended that procedures for ongoing surveillance activities be formalised, including a process to ensure evaluation, quality assurance and follow up of routine monthly surveillance data collected**.

An established deformation survey programme exists, however, this report makes recommendation to improve its evaluation.

The remainder of existing dam safety activities include Comprehensive Dam Safety Reviews, some deficiency resolution (although a formalised process does not yet exist) and emergency preparedness**. (Rec-29) It is recommended that procedures for the investigation, assessment and resolution of dam safety deficiencies be formalised**.

Appurtenant structures and gates and valves that contribute to reservoir safety have not been formally identified. **(Rec-30) It is recommended that Maitai Dam appurtenant structures and gates and valves that contribute to reservoir safety be formally identified and testing arrangements made.**

Emergency Action Plan

The Maitai Dam Emergency Action Plan (EAP) last issued in August 2010 (Tonkin and Taylor, 2010) meets NZSOLD requirements except for the following items:

- Incorporate the dambreak inundation maps developed (Tonkin & Taylor, 2013) and include flood travel times and depths at key populated areas and evacuation routes (**Rec-01**),
- evacuation planning including prioritised population details, methods of notification, and safe evacuation routes/destinations,
- the contact list is not complete,
- the downstream residents contacts list is not complete,
- dam dewatering information and procedures are not provided,
- sources of equipment and materials details are not complete, and
- a record of emergency action plan tests is not given.

Given Maitai Dam's High potential impact and close proximity to Nelson it is vitally important that emergency preparedness is maintained to the highest standard. In this context the above missing items are serious shortcomings. It is understood the dambreak inundation maps are were prepared (Tonkin & Taylor, 2013). However, for emergency preparedness these maps should include a table of flood travel times and depths at key populated areas and evacuation routes. It is not clear if any testing of the EAP has been performed by NCC based on this CSR review.

The fundamental objective of the EAP is to allow time for response such that dam incidents and failures are averted and/or the effects mitigated, e.g. providing sufficient notification and means for population at risk to be evacuated. **(Rec-31) It is recommended that the Maitai Dam Emergency Action Plan is completed, that NCC staff and emergency agencies become highly familiar with it, and that it is tested for effectiveness and areas identified for improvement addressed.**

Adequacy of DSAP

NCC's current dam safety activities do not conform with DSAP requirements for a High potential impact dam under current NZSOLD Guidelines and the impending Building (Dam Safety) Regulations.

The following items have been recommended to be addressed as part of the current development of a formal DSAP:

- Formalisation of ongoing surveillance activities including a process to ensure evaluation, quality assurance and follow up of routine monthly surveillance data collected,
- Completion and testing of the EAP,

- Formalisation of requirements for inspection of appurtenant structures, including testing of gates and valves that contribute to reservoir safety, and
- Formalisation of procedures for the investigation, assessment and resolution of dam safety deficiencies.

10.0 REFERENCES

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Appendix A Drawings

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM CONTRACT

SOUTH BRANCH INTAKE AND PIPELINES

AS BUILT DRAWINGS

CONSULTING ENGINEERS
TONKIN & TAYLOR LTD
in association with
WORSELDINE & WELLS

REFERENCE 6516
JUNE 1988

MAITAI WATER SUPPLY PROJECT

LIST OF DRAWINGS

AS BUILT

NORTH BRANCH DAM

6516 - 6AB LAYOUT NORTH OF SERVICE SPILLWAY

- 6ABA SEEPAGE CONTROL AND FOUNDATION CONCRETE
- 7AB LAYOUT SOUTH OF SERVICE SPILLWAY
- 8AB DAM FOUNDATION PREPARATION AND SHAPING
- 8ABA BASE PROFILES AND INITIAL CULVERT BACKFILL AT RIGHT ABUTMENT
- 9AB DAM CROSS SECTIONS
- 10AB DAM INTERNAL DRAINAGE PLAN AND DETAILS
- 10ABA LONGITUDINAL SECTION OF ABUTMENT BLANKET DRAINS
- 11AB DAM INTERNAL DRAINAGE DETAILS
- 12AB INSTRUMENTATION LAYOUT AND DETAILS
- 13AB AUXILIARY SPILLWAY FUSE PLUG DETAILS
- 14AB DOWNSTREAM DRAINAGE
- 15AB DOWNSTREAM CULVERT AREA AND CONTROL BUILDING PLATFORM GENERAL ARRANGEMENT AND DRAINAGE
- 16AB DOWNSTREAM SURFACE DRAINAGE DETAILS
- 17AB SITE ROADING AND SURFACE DRAINAGE LAYOUT
- 18AB LONGITUDINAL SECTION OF ROADS
- 19AB NORTH BRANCH CULVERT, DIVERSION INLET AND PLUNGE POOL DETAILS
- 19ABA PLUNGE POOL DETAILS
- 20AB PRELIMINARY CULVERT EXCAVATION AND LANDSCAPING WORK. AREA FOR SITE OFFICES.
- 101AB CULVERT - GENERAL ARRANGEMENT
- 102AB CULVERT JOINTS SHEET 1 OF 2
- 103AB CULVERT JOINTS SHEET 2 OF 2
- 104AB CULVERT - TYPE A REINFORCEMENT
- 105AB CULVERT - TYPE B REINFORCEMENT
- 106AB CULVERT AND TOWER METALWORK
- 111AB VALVE CHAMBER CONCRETE LAYOUT
- 112AB VALVE CHAMBER ROOF DETAILS AND CULVERT GATE
- 113AB VALVE CHAMBER REINFORCEMENT - SHEET 1 OF 5
- 114AB VALVE CHAMBER REINFORCEMENT - SHEET 2 OF 5
- 115AB VALVE CHAMBER REINFORCEMENT - SHEET 3 OF 5
- 116AB VALVE CHAMBER REINFORCEMENT - SHEET 4 OF 5
- 117AB VALVE CHAMBER REINFORCEMENT - SHEET 5 OF 5
- 118AB VALVE CHAMBER PIPEWORK LAYOUT
- 121AB INTAKE TOWER - ELEVATION AND SECTIONS
- 122AB INTAKE TOWER STEEL DETAILS - SHEET 1 OF 2
- 123AB INTAKE TOWER STEEL DETAILS - SHEET 2 OF 2
- 124AB INTAKE TOWER SPIRAL STAIRWAY
- 125AB INTAKE TOWER PLATFORM FLOOR PLAN
- 126AB INTAKE TOWER PLATFORM ROOF PLAN AND CROSS SECTIONS
- 127AB INTAKE TOWER PLATFORM DETAILS
- 131AB CULVERT EXIT AREA - CONCRETE LAYOUT
- 131ABA DUCTS AND SERVICE TRENCH DETAILS, CULVERT EXIT AREA AND CONTROL BUILDING
- 132AB CULVERT EXIT AREA. EXIT SLAB AND ANCHOR BLOCKS. REINFORCEMENT
- 133AB CULVERT EXIT AND CONTROL BUILDING PLATFORM. RETAINING WALLS. REINFORCEMENT
- 134AB VALVE CHAMBER AND CULVERT EXIT AREA. ANCHOR BLOCK. REINFORCEMENT
- 135AB CONTROL BUILDING TO MIXING BOX ANCHOR BLOCKS. DIMENSIONS AND REINFORCEMENT.
- 136AB CULVERT EXIT AREA PIPEWORK
- 137AB PIPEWORK - GENERAL ARRANGEMENT CONTROL BUILDING TO MIXING BOX MIXING BOX DIMENSIONS
- 138AB PIPE LAYOUT - CONTROL BUILDING TO MIXING BOX AND PLATFORM
- 139AB CULVERT EXIT AREA. REINFORCING AT TOP OF STAIRS
- 140AB PIPEWORK FABRICATION. REDUCER DETAILS
- 141AB CONTROL BUILDING CARCASS. PLANS AND SECTIONS
- 142AB CONTROL BUILDING. ARCHITECTURAL DETAILS. SHEET 1 OF 2
- 143AB CONTROL BUILDING. ARCHITECTURAL DETAILS. SHEET 2 OF 2
- 144AB CONTROL BUILDING FLOOR AND ROOF REINFORCEMENT

MAITAI WATER SUPPLY PROJECT

LIST OF DRAWINGS

AS BUILT

- 145AB CONTROL BUILDING WALL REINFORCEMENT AND MISC. DETAILS
- 151AB MIXING BOX ROOF DETAILS
- 152AB MIXING BOX REINFORCEMENT - SHEET 1 OF 2
- 153AB MIXING BOX REINFORCEMENT - SHEET 2 OF 2
- 201AB SERVICE SPILLWAY. PLAN AND LONGITUDINAL SECTION
- 202AB SPILLWAY. LEFT ENTRY WALL AND WEIR DIMENSIONS
- 203AB SPILLWAY. RIGHT ENTRY WALL. DIMENSIONS
- 204AB SPILLWAY. CHUTE DIMENSIONS
- 205AB SPILLWAY. FLIP BUCKET AND APRON. PLAN AND SECTIONS
- 206AB SPILLWAY. JOINT DETAILS
- 207AB SPILLWAY DRAINAGE. LAYOUT AND DETAILS
- 208AB SPILLWAY OBSERVATION MANHOLE AND DRAINAGE DETAILS
- 209AB SPILLWAY BRIDGE
- 210AB AUXILIARY SPILLWAY PLAN, SECTIONS AND DETAILS
- 211AB SPILLWAY - LEFT ENTRY WALL AND WEIR. REINFORCEMENT
- 212AB SPILLWAY - RIGHT ENTRY WALL. REINFORCEMENT
- 213AB SPILLWAY CHUTE. REINFORCEMENT
- 214AB SPILLWAY. FLIP BUCKET REINFORCEMENT
- 215AB SPILLWAY. TYPICAL WALL SECTIONS
- 216AB SPILLWAY. JOINT DETAILS 2
- 217AB SPILLWAY. BOOM DETAILS
- 218AB AUXILIARY SPILLWAY. FUSE PLUG TRIGGERING DETAILS
- 230AB FOUNDATION AND ABUTMENT LEVELS - CROSS SECTIONS
- 231AB FOUNDATION AND ABUTMENT LEVELS - CROSS SECTIONS
- 232AB FOUNDATION AND ABUTMENT LEVELS - CROSS SECTIONS
- 240AB RESERVOIR AREA - AS-BUILT CONTOURS
- 241AB RESERVOIR AREA - AS-BUILT CONTOURS
- 242AB RESERVOIR AREA - AS-BUILT CONTOURS
- 243AB RESERVOIR AREA - AS-BUILT CONTOURS

PIPE CONNECTIONS DOWNSTREAM OF NORTH BRANCH DAM AND SOUTH BRANCH SUPPLY INTAKE AND BACKFEED DISCHARGE

5574 - 500AB LOCALITY AND EXTENT OF WORKS

- 501AB SITE PLAN SHOWING CONNECTION LINES A, B & C
- 502AB PART PLAN ON CITY SUPPLY CONVECTION LINE C. TYPICAL TRENCH CROSS SECTIONS
- 503AB LONGITUDINAL SECTIONS ON 450 DIA BACKFEED CONNECTION LINE A AND 600 DIA SUPPLY CONNECTION LINE B
- 504AB LONGITUDINAL SECTION ON 600 DIA CITY SUPPLY CONNECTION LINE C
- 505AB CROSS SECTIONS 155.70 TO 377.65 - 600 DIA CITY SUPPLY CONNECTION LINE C
- 506AB PIPEWORK DETAILS, SLUICE VALVE AND DISCHARGE CHAMBER
- 507AB MEASURING CHAMBER
- 508AB SITE PLAN & LONGITUDINAL SECTIONS ON 600 DIA SUPPLY EXTENSION LINE D AND 200 DIA INTAKE SLUICE PIPE LINE F
- 509AB PART SETTING OUT PLAN, LONGITUDINAL SECTION ON 450 DIA BACKFEED EXTENSION LINE E
- 510AB PART SETTING OUT PLAN
- 511AB RIVER CROSS SECTION AT INTAKE AND ASSOCIATED DETAILS. SUBMERGED WEIR DETAILS
- 512AB INTAKE STRUCTURE PLAN, ELEVATIONS AND SECTIONS
- 513AB INTAKE STRUCTURE ROOF PLAN & H.D. BOLT DETAIL OVERFLOW WEIR SCREEN DETAILS
- 514AB INTAKE STRUCTURE SIDE INTAKE SCREEN & BAFFLE DETAILS
- 515AB SCREEN HOUSE MAIN PLANS & SECTIONS
- 516AB SCREEN HOUSE ELEVATIONS & ROOF DETAILS
- 517AB SCREEN HOUSE PRECAST PANELS FLOOR SINKING AND SCREEN DETAILS
- 518AB BACKFEED EXTENSION LINE E. THRUST BLOCK AND AERATION CHANNEL DETAILS
- 519AB COMMON DETAIL SHEET. ANCHOR BLOCKS, THRUST BLOCKS AND TABLES OF HORIZ. AND VERT. BENDS

SOUTH BRANCH SUPPLY AND BACKFEED PIPELINES

5574 - 301AB PART PLAN AND LONGITUDINAL SECTION

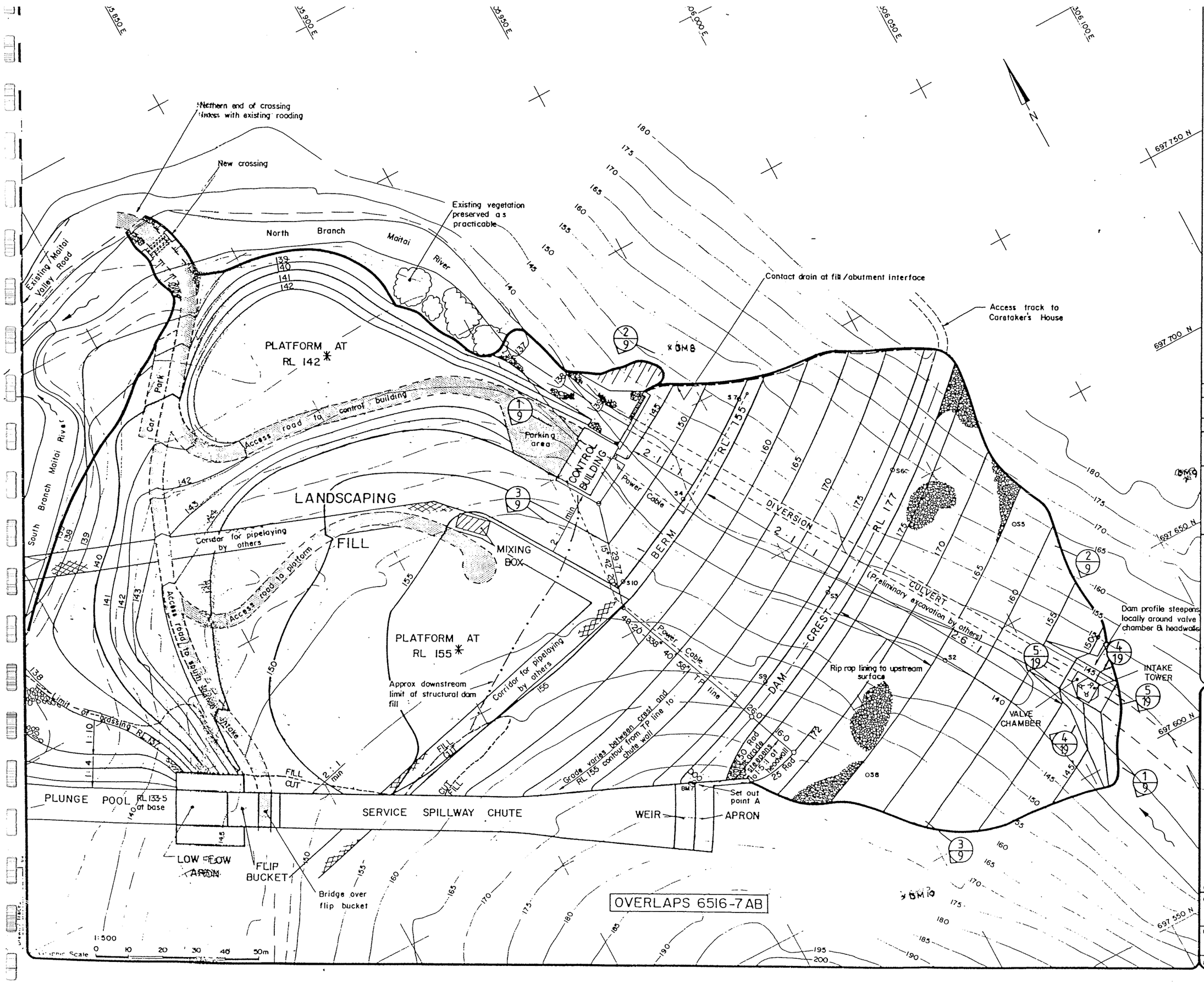
- 302AB PART PLAN AND LONGITUDINAL SECTION
- 303AB PART PLAN AND LONGITUDINAL SECTION
- 304AB CROSS SECTIONS 0.00 TO 498.47
- 305AB CROSS SECTIONS 530.53 TO 692.78
- 306AB BEDDING, RIVER CROSSING AND ANCHOR BLOCK DETAILS. TABLES OF ANGLES
- 307AB SLUICE VALVES, AIR RELEASE VALVES AND PIPE CONNECTIONS AT SPILLWAY
- 308AB AMENDED DETAILS

GEOLOGICAL PLAN

6516 - G4 GEOLOGICAL PLAN OF FOUNDATION

OVERLAY PREPARED FOUNDATION CONTOUR PLAN

Not included.



NOTES

1 For details of Preliminary culvert excavation & landscape filling by others refer Drawings 6516-20 AB

2 - - - - - - - - Signify approximate cut / fill lines

3 * Platform levels nominal only refer drawing 6516-17 AB

4 Final dam and landscaping surfaces outside road and building formations downstream from crest and above RL 137 at plunge pool ~~to be~~ topsoiled and grassed or hydroseeded. ~~as directed~~

0	First Issue		
REVISION		CHECKED	
		Initial	Date
Drawing Checked			
APPROVED			

This drawing is not to be used for construction purposes unless signed as approved

REFERENCE

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Layout North of Service Spillway

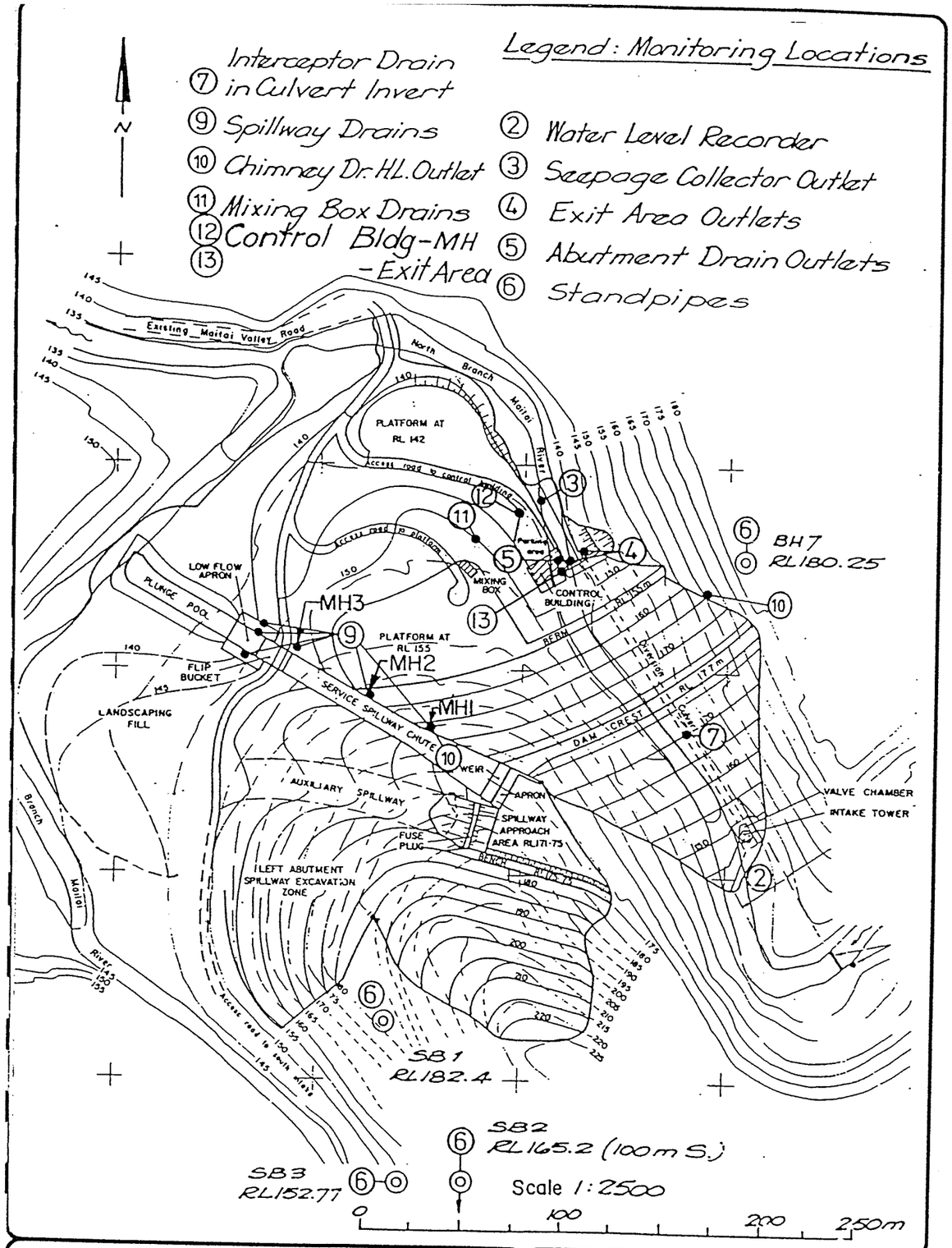
AS BUILT

ORIGINAL SCALES 1:500

DRAWING No 6516-6 AB

DATE Sept 1987

Revision 0



MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Seepage Measurement and Standpipe Locations

TONKIN & TAYLOR
CONSULTING ENGINEERS

Drawing No.	Rev.
	1
Date April 1994	
Drawn M.L.J.	Checked

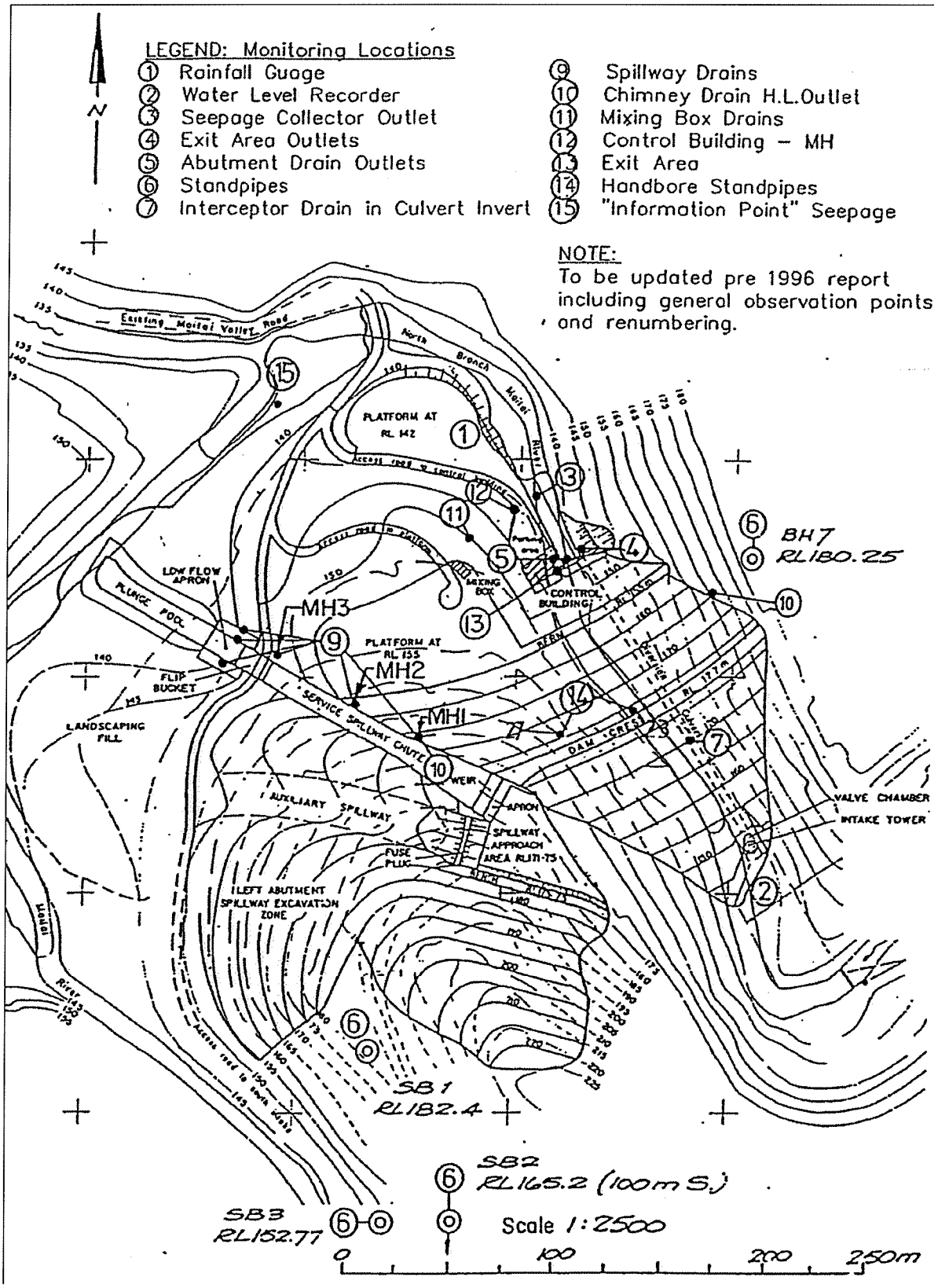
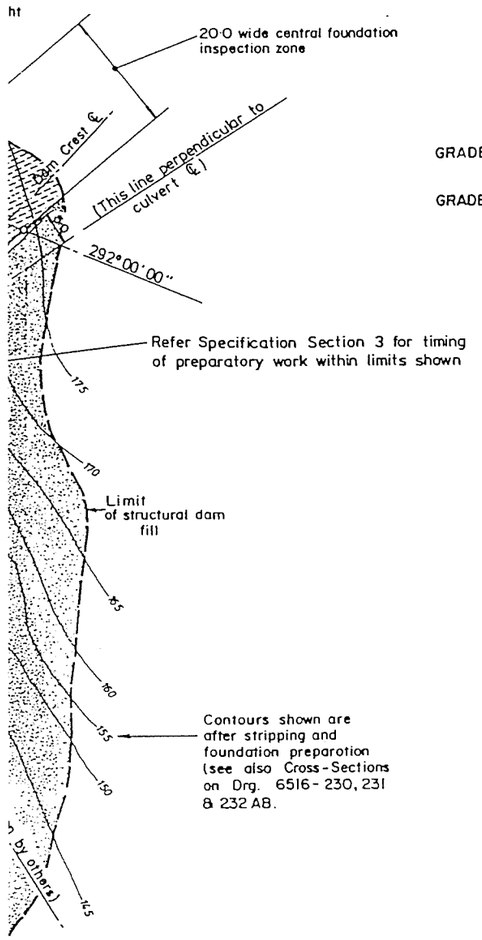


Figure 12-1 Seepage measurement and standpipe locations

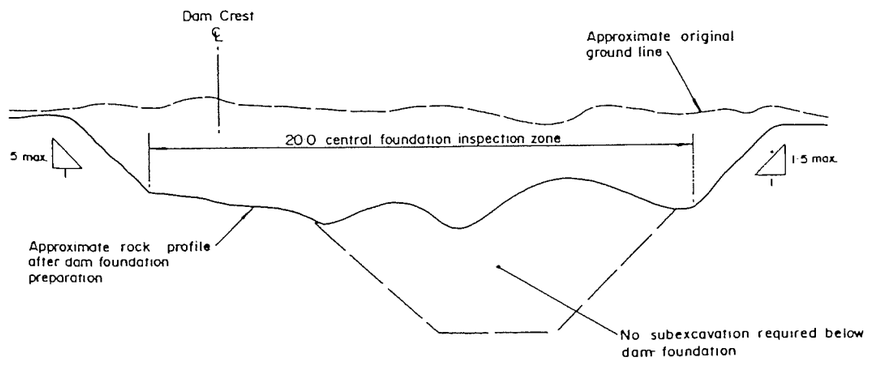


N

LEGEND

GRADE A — Foundation preparation in this area to conform to Specification – Section 3

GRADE B — Foundation preparation in this area to conform to Specification – Section 3



7/8 SCHEMATIC SECTION THROUGH CENTRAL FOUNDATION INSPECTION ZONE

N.T.S.

NOTES

1. Preliminary diversion culvert excavations carried out by others, not shown on this drawing. For details Ref. Drg. 6516-20AB
2. For foundation preparation requirements beneath the diversion culvert, refer Specification – Section 3.

REVISION		CHECKED Initial	CHECKED Date
0	First Issue		
Drawing Checked			

APPROVED ..... / ..... /

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CONSULTING CIVIL & STRUCTURAL ENGINEERS
NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Dam Foundation Preparation and Shaping

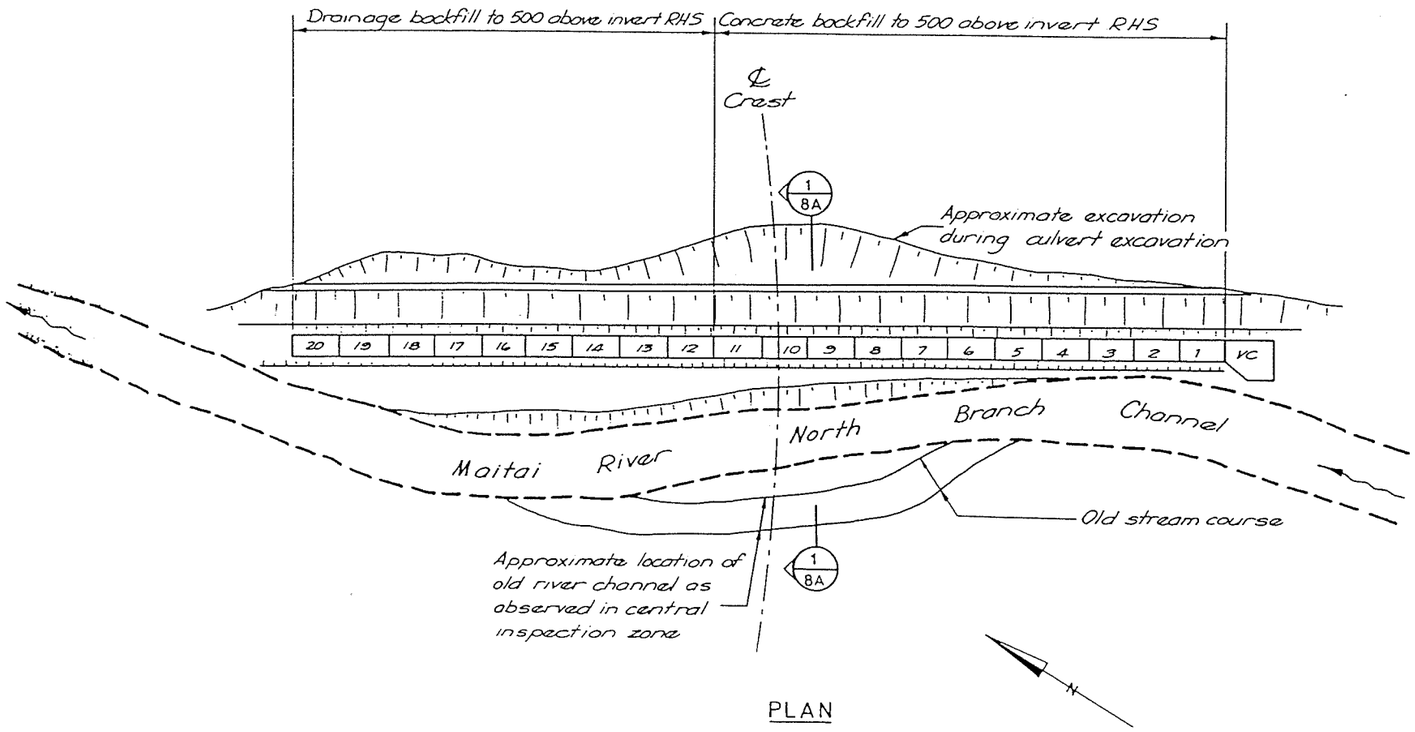
AS BUILT

ORIGINAL SCALES 1:500

DRAWING No 6516-8AB

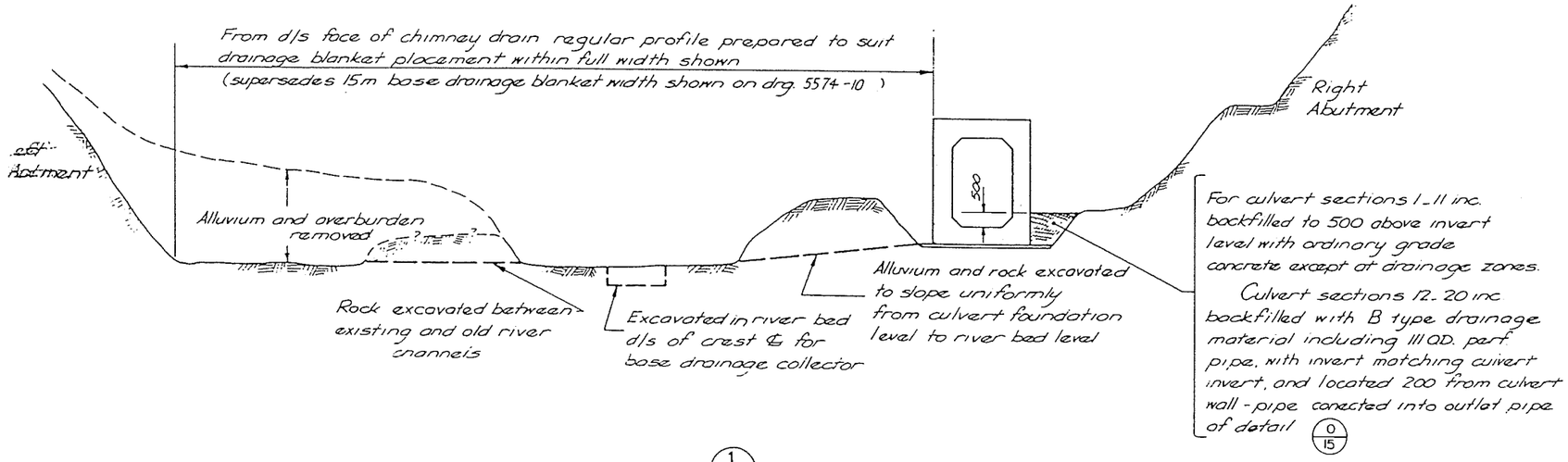
DATE Sept 1987

Revision 10



PLAN

SCALE 1 : 500



1 / 8A

SCALE 1 : 100

NOTES

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		Initial	Date
Drawing Checked			

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TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Base Profiles and Initial Culvert Backfill at Right Abutment

AS BUILT

ORIGINAL SCALES
1 : 100 1 : 500

DRAWING No 6516 - 8 ABA

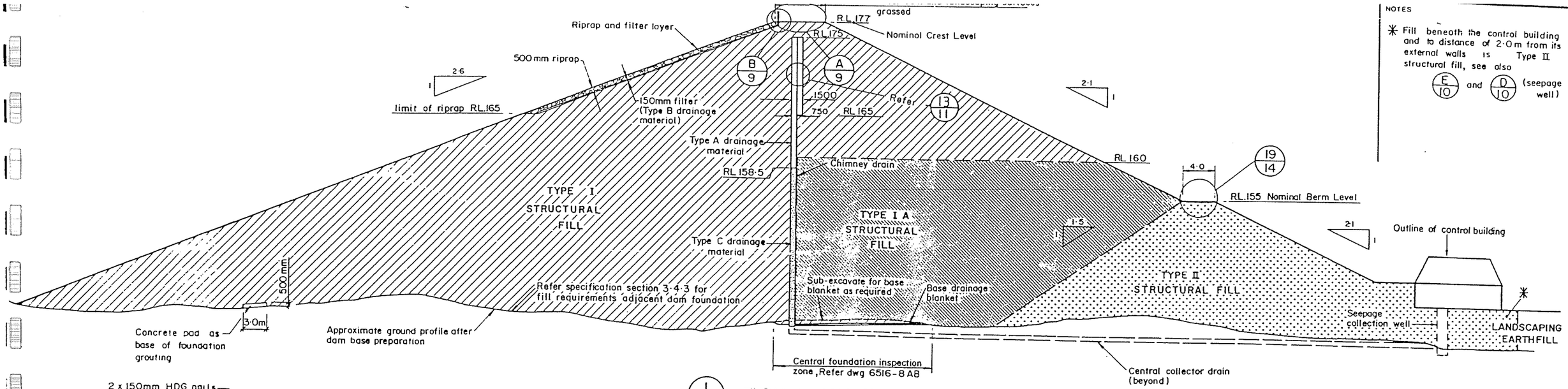
DATE Sept 1987

Revision 0

Graphic Scale

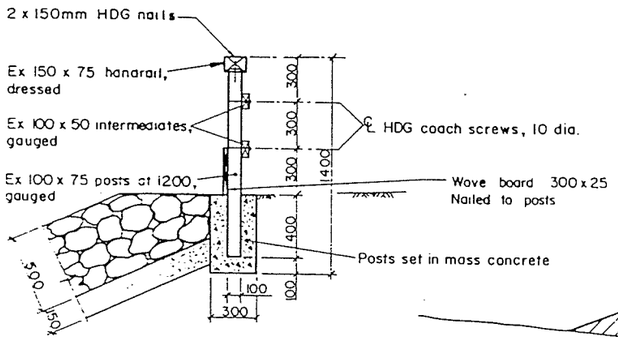
NOTES

✳ Fill beneath the control building and to distance of 2·0m from its external walls is Type II structural fill, see also E/10 and D/10 (seepage well)

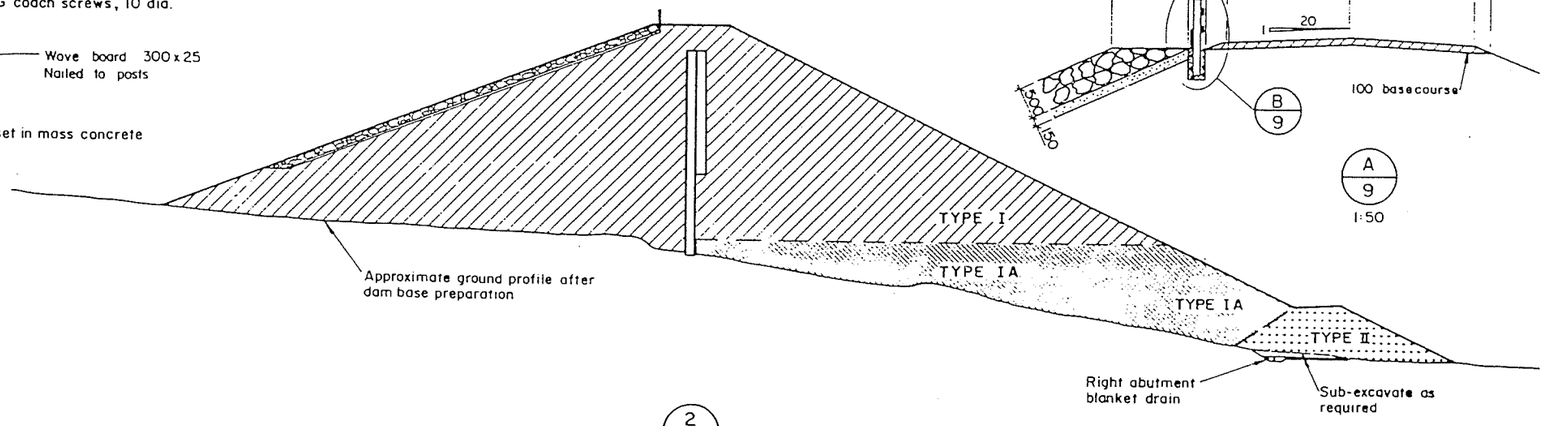


1/6
1:250

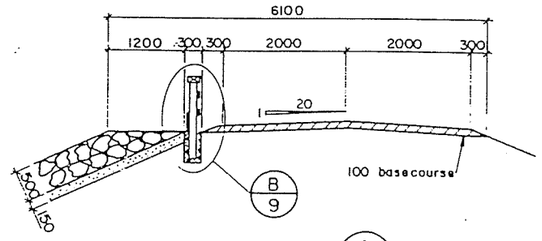
TYPICAL DAM SECTION



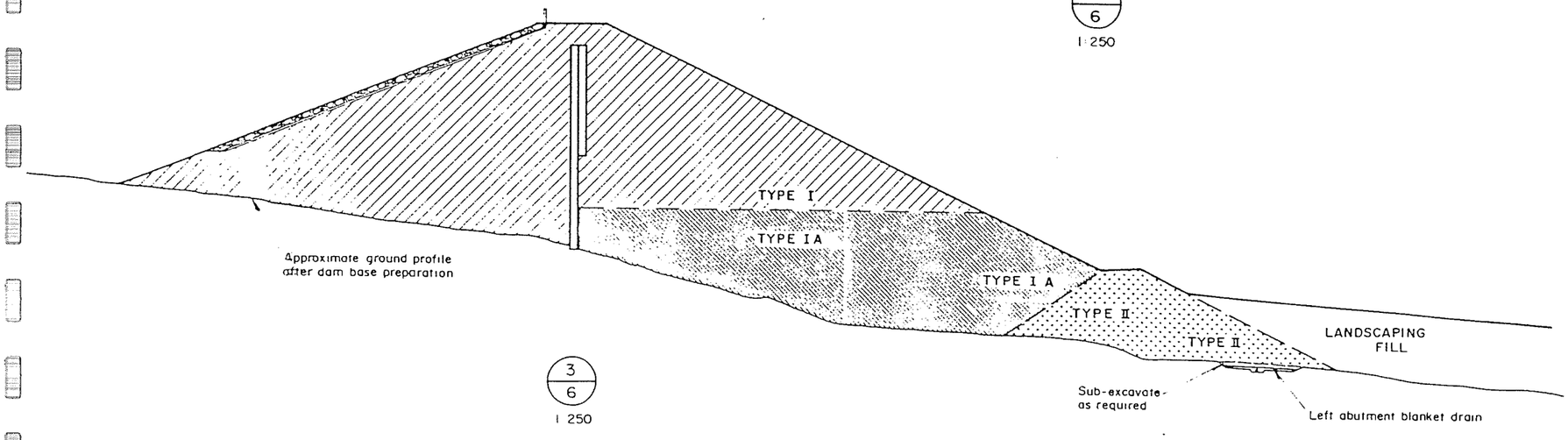
B/9
1 20



2/6
1:250



A/9
1:50



3/6
1 250

Graphic Scale

0	First Issue	
REVISION	CHECKED Initial	Date
Drawing Checked		

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Dam Cross sections

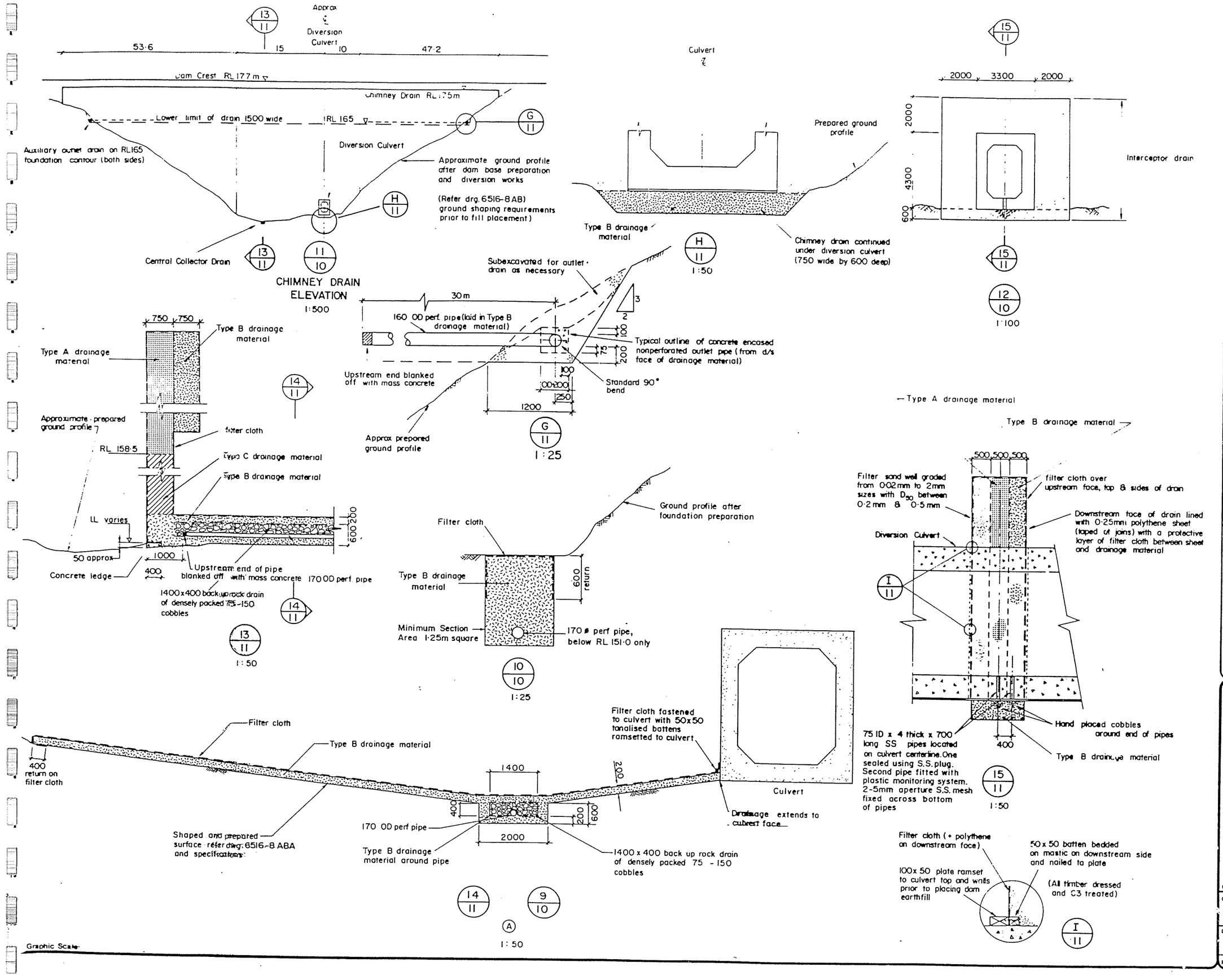
AS BUILT

ORIGINAL SCALES
1:20, 1:50, 1:250

DRAWING No 6516-9AB

DATE Sept 1987

Revision 0



NOTES

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TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

AS BUILT

Dam Internal Drainage Details

ORIGINAL SCALES
1:500 1:100 1:50 1:25

DRAWING No. 6516-11 AB

DATE Sept 1987

Revision 0

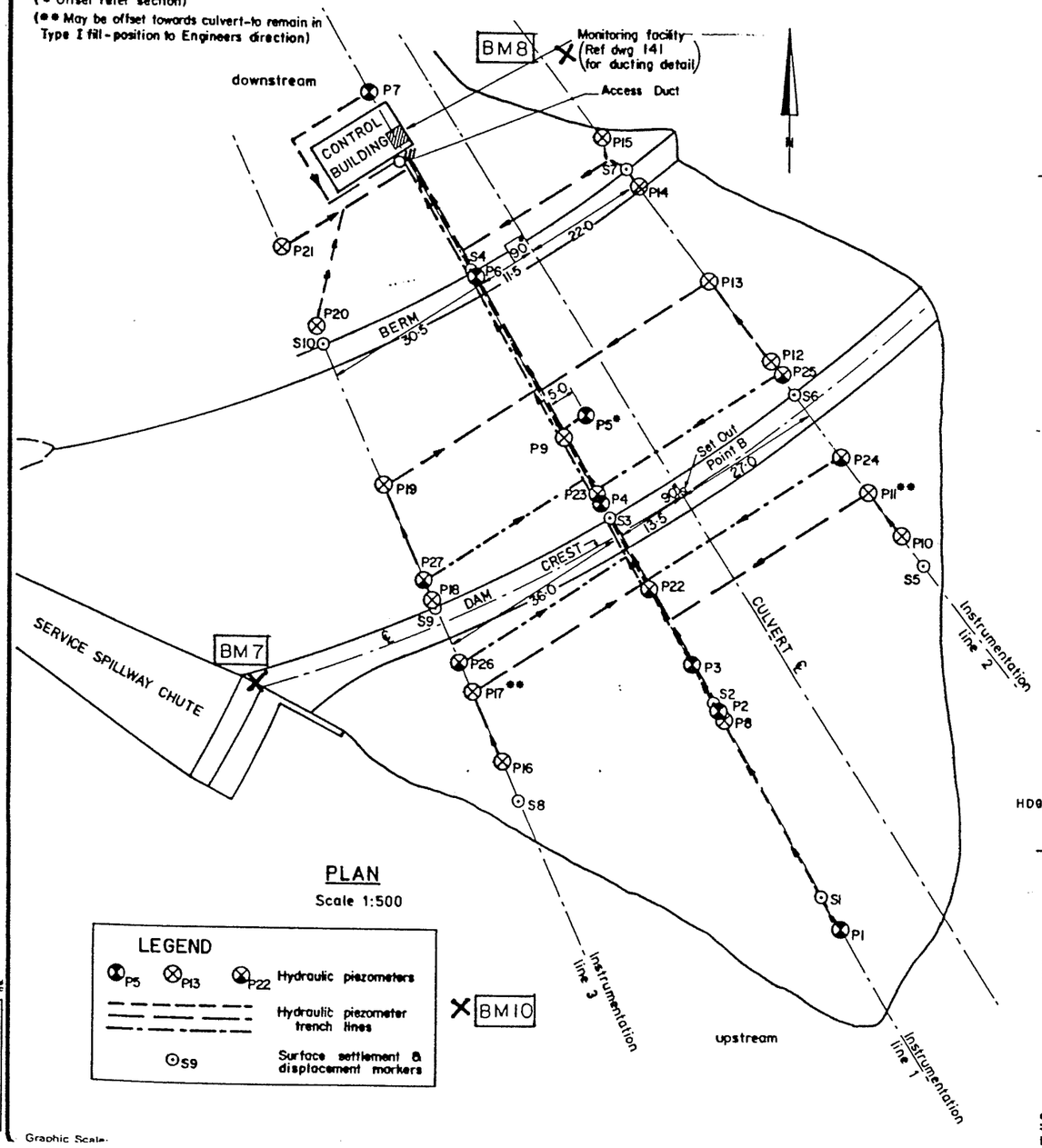
PIEZOMETER LOCATIONS

LINE 1			LINE 2			LINE 3		
Code	RL	Dist from ℄ Crest	Code	RL	Dist from ℄ Crest	Code	RL	Dist from ℄ Crest
P 1	135·28	77·5 u/s	P 10	156·5	28·0 u/s	P 16	155·6	27·0 u/s
P 2	129·8	35·5 u/s	P 11	158·0	18·5 u/s ●●	P 17	155·6	13·5 u/s ●●
P 3	143·0	26·0 u/s	P 12	149·3	10·5 d/s	P 18	150·9	4·5 d/s
P 4	143·0	6·5 d/s	P 13	156·3	28·5 d/s	P 19	156·3	26·5 d/s
P 5	137·0	21·0 d/s ●	P 14	153·5	50·5 d/s	P 20	149·4	57·3 d/s
P 6	142·1	52·0 d/s	P 15	148·4	60·5 d/s	P 21	146·45	70·3 d/s
P 7	135·7	90·0 d/s	P 24	169·25	10·5 u/s	P 26	169·3	7·5 u/s
P 8	156·9	36·5 u/s	P 25	169·35	8·0 d/s	P 27	169·35	8·5 d/s
P 9	156·5	20·0 d/s						
P 22	169·25	11·0 u/s						
P 23	169·4	7·0 d/s						

MARKER LOCATIONS

LINE 1			LINE 2			LINE 3		
Code	RL	Dist from ℄ Crest	Code	RL	Dist from ℄ Crest	Code	RL	Djst from ℄ Crest
S 1	Surface	73·2 u/s	S 5	Surface	34·2 u/s	S 8	Surface	34·2 u/s
S 2	Surface	34·2 u/s	S 6	Surface	3·0 d/s	S 9	Surface	3·0 d/s
S 3	Surface	3·0 d/s	S 7	Surface	53·2 d/s	S 10	Surface	53·2 d/s
S 4	Surface	53·2 d/s						

(● Offset refer section)
(●● May be offset towards culvert-to remain in Type I fill-position to Engineers direction)

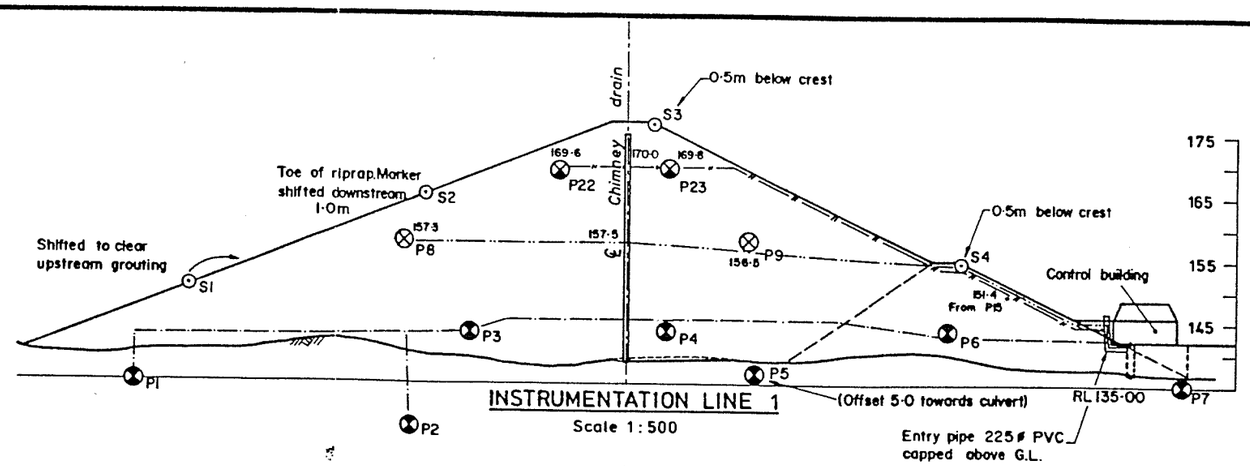


PLAN

Scale 1:500

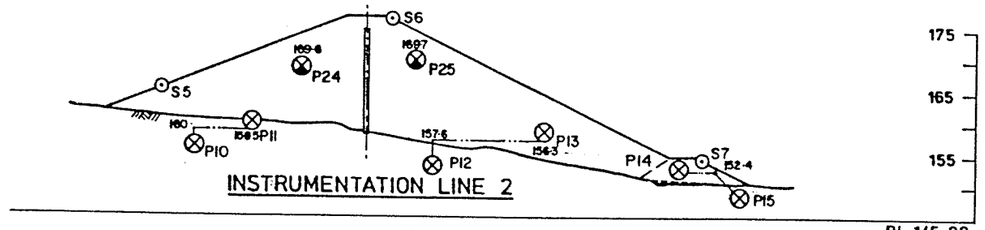
LEGEND

- P5, P13, P22 — Hydraulic piezometers
- Hydraulic piezometer trench lines
- S9 — Surface settlement & displacement markers

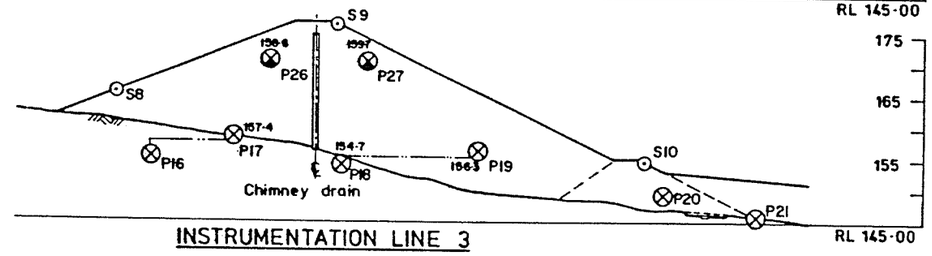


INSTRUMENTATION LINE 1

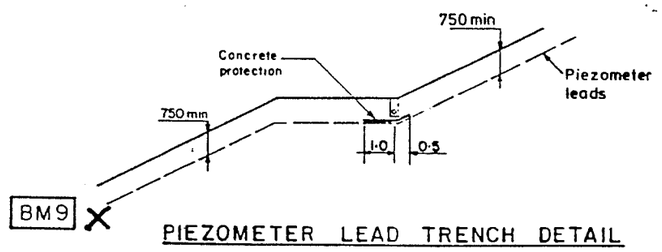
Scale 1:500



INSTRUMENTATION LINE 2



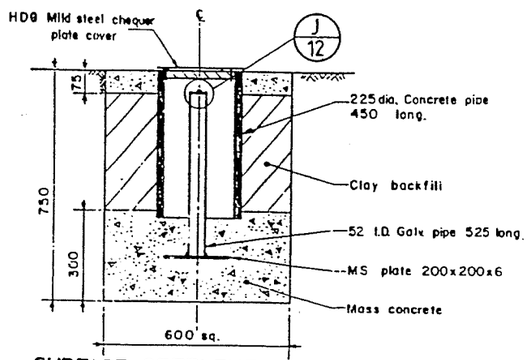
INSTRUMENTATION LINE 3



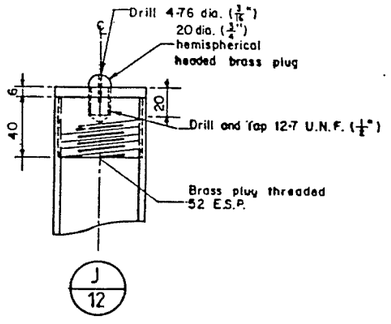
PIEZOMETER LEAD TRENCH DETAIL AT BERM

1:100

THIS DRAWING HAS BEEN REPRODUCED AT A REDUCED SCALE



SURFACE SETTLEMENT AND DISPLACEMENT MARKERS



1:2

NOTES

1 The contractor has avoided routing the instrument lead trenching though granular drainage layers where possible

2 Where the instrument leads poss through the chimney drain they are encased in HDPE pipe 1·0m long

3 The level of the fill at the time of piezometer installation was at or just above the level indicated by the piezometer symbols :-

- RL 145
- RL 158
- RL 170

4 Piezometer tip levels shown on cross sections indicative only

5 Levels on section are trench invert profiles.

Trench Profiles

6 ✕ Indicates new BM

1	BMs 7-10 added	RMK	494
0	First Issue		

REVISION — CHECKED (Initial / Date)

Drawing Checked

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Instrumentation Layout and Details

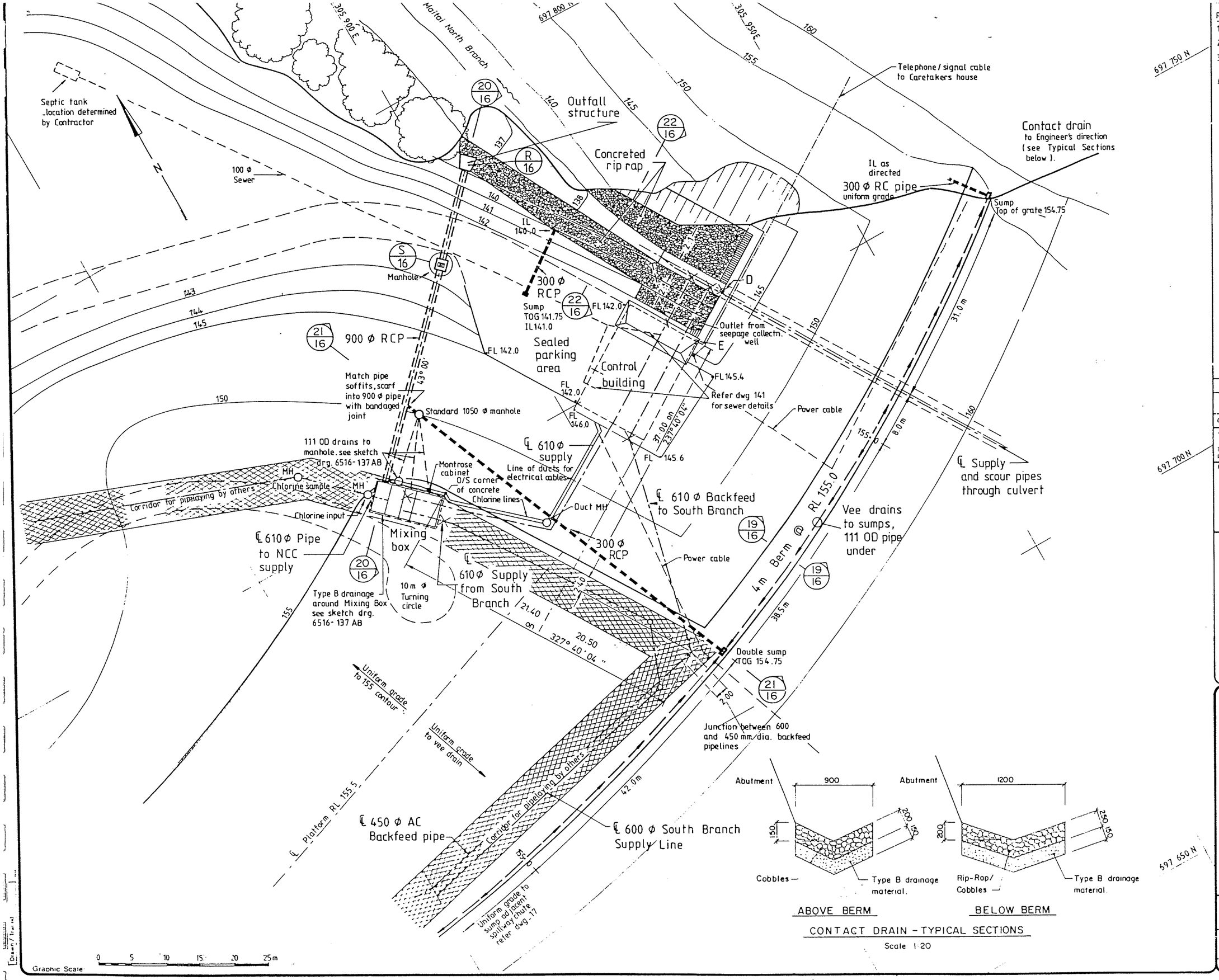
ORIGINAL SCALES 1:500 1:100 1:10 1:2

DRAWING No. FIG.

DATE APR. 94

Drawn / Traced ..... n.s.

Graphic Scale



REFERENCES

1. Road set_out data dwgs_5 & 18
2. 610 ϕ pipe set_out data dwg_138
3. Std. sump, manhole and pipe bedding details_specification
4. Mixing Box layout dwg_137

0	First Issue	
REVISION	CHECKED Initial	Date
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TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Downstream Drainage

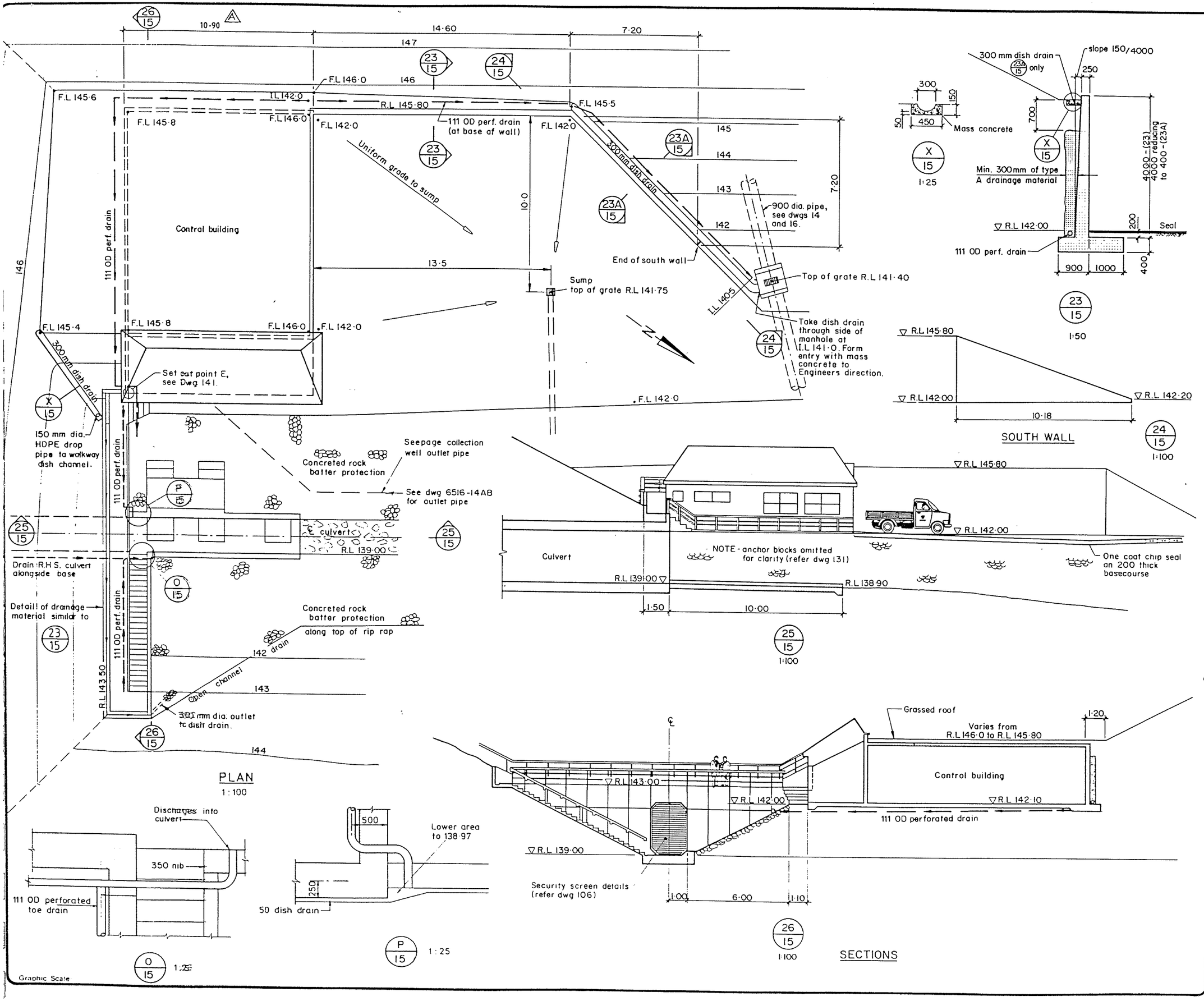
AS BUILT

ORIGINAL SCALES 1 : 250

DRAWING No 6516-14 AB

DATE Sept 1987

Revision 0



NOTES

1. Refer dwg. 141 for Control Building details.
2. Refer Specification for standard sump and pipe bedding details.
3. Refer dwg 131 for culvert exit area structural details.

0	First Issue		
REVISION		CHECKED Initial	Date
Drawing Checked			

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TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

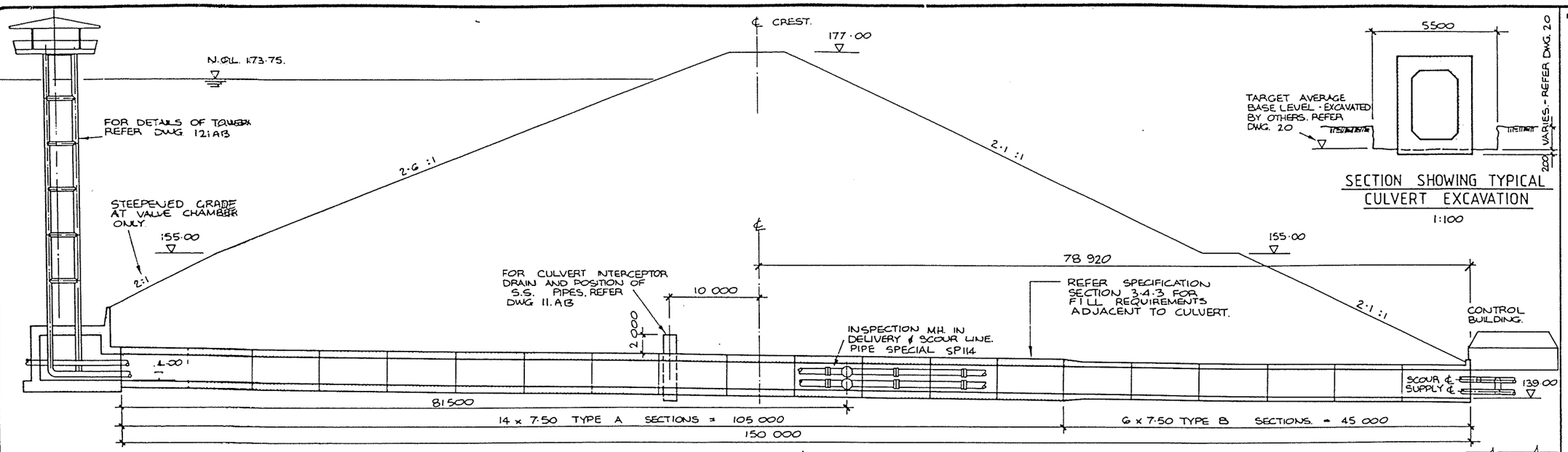
Downstream Culvert Area & Control Building Platform
AS BUILT
General Arrangement & Drainage

ORIGINAL SCALES
1:50, 1:100, 1:25

DRAWING No 6516-15 AB

DATE Sept 1987

Revision 10

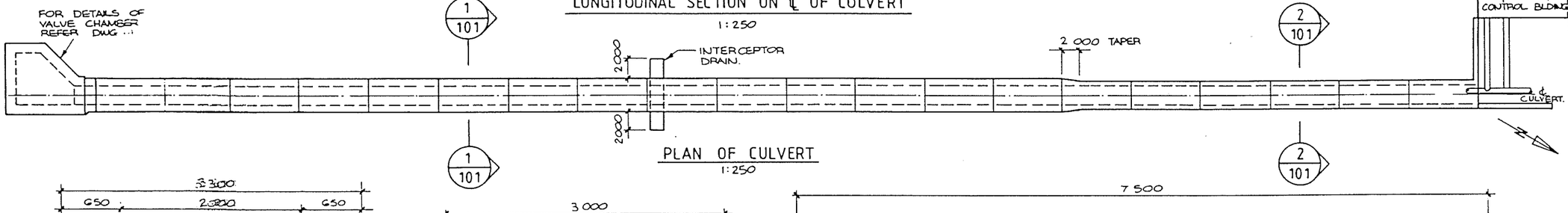


LONGITUDINAL SECTION ON ℄ OF CULVERT

1:250

SECTION SHOWING TYPICAL CULVERT EXCAVATION

1:100



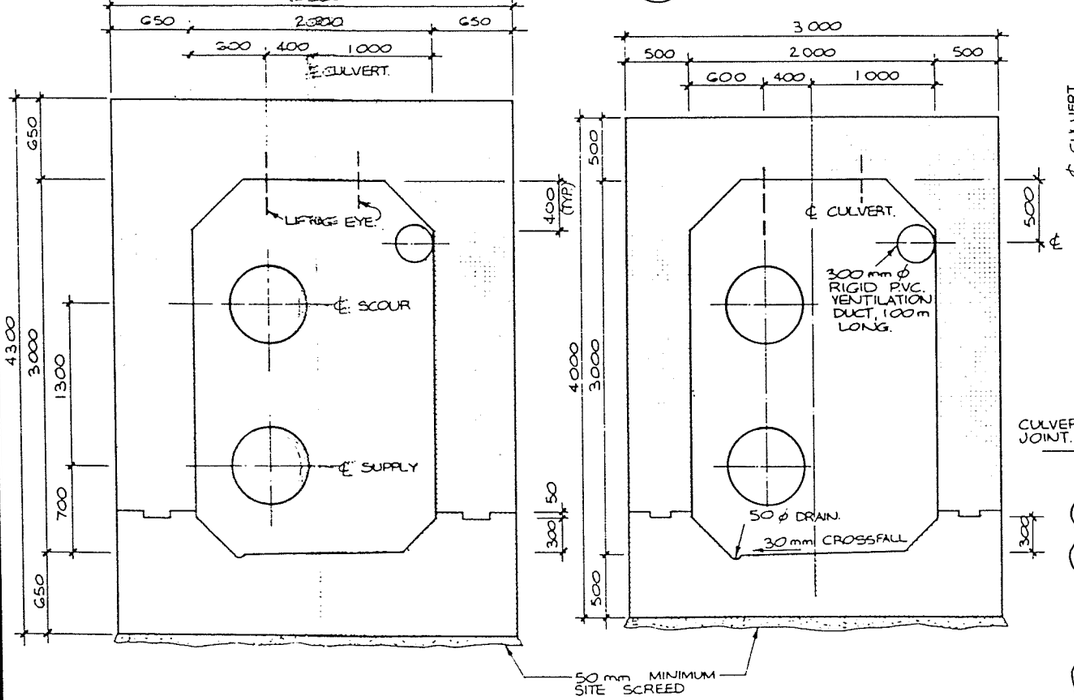
PLAN OF CULVERT

1:250

1/101

TYPE A CULVERT

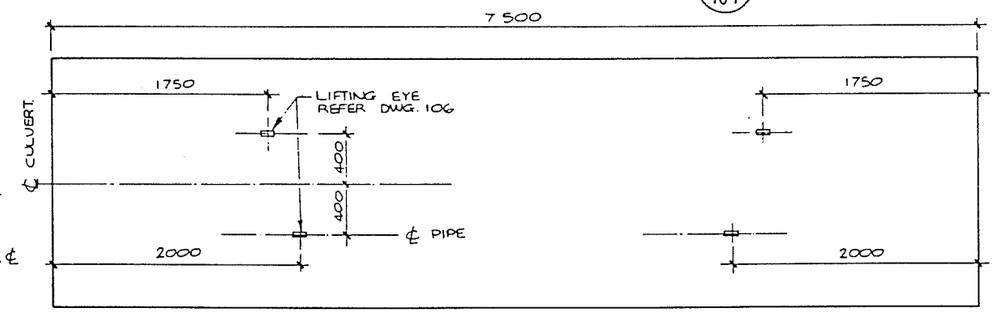
1:25



2/101

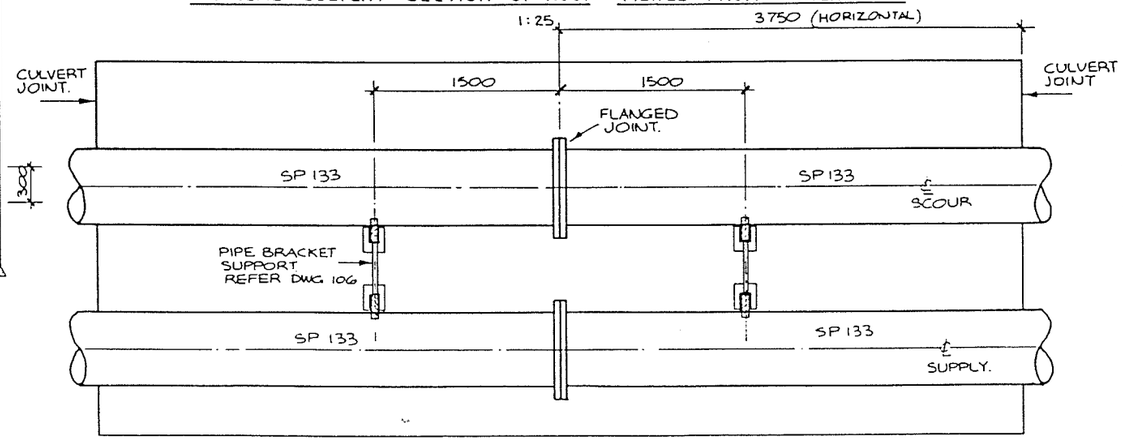
TYPE B CULVERT

1:25



TYPICAL CULVERT SECTION OF ROOF - VIEWED FROM UNDERNEATH.

1:25



TYPICAL CULVERT SECTION WEST WALL ELEVATION — VIEWED FROM INSIDE

1:25

Graphic Scale

NOTES

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First Issue		
Drawing Checked		

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Culvert
General Arrangement

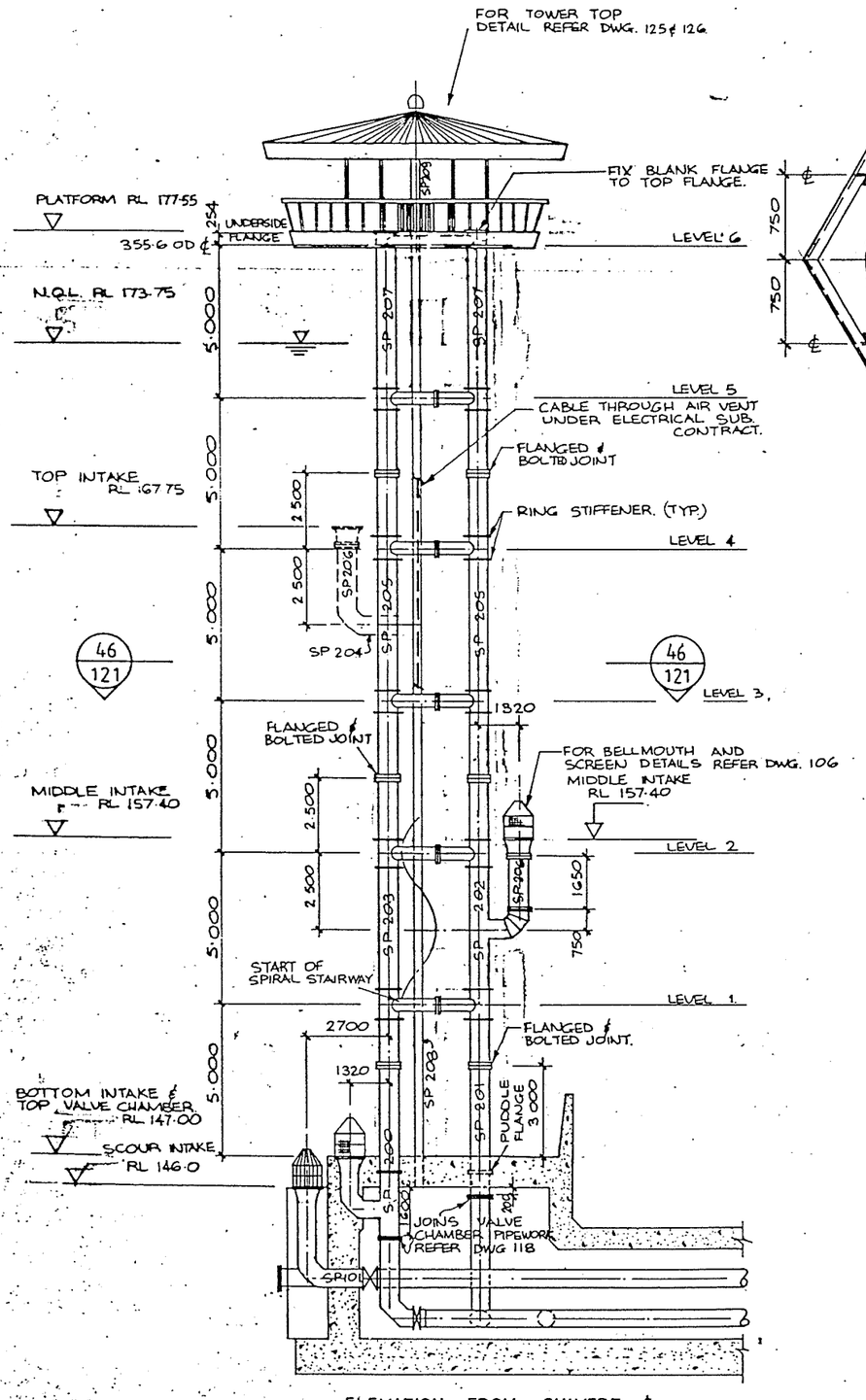
AS BUILT

ORIGINAL SCALES
1:25, 1:100, 1:250.

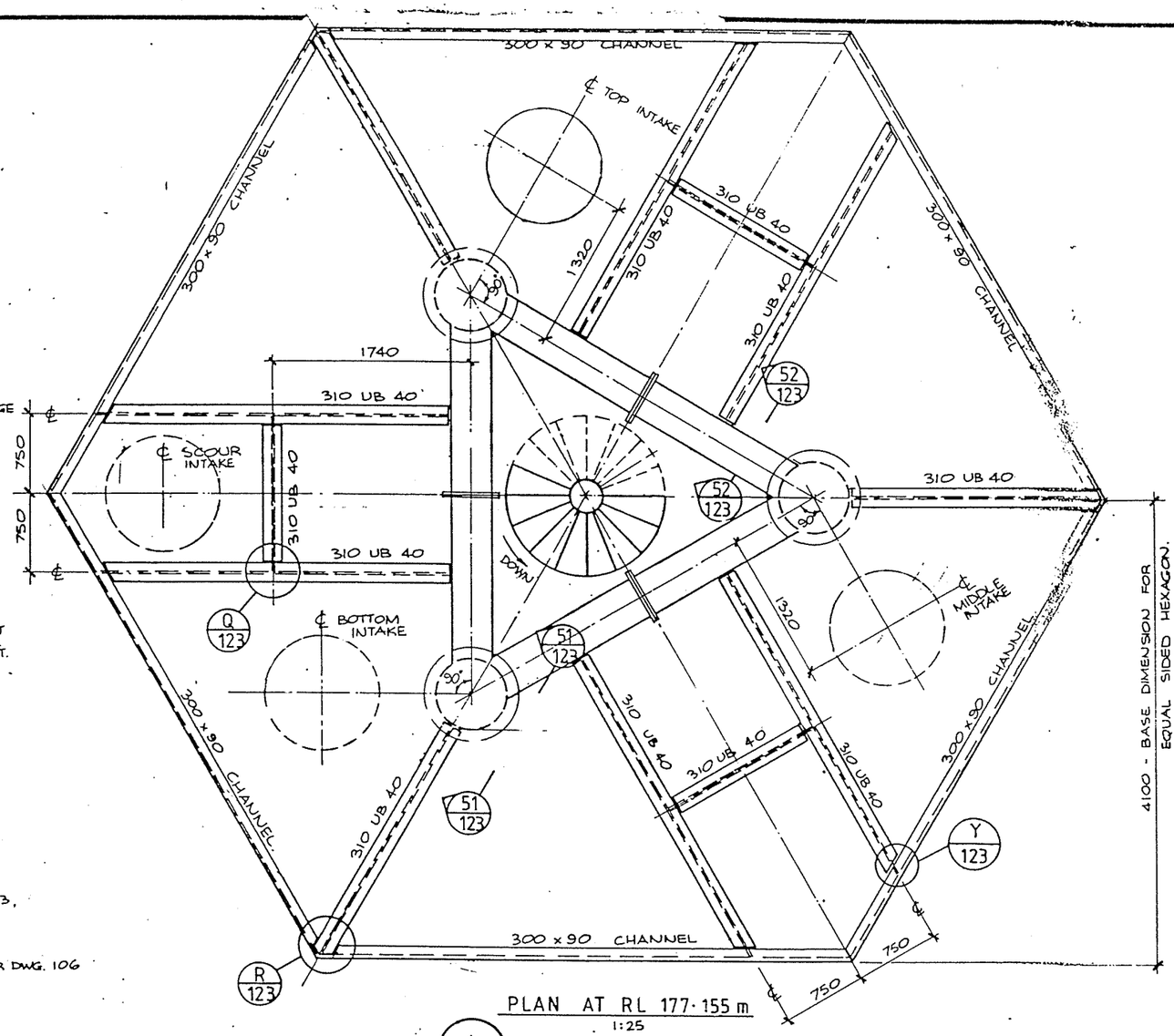
DRAWING No 6516-101 AB

DATE Oct 1987

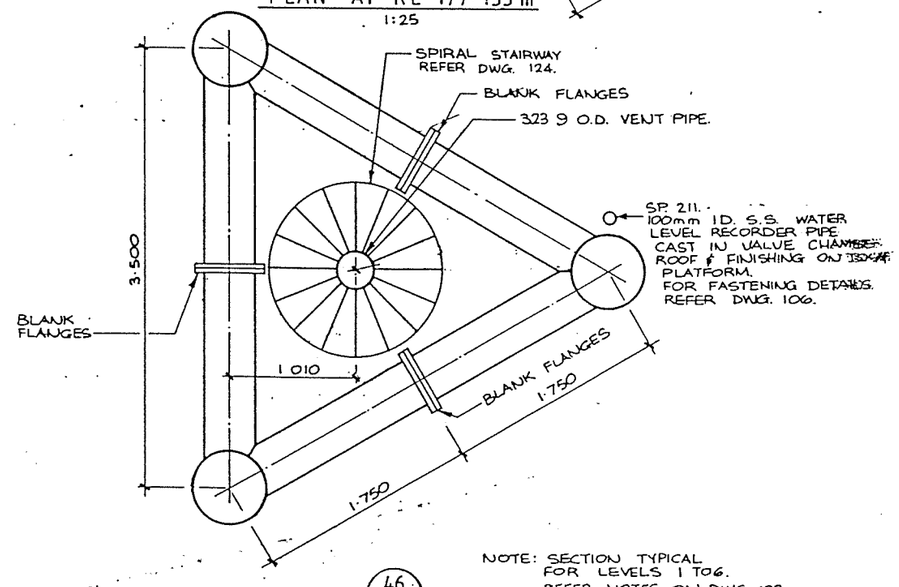
Revision



ELEVATION FROM CULVERT ℄
1:100



PLAN AT RL 177·155 m
1:25



NOTE: SECTION TYPICAL FOR LEVELS 1 TO 6. REFER NOTES ON DWG. 122.

1:25

NOTES

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Intake tower
Elevation and sections.

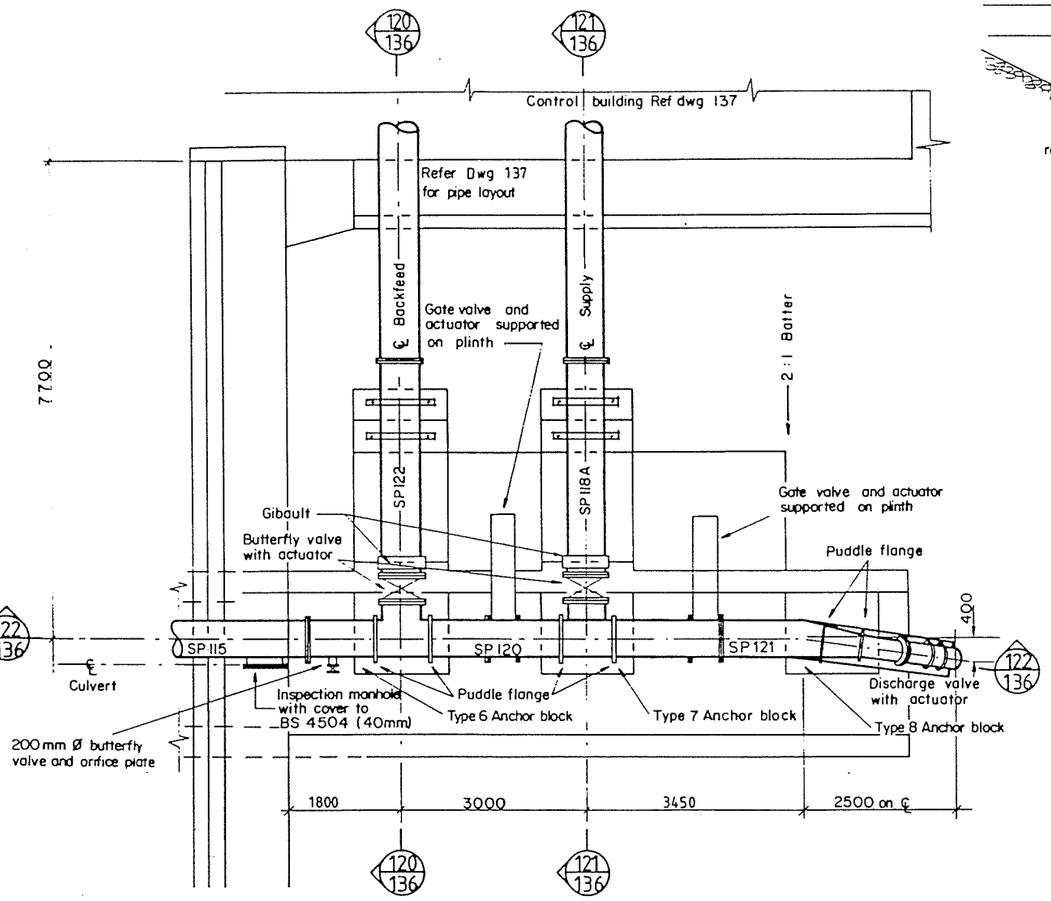
AS BUILT

ORIGINAL SCALES
1:25 , 1:100

DRAWING No. 6516-121 AB

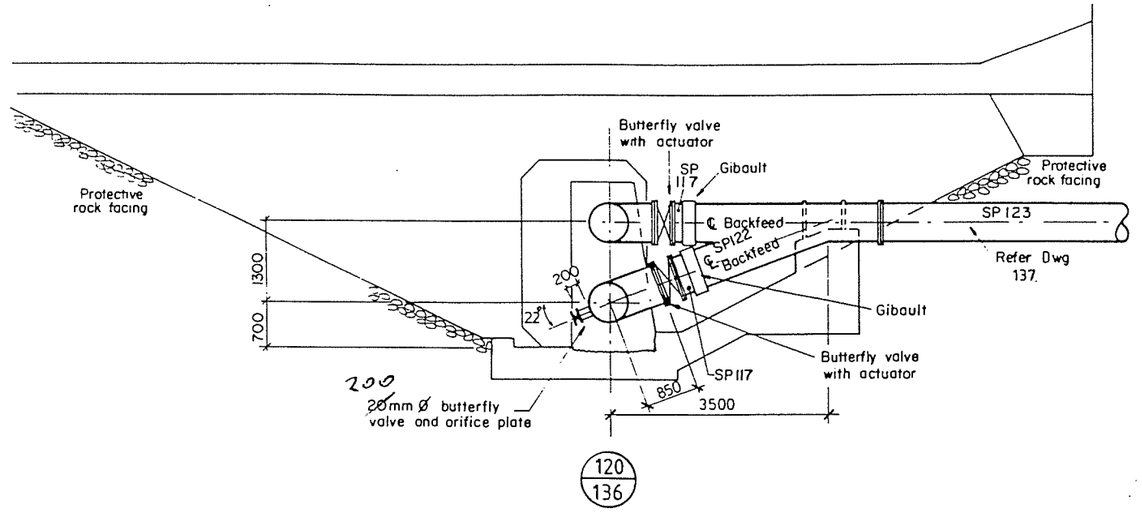
DATE Oct 1987

Revision 0

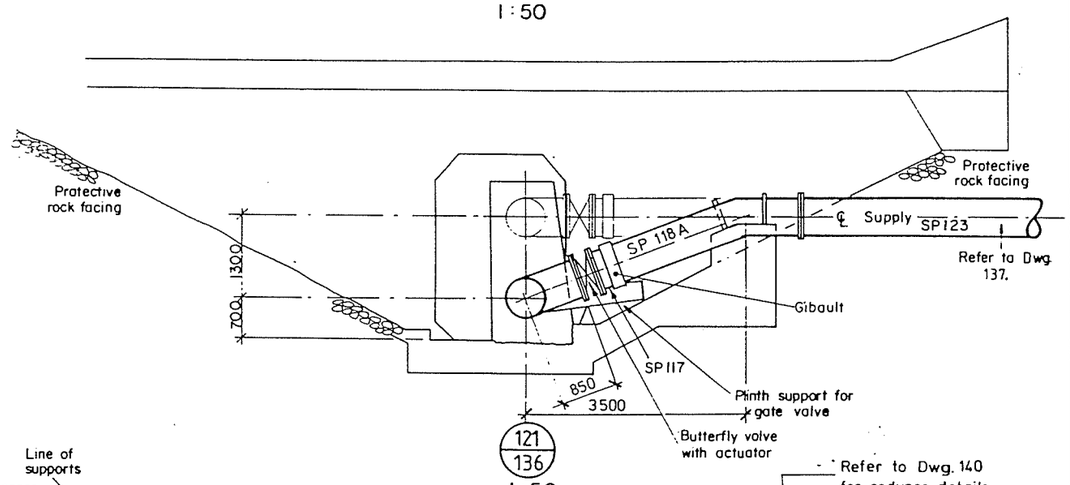


PLAN OF CULVERT EXIT PIPEWORK

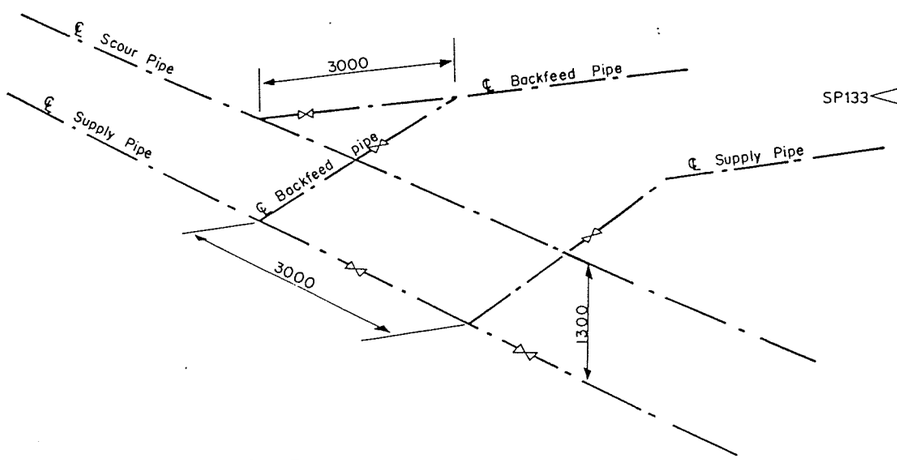
1:50



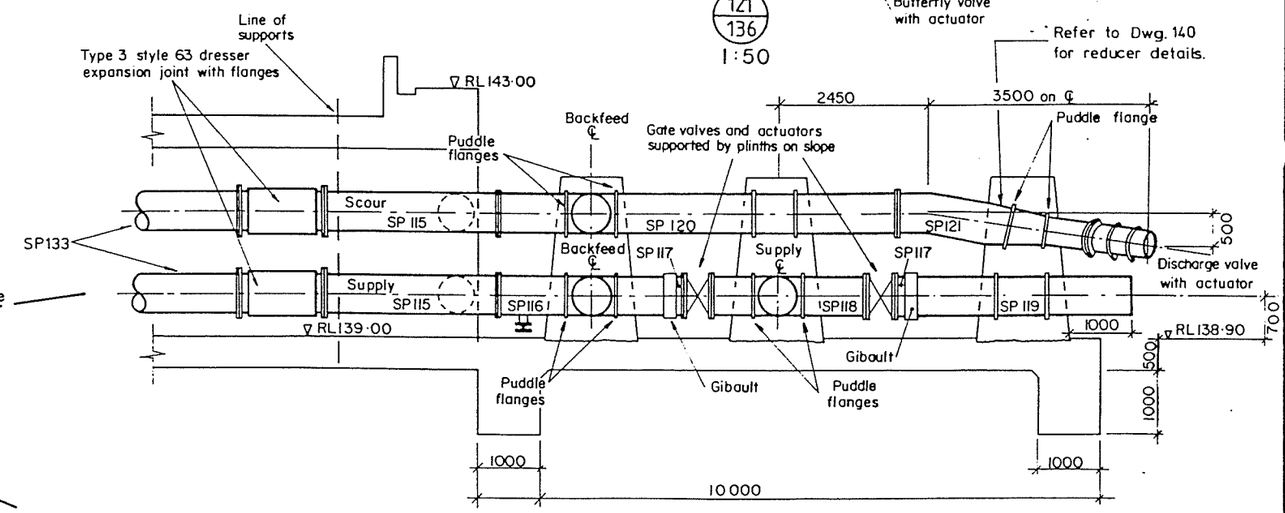
1:50



1:50



ISOMETRIC PIPE ARRANGEMENT DOWNSTREAM OF CULVERT



1:50

Graphic Scale

NOTES

1 All flanges 40mm thick and conforming to BS 4504 table 10/3 unless otherwise specified

2 All pipe bends conform to NZS 4442 unless shown otherwise

0	First Issue		
REVISION		CHECKED Initial	Date

Drawing Checked

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NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM

Culvert Exit Area Pipework

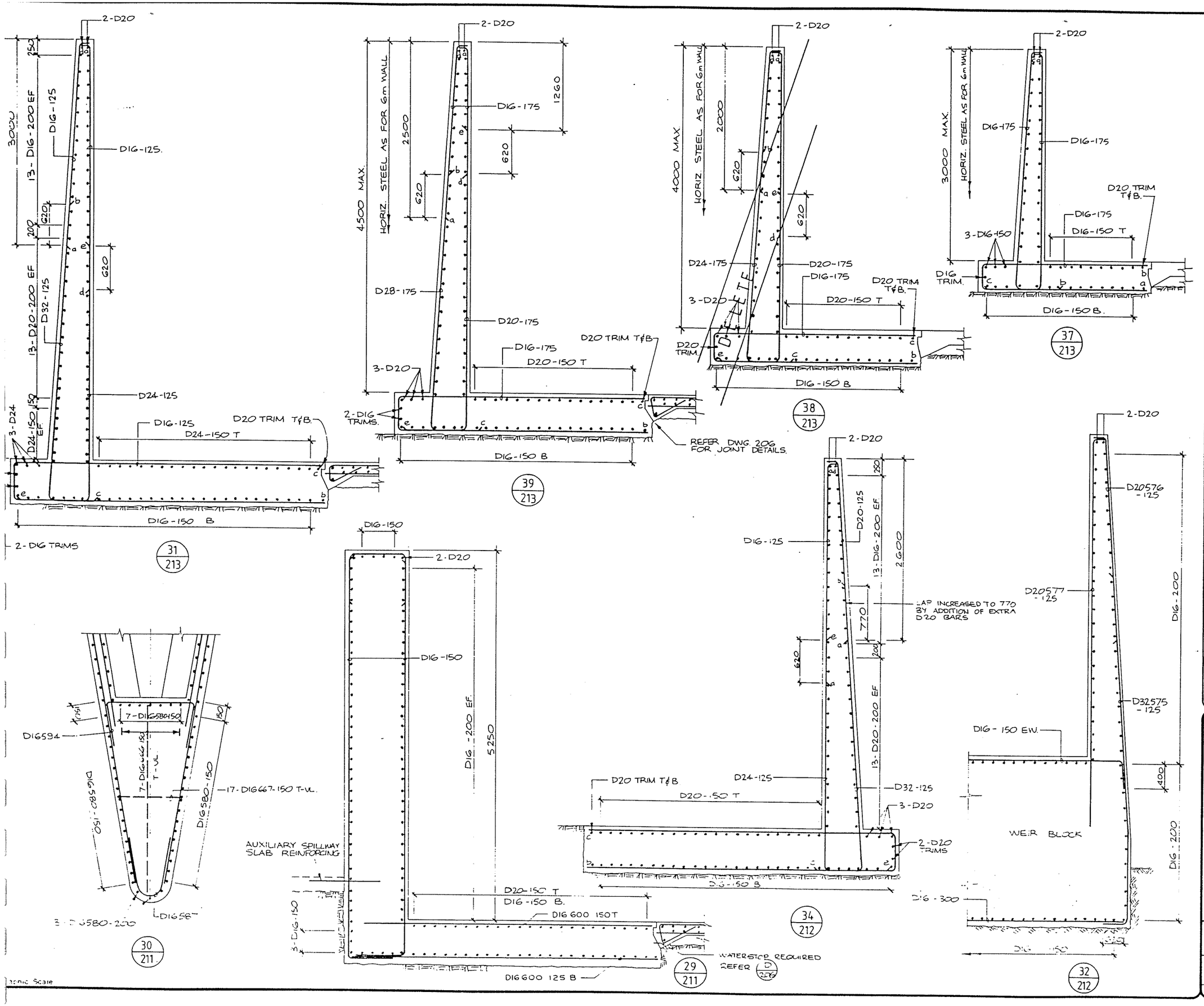
AS BUILT

ORIGINAL SCALES 1:50

DRAWING No 6516-136 AB

DATE Oct 1987

Revision 01



NOTES

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TONKIN & TAYLOR LTD

CONSULTING ENGINEERS
REGISTERED SURVEYORS
WELLINGTON

IN ASSOCIATION WITH
WORSELDINE & WELLS
CONSULTING CIVIL & STRUCTURAL ENGINEERS
NELSON

TITLE

NELSON CITY COUNCIL

MAITAI WATER SUPPLY PROJECT

NORTH BRANCH DAM
Spillway
Typical wall sections.

AS BUILT

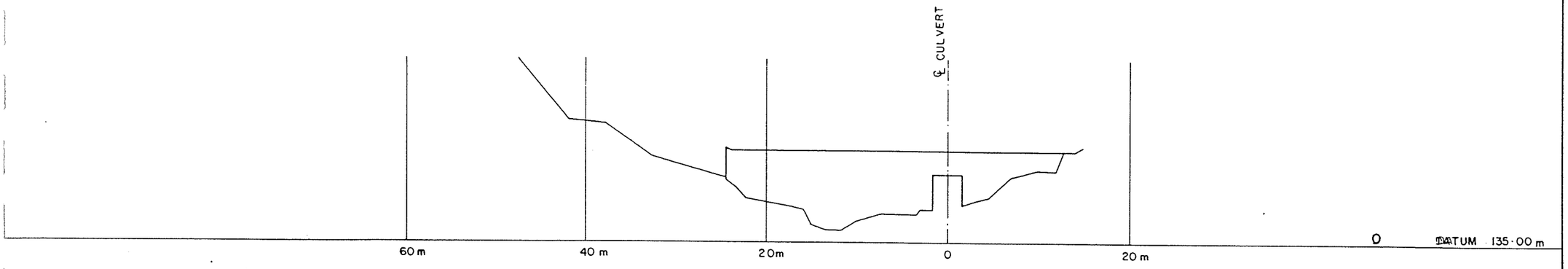
ORIGINAL SCALES 1:25

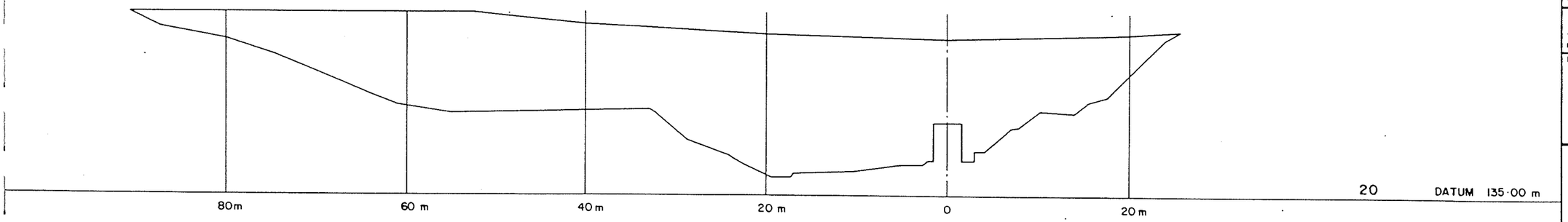
DRAWING No 6516-215 AB

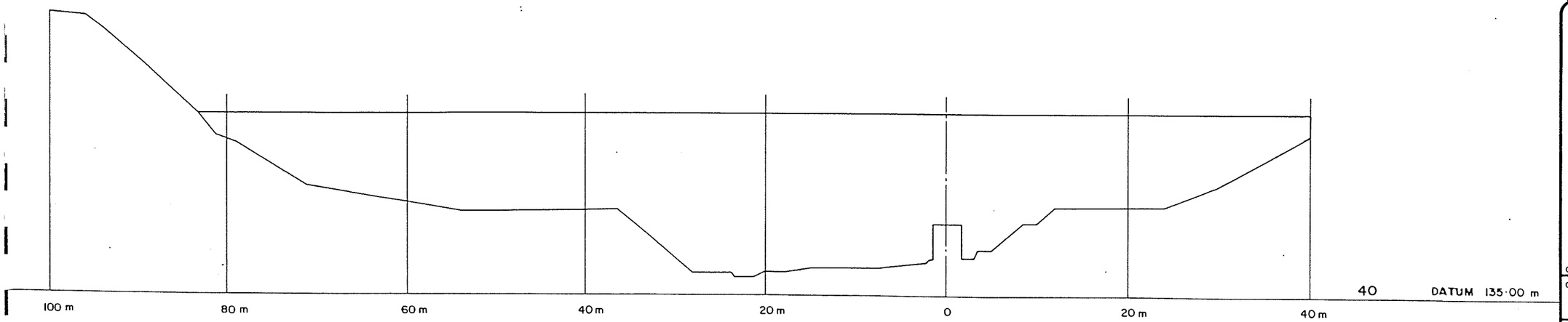
DATE DEC 1987

Revision 10

Scale







Graphic Scale:

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TOWN PLANNERS

19 Morgan Street, Newmarket, Auckland
101 Molesworth Street, Wellington
7 Hunt Street, Whangarei
111 Cameron Road, Tauranga

TITLE

NELSON CITY COUNCIL

MAITAI DAM

FOUNDATION & ABUTMENT LEVELS

Cross Sections

ORIGINAL SCALES

1 : 250

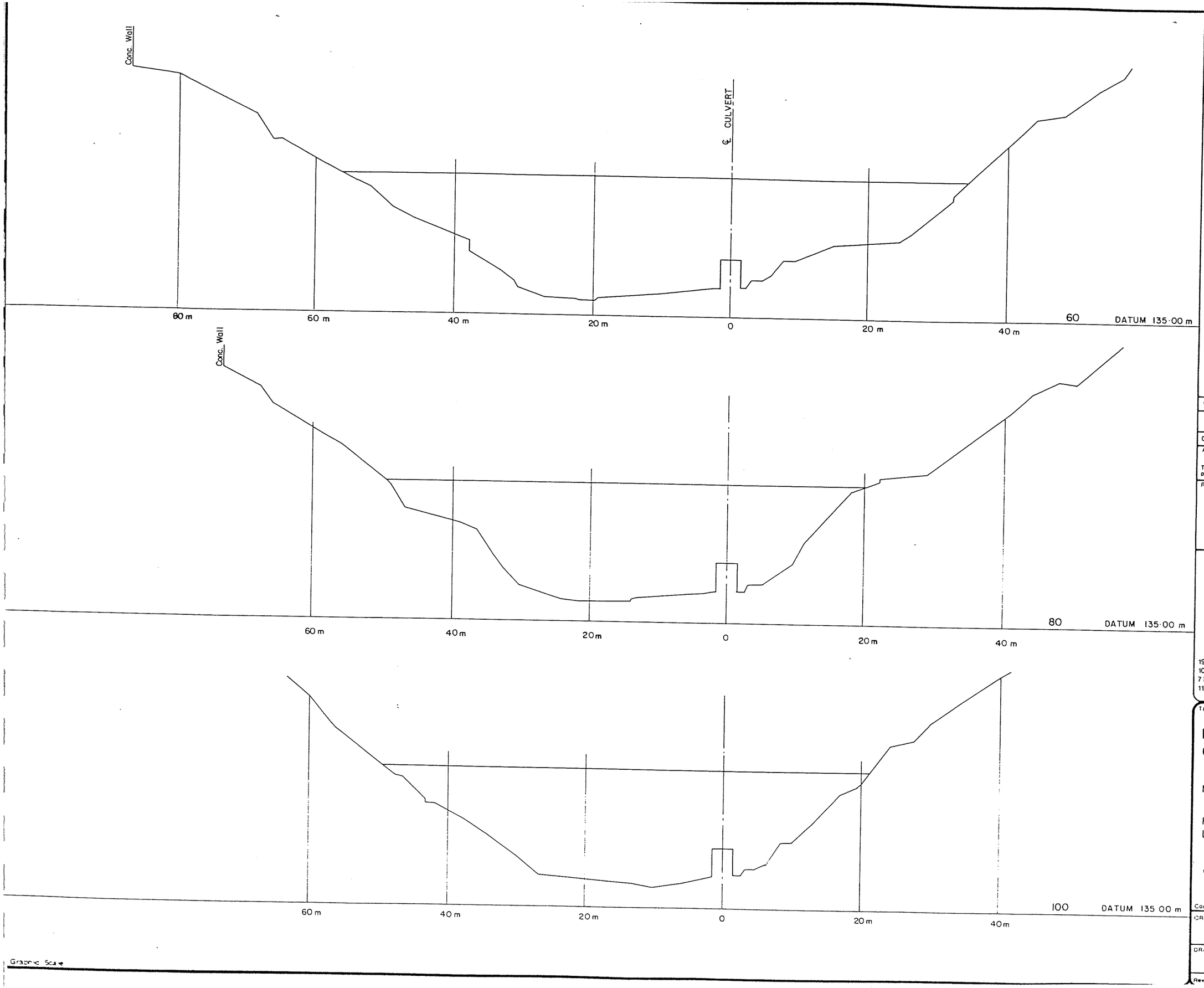
DRAWING No

6516 - 230AB

DATE

MARCH '88

Revision 10



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7 Hunt Street, Whangarei
111 Cameron Road, Tauranga

TITLE

NELSON CITY COUNCIL

MAITAI DAM

FOUNDATION & ABUTMENT LEVELS

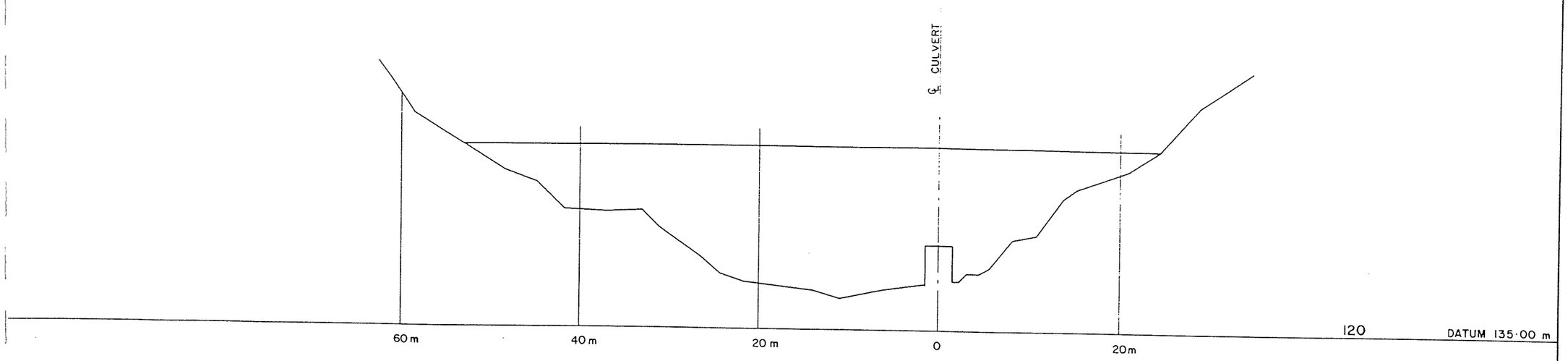
Cross Sections

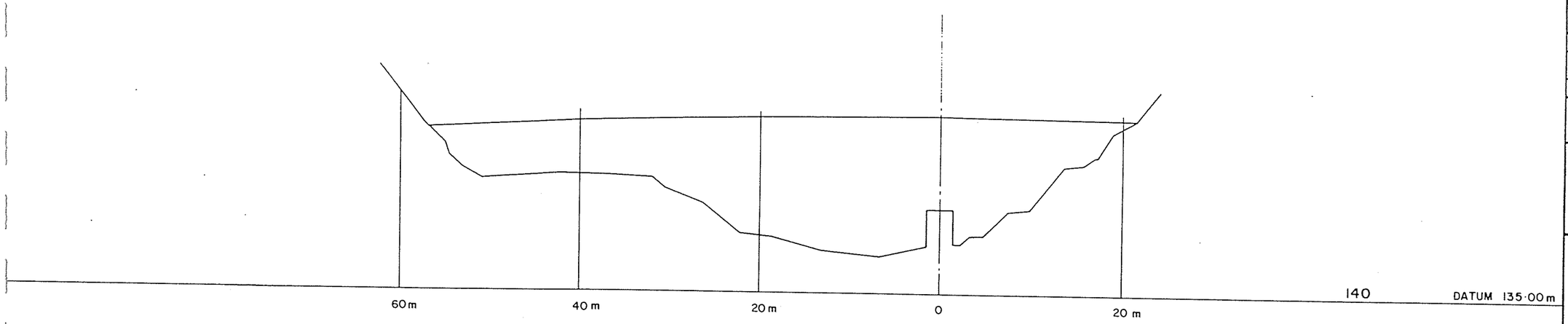
ORIGINAL SCALES

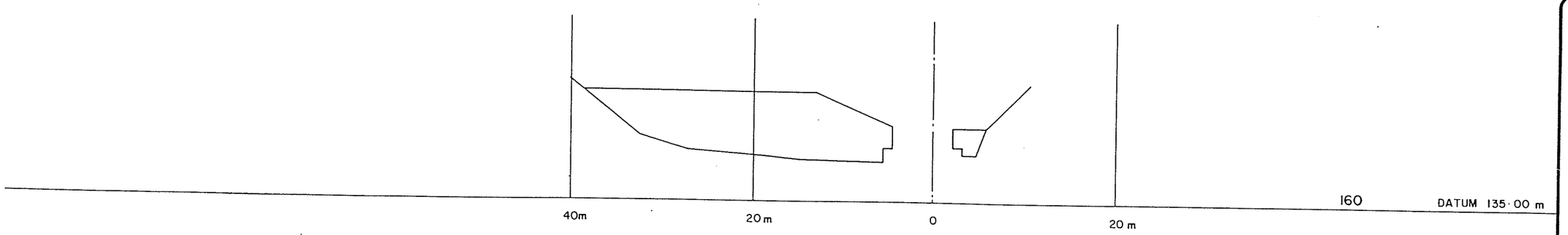
1:250

DRAWING No	DATE
6516-231 AB	MARCH '88

Revision 0







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7 Hunt Street, Whangarei
111 Cameron Road, Tauranga

TITLE

NELSON CITY COUNCIL

MAITAI DAM

FOUNDATION & ABUTMENT LEVELS

Cross Sections

ORIGINAL SCALES
1 : 250

DRAWING No. 6516 - 232 AB

DATE MARCH '88

Appendix B Construction Photographs

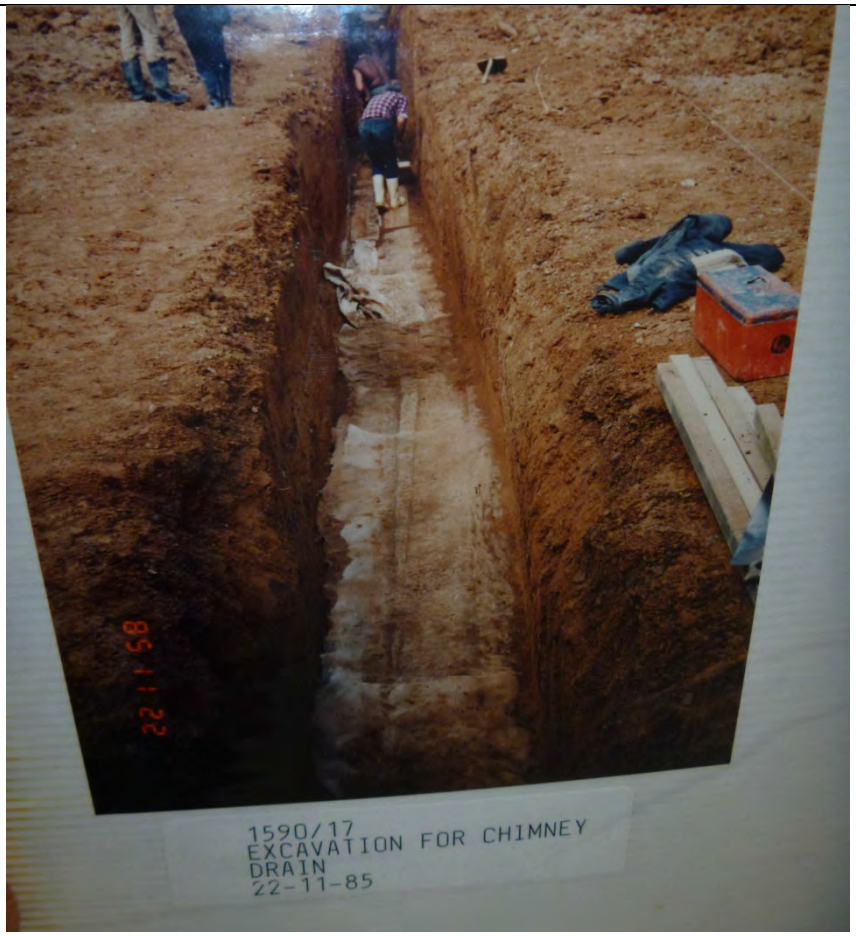

Figure B24: Trench of Chimney Drain to Expose Geotextile Covering of Previous Lift, 22/11/85.

Figure B25: Placement of Drainage Material (likely Type C) with Geotextile, 22/11/85.

Figure B26: Placement of Foundation Blanket Drainage Material, 25/10/85

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Photo 27: Upstream Embankment Slope (L.H.S.)

Photo 28: Upstream Embankment Slope (R.H.S.)

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Photo 4 Upstream Embankment Slope, New Rip-Rap

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Photo 7: Downstream Slope and Control Building

Photo 8: Downstream Slope, Looking at Right Abutment

Photo 9: Downstream Slope, Looking at Left Abutment

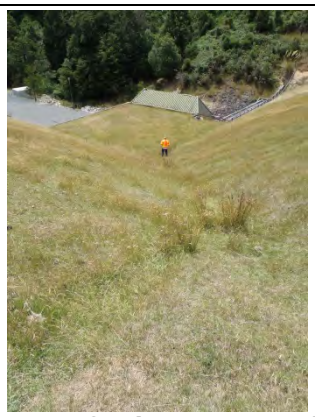

Photo 10: Left Abutment Groin with Scattered Reeds

Photo 11: Reeds at Corner of Toe Buttress Bermand and Left Abutment Groin

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Photo 13: Right Abutment Groin and Culvert area Pipeworks

Photo 14: Right Abutment Groin and Overview of Slip Area

Photo 15: Right Abutment Slip Area

Photo 16: Right Abutment Slip Area

Photo 17: Culvert Bay Joint

Photo 18: Culvert Leakage and Bacterial Growth

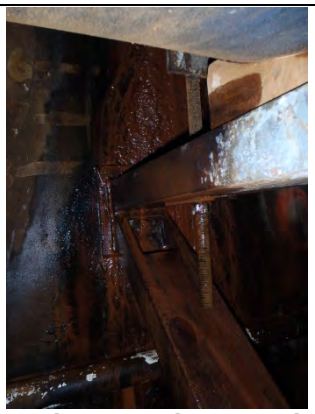

Photo 19 Culvert Leakage and Bacterial Growth on Pipe Bracket

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APPENDIX D Maitai Dam, Hydraulic Piezometer De-airing (Tonkin & Taylor 2014)

REPORT

Nelson City Council

Maitai Dam
Hydraulic Piezometer De-airing

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

REPORT

Nelson City Council

Maitai Dam
Hydraulic Piezometer De-airing

Report prepared for:

NELSON CITY COUNCIL

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

NELSON CITY COUNCIL	1 copy
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April 2014

T&T Ref: 24480.73

Table of contents

Executive summary

Tonkin & Taylor Ltd undertook instrument maintenance from 7 to 9 October 2013. Works were undertaken to assess the condition of the hydraulic piezometers and carry out de-airing of the instruments.

Initial works consisted of removing non-standard fittings on the de-airing board. Board cylinders were drained and refilled with de-aired water that was treated with bacterial retardant. The operation of the board was checked and no leaks were found.

De-airing of all the hydraulic piezometers was carried out.

The non-standard Swagelock fitting was left taped to the de-airing board.

The works carried out on site highlighted the following comments:

- The installation is well maintained and in a dry environment
- All equipment and fittings were in good order
- Piezometers P5, P7, P11, P19, P26, and P27 required two attempts at de-airing
- Piezometers P24 and P25 required three attempts at de-airing
- All gauges were checked across their full operating range to check needle operation and determined to be reading within acceptable tolerances
- Gauges P7 and P8 were both reading outside gauge limits and should be investigated as part of the upcoming Comprehensive Safety Review (CSR).

1 Introduction and background

Tonkin & Taylor Ltd (T&T) undertook an inspection and condition assessment of the hydraulic piezometers at Maitai Dam from 7 to 9 October 2013. The results of that assessment, along with recommendations for further works, are presented in this report.

The hydraulic piezometers provide a critical component of dam safety monitoring by enabling measurement of water pressures at particular locations within the embankment. Air can enter a piezometer system and because it is compressible, air will affect the readings. How much it can affect the results can be a function of how much air is present, the size of the pressures being measured and the sensitivity of the system. Regular de-airing is required to ensure the pressure readings remain representative. The flushing process that is part of de-airing also assists to prevent the build-up of organic matter in the system (fungus etc.) which, if unchecked, can lead to blockages in the system and therefore false readings. Additionally, periodic checks of gauge accuracy are recommended to ensure readings are within acceptable tolerances.

The piezometer de-airing and assessment has been undertaken as part of routine maintenance.

2 Scope of works

The scope of work was presented in T&T's proposal dated 13 September 2013 (T&T ref 24480.73) and is summarised here for ease of reference.

The works undertaken were:

- Site work from 7 to 9 October 2013 by T&T employee Liam Arundel, including:
  - Assessment of the condition of the hydraulic piezometer board
  - De-airing of the hydraulic piezometer leads and flushing
  - Assessment of the condition of the de-airing equipment on site
- Preparation of this short summary report on the works outlined above.

3 Procedures

The inspection and maintenance component was undertaken by Liam Arundel of T&T, an experienced technician with many years' experience in reading and servicing soil instruments in New Zealand.

Inspection, calibration checks and de-airing of the manifold and hydraulic piezometers was undertaken in accordance with manufacturer's instructions provided by Soil Instruments Ltd and titled *"Hydraulic Piezometer Users Manual".*

4 Condition assessment and maintenance undertaken

4.1 Hydraulic piezometer manifold

Initial inspection of the board indicated that the installation is well maintained and in a dry environment. All equipment and fittings were in good order.

Non-standard fittings were attached to the de-airing board. These were removed and the unit charged with de-aired water. The non-standard Swagelock fitting was not re-fitted but left taped to the board.

No leaks were found during operation of the board.

4.2 Hydraulic piezometers

4.2.1 Piezometer gauges

As part of the instrument maintenance, valves and gauges connected to all twenty seven piezometers were lubricated prior to de-airing of the hydraulic piezometers.

Gauge needle checks were undertaken using the testing footpump by taking the gauges through full operating range (0 to 30 m H_20) to test the gauge response and ensure the needles weren't sticking. The results of this check are tabulated in Appendix A.

It is important to note that the check undertaken is not a measure of calibration of the gauges.

All gauges responded well through the operating range and no needle "sticking" was reported.

The gauges are identified as "class 1 mechanical gauge" based on a 1996 calibration undertaken on the instruments. This level has a very tight specification through the mid-range of the gauge. Based on this standard several of the gauges (P1, P2, P8, P10, P23 and P27) would be considered to be operating outside the specification. However, we note that the markers on the gauges are in 1 m H_20 (~10 kPa) increments and we therefore expect that visual reading of the gauges could be made only to $\pm$ ~0.5 m H_20 (~5kPa) with variance based on angle of reading and "rounding".

Therefore we consider all gauges to be reading within acceptable tolerances for this installation.

Notwithstanding this, gauges for P7 and P8 were reading beyond the limits of the gauge (see photo below taken in April 2013).

Figure 1 - Photograph of gauges showing P7 below the lower bound of the gauge and P8 close to the upper limit.

We recommend that the upcoming CSR consider whether the pressures measured by P7 and P8 are reflective of the expected pressures at these locations. The gauges themselves appear to be fully functional (see above) and options to make these gauges more useful may include replacement with gauges more appropriate for the expected operating range of the respective piezometers (i.e. extending lower than -5 m H_20 for P7, and a -5 to 40 m H_20 range for P8 - similar to that used for P10).

4.2.2 Piezometer tubes

The detailed hydraulic piezometer de-airing results are included in Appendix B. Table 1 provides a summary of de-airing results for ease of reference.

It is noted that regular de-airing is required to ensure pressure readings are representative. However, it is common for all air not to be removed from the system. This does not render the instrument un-useable and the data produced can still be analysed, but careful consideration should be given when doing so.

Generally, most of the hydraulic piezometers were able to be adequately de-aired.

P11 had minor inconsistent gauge readings with small fluctuations noted during de-airing. These were considered to be minor and within tolerances – no specific actions are recommended.

Piezometers P5, P7, P11, P19, P26, and P27 required two attempts at de-airing.

Piezometers P24 and P25 required three attempts at de-airing. Following the third attempt both piezometers recorded occasional minor air returns. The 2013 Annual Inspection Report (T&T ref 24480.7) noted no unusual data trends or exceedances in instruments P24 and P25. While complete de-airing was not able to be achieved on these instruments, it is still considered that they read within acceptable tolerances and should still continue to be monitored.

Table 1: Hydraulic piezometer inspection and de-airing result summary

Piezometer	Initial Reading (gauge)			Final Reading (gauge)			Comments
	Left	Right	Both	Left	Right	Both	
P1	29.2	29.6	29.5	29.8	29.8	29.8	Nothing seen.
P2	27.8	27.8	27.8	28.0	28.0	28.0	Nothing seen.
P3	17.4	17.3	17.4	17.2	17.2	17.2	Nothing seen.
P4	-0.2	-0.1	-0.1	0	0	0	Water loss.
P5	-0.5	-0.5	-0.9	-5.4	-5.4	-5.4	De-air twice. Air at 200 ml return.
P6	-1.4	-1.3	-1.4	-1.4	-1.4	-1.4	Water loss.
P7	-6.0	-5.5	-6.0	-5.8	-5.8	-5.8	De-aired twice. Air at 200 ml. Water loss.
P8	29.9	29.9	29.9	30.1	30.1	30.1	Excess return.
P9	12.7	12.7	12.7	12.2	12.2	12.2	Nothing seen.
P10	30.0	30.0	30.0	30.0	30.0	30.0	Excess return.
P11	19.2	21.9	21.9	21.3	21.2	21.3	De-aired twice. Minor reading fluctuation.
P12	12.8	12.9	12.8	6.1	6.1	6.1	Sporadic air to 1600 ml. Return.
P13	12.8	12.6	12.8	12.8	12.8	12.8	Air 500 ml to 900 ml. Return.
P14	10.1	10.1	10.1	10.8	10.8	10.8	Nothing seen.
P15	4.6	5.9	4.8	4.6	4.6	4.6	Air 500 ml. To 800 ml. Return. Water loss
P16	26.6	26.6	26.6	26.8	26.8	26.8	Excess return.
P17	17.4	17.4	17.4	17.5	17.5	17.5	Nothing seen.
P18	10.5	10.5	10.5	10.5	10.5	10.5	Nothing seen.
P19	12.9	12.9	12.9	12.5	12.5	12.5	De-air twice. Sporadic air 100 ml to 1300 ml return.
P20	6.4	6.4	6.4	6.4	6.4	6.4	Nothing seen.
P21	5.3	5.9	5.1	2.3	2.3	2.3	Air at 400 ml return.
P22	30.3	30.3	30.3	30.3	30.3	30.3	Excess return.
P23	26.4	26.3	26.4	26.3	26.3	26.3	Nothing seen.
P24	28.5	28.4	28.5	27.5	27.3	27.4	De-aired three times. Still minor air.
P25	18.1	0.2	8.8	21.3	21.3	21.3	De-aired three times Still minor air.
P26	28.3	28.0	28.2	28.1	28.1	28.1	De-aired twice. Air diminishing to 7500 ml.
P27	26.8	26.7	26.7	26.5	26.5	26.5	De-aired twice. Air to 2100 ml return.

Shaded instruments were unable to be fully de-aired, but are still expected to operate within acceptable tolerances.

5 Summary of recommendations

The only recommendation made during our inspection and assessment is for the upcoming CSR to consider whether the pressures measured by P7 and P8 are reflective of the expected pressures at these locations. This review should also consider options to make these gauges more useful such as replacement with gauges more appropriate for the expected operating range of the respective piezometers (refer Section 4.2.1).

Results of this review should be discussed along with the data review in the 2014 annual inspection.

6 Applicability

This report has been prepared for the benefit of Nelson City Council with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:	Reviewed by:
.....	.....
Paul McCallum	David Bouma
Geotechnical & Water Resources Engineer	Project Director

PDM

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Appendix A: Gauge needle check results

Hydraulic piezometer gauge needle check

Matai Dam
Job no. 24480.73

	Gauge type	Foot pump reference gauge Incremental readings (m. H^2O)						Remarks
		Atmos	7.0	17.5	25.5	29.0	Normal Reading	
P1	Budenberg 150mm. 40m to -0.5m	-0.5	7.4	17.6	25.9	28.9	29.8	
P2	Budenberg 150mm. 30m to -5m	-0.5	7.3	17.6	25.9	29.0	28.0	
P3	Budenberg 150mm. 30m to -5m	-0.6	7.2	17.5	25.8	28.9	17.2	
P4	Budenberg 150mm. 30m to -5m	-0.8	7.1	17.6	25.8	28.9	0.0	
P5	Budenberg 150mm. 30m to -5m	-0.9	7.1	17.2	25.5	28.8	5.4	
P6	Budenberg 150mm. 30m to -5m	-0.8	7.0	17.2	25.6	28.8	-1.4	
P7	Budenberg 150mm. 30m to -5m	-0.9	7.1	17.4	25.5	28.7	-5.8	Outside normal gauge range
P8	Budenberg 150mm. 30m to -5m	-0.2	7.6	17.9	26.2	29.3	30.1	Outside normal gauge range
P9	Budenberg 150mm. 30m to -5m	-0.8	7.3	17.5	25.7	28.8	12.2	
P10	Budenberg 150mm. 40m to -5m.	-0.8	7.5	17.6	26.0	29.0	30.0	
P11	Budenberg 150mm. 30m to -5m.	-0.8	7.2	17.4	25.6	28.7	21.2	
P12	Budenberg 150mm. 30m to -5m.	-0.7	7.1	17.5	25.9	28.9	6.1	
P13	Budenberg 150mm. 30m to -5m.	-0.7	7.1	17.5	25.8	28.9	12.8	
P14	Budenberg 150mm. 30m to -5m.	-0.8	7.2	17.4	25.7	28.9	10.8	
P15	Budenberg 150mm. 30m to -5m.	-0.8	7.1	17.5	25.7	28.7	4.6	

Read by.....L.A.....
Date.....8.10.13...

Checked by.....
Date.....

Hydraulic piezometer gauge needle check

Matai Dam
Job no. 24480.73

	Gauge type	Foot pump reference gauge Incremental readings (m. H^2O)						Remarks
		Atmos	7.0	17.5	25.5	29.0	Normal Reading	
P16	Budenberg 150mm 30m to -5m.	-0.6	7.1	17.5	25.8	28.8	26.8	
P17	Budenberg 150mm 30m to -5m.	-0.7	7.0	17.4	25.8	28.8	17.5	
P18	Budenberg 150mm 30m to -5m.	-0.7	7.1	17.5	25.8	28.8	10.5	
P19	Budenberg 150mm 30m to -5m.	-0.9	7.1	17.3	25.4	28.6	12.5	
P20	Budenberg 150mm 30m to -5m.	-0.6	7.1	17.4	25.8	28.8	6.4	
P21	Budenberg 150mm 30m to -5m.	-0.8	7.2	17.6	25.8	28.8	2.3	
P22	Budenberg 150mm 40m to -5m.	-0.8	7.1	17.5	25.9	28.9	30.3	
P23	Budenberg 150mm 30m to -5m.	-0.7	7.3	17.6	25.8	29.0	26.3	
P24	Budenberg 150mm 30m to -5m.	-0.7	7.2	17.6	25.8	28.9	28.6	
P25	Budenberg 150mm 30m to -5m.	-0.7	7.2	17.5	25.9	29.0	21.1	
P26	Budenberg 150mm 30m to -5m.	-0.6	7.2	17.5	25.8	29.0	28.1	
P27	Budenberg 150mm 30m to -5m.	-0.6	7.3	17.5	25.9	29.0	26.5	

Read by.....L.A.....
Date.....8.10.13...

Checked by.....
Date.....

Appendix B: De-airing results

Job no. 24480.73 Matai Dam 7.10.13

Piezometer sheet

P 1

m. H^2O	L	R	Both
Initial	29.2	29.6	29.5
Final	29.8	29.8	29.8

Tube vol. 2400ml. Tip ht. (-)7.7m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	200	0	12.12	12.33	21min.	126
Vol.water 1	10000	550				
Vol. water 2	7000	4250				
Used ml.	*2650	3700				

Notes

* 12% error on cylinder marked volumes.

No air seen

Excess water return

Job no. 24480.73 | Matai Dam | 7.10.13

Piezometer sheet

P 2

m. H^2O	L	R	Both
Initial	27.8	27.8	27.8
Final	28.0	28.0	28.0

Tube vol. 1600ml. Tip ht. (-)13.7m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	5	12.36	1.10	34min.	51
Vol.water 1	7000	4250				
Vol. water 2	5000	6300				
Used ml.	*1750	2050				

Notes

* 12% error on cylinder marked volumes.

No air seen

Job no. 24480.73 | Matai Dam | 7.10.13

Piezometer sheet

P 3

m. H^2O	L	R	Both
Initial	17.3	17.4	17.3
Final	17.2	17.2	17.2

Tube vol. 1450ml. Tip ht. (-) 0.5m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	0	1.14	1.44	30min.	58
Vol.water 1	5000	6300				
Vol. water 2	3000	8200				
Used ml.	1750	1900				

Notes

* 12% error on cylinder marked volumes.

No air seen

Job no. 24480.73 | Matai Dam | 7.10.13

Piezometer sheet

P 4

m. H^2O	L	R	Both
Initial	-0.2	-0.1	-0.1
Final	0.0	0.0	0.0

Tube vol. 1000ml. Tip ht. (-) 0.5m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	350	50	2.10	2.30	20min	119
Vol.water 1	10000	1800				
Vol. water 2	7300	3200				
Used ml.	2375	1400				

Notes

* 12% error on cylinder marked volumes.

High pressure to get return flow

Water loss

No air seen

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Piezometer sheet

P 5

m. H^2O	L	R	Both
Initial	-0.5	-0.5	-0.9
Final	-5.4	-5.4	-5.4

Tube vol. 960ml Tip ht. (-) 5.4m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	100	0	2.32	2.58	26min.	61
Vol.water 1	7000	3200				
Vol. water 2	5200	4600				
Used ml.	*1590	1400				

Second attempt,9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	100	0	9.06	9.25	19min.	73
Vol.water 1	7500	3400				
Vol. water 2	5900	4150				
Used ml.	1400	750				

Notes

* 12% error on cylinder marked volumes.

First; Lots of air 200ml to 400ml in return cylinder

Second; No air seen .Water loss.

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Piezometer sheet

P 6

m. H^2O	L	R	Both
Initial	-1.4	-1.3	-1.4
Final	-1.4	-1.4	-1.4

Tube vol. 370ml. Tip ht. (-) 1.4m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	2.57	3.15	18min.	73
Vol.water 1	5200	4600				
Vol. water 2	3700	4900				
Used ml.	*1325	300				

Notes

* 12% error on cylinder marked volumes.

No air seen.
Water loss.

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Piezometer sheet

P 7

m. H^2O	L	R	Both
Initial	-5.9	-5.7	-5.8
Final	-5.8	-5.8	-5.8

Tube vol. 1170ml. Tip ht. (-) 7.8m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	50	0	3.15	3.35	20min.	66
Vol.water 1	3700	4900				
Vol. water 2	2200	5500				
Used ml.	*1325	600				

Second attempt,9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	50	0	9.26	9.40	14 min.	57
Vol.water 1	5900	4150				
Vol. water 2	5000	4400				
Used ml.	*800	250				

Notes

* 12% error on cylinder marked volumes.

First; Minor air at 200ml in return cylinder.
Water loss

Second; No air seen.
Water loss.

Job no. 24480.73 Matai Dam 7.10.13

Piezometer sheet

P 8

m. H^2O	L	R	Both
Initial	29.9	29.9	29.9
Final	30.1	30.1	30.1

Tube vol. 1625ml Tip ht. 13.4m

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	40	3.35	4.10	35min.	45
Vol.water 1	2200	5500				
Vol. water 2	400	8200				
Used ml.	*1590	2700				

Notes

* 12% error on cylinder marked volumes.

No air seen .

Excess water return.

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Piezometer sheet

P 9

m. H^2O	L	R	Both
Initial	12.7	12.7	12.7
Final	12.2	12.2	12.2

Tube vol. 1050ml. Tip ht. 12.7m

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	4.30	4.52	22min.	63
Vol.water 1	10000	750				
Vol. water 2	8400	2150				
Used ml.	*1400	1400				

Notes

* 12% error on cylinder marked volumes.

No air seen .

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Piezometer sheet

P 10

m. H^2O	L	R	Both
Initial	30.0	30.0	30.0
Final	30.0	30.0	30.0

Tube vol. 1680ml. Tip ht. 13m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	350	50	4.54	5.40	46min.	45
Vol.water 1	8400	2150				
Vol. water 2	6000	5550				
Used ml.	*2100	3400				

Notes

* 12% error on cylinder marked volumes.

No air seen .

Excess water return.

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Piezometer sheet

P 11

m. H^2O	L	R	Both
Initial	19.2	21.9	21.9
Final	21.2	21.3	21.2

Tube vol. 1930ml. Tip ht. 14.5m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	25	5.42	6.12	30min.	58
Vol.water 1	6000	5550				
Vol. water 2	4000	7650				
Used ml.	*1750	2100				

Second attempt;9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	0	8.15	9.00	45min.	46
Vol.water 1	9900	400				
Vol. water 2	7500	3400				
Used ml.	*2100	3000				

Notes

* 12% error on cylinder marked volumes.

First;

No air seen.

Gauge readings inconsistent.

Second; No air seen .

Water loss.

Still minor discrepancy in face readings.

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Piezometer sheet

P 12

m. H^2O	L	R	Both
Initial	12.8	12.9	12.8
Final	6.1	6.1	6.1

Tube vol. 1190ml. Tip ht. 14.5m

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	6.15	6.55	40min.	33
Vol.water 1	4000	6900				
Vol. water 2	2500	7600				
Used ml.	*1325	700				

Second attempt;9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	200	0	9.41	10.15	34min.	44
Vol.water 1	5000	4400				
Vol. water 2	3300	5600				
Used ml.	*1500	1200				

Notes

* 12% error on cylinder marked volumes.

First; Air at 400ml. return
600ml. return
700ml. return Water loss

Second; Minor air : 50ml. return
300ml return 400ml return
Sporadic to 900ml return then stopped.
Reduced water loss.

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Piezometer sheet

P 13

m. H^2O	L	R	Both
Initial	12.8	12.6	12.8
Final	12.8	12.8	12.8

Tube vol. 1000ml Tip ht. 12.8m

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	8.00	8.30	30min	45
Vol.water 1	10000	450				
Vol. water 2	8000	1800				
Used ml.	*1750	1350				

Notes

* 12% error on cylinder marked volumes.

Air at 500 to 600ml return.
air at 800 to 900ml. return
Water loss

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Piezometer sheet

P 14

m. H^2O	L	R	Both
Initial	10.1	10.1	10.1
Final	10.8	10.8	10.8

Tube vol. 800ml. Tip ht. 10m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	8.30	8.50	20min.	66
Vol.water 1	8000	1800				
Vol. water 2	6500	3100				
Used ml.	*1325	1300				

Notes

* 12% error on cylinder marked volumes.

No air seen

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Piezometer sheet

P 15

m. H^2O	L	R	Both
Initial	4.6	5.9	4.8
Final	4.6	4.6	4.6

Tube vol. 700ml. Tip ht. 4.9m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	50	0	8.52	9.10	18min.	73
Vol.water 1	6500	3100				
Vol. water 2	5000	3900				
Used ml.	*1325	800				

Notes

* 12% error on cylinder marked volumes.

Air at 500ml. return
Air at 800ml. return
Water loss.

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Piezometer sheet

P 16

m. H^2O	L	R	Both
Initial	26.6	26.6	26.6
Final	26.8	26.8	26.8

Tube vol. 1920ml. Tip ht. 12.1m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	0	9.10	10.10	60min.	32
Vol.water 1	5000	3900				
Vol. water 2	2800	8100				
Used ml.	*1925	4200				

Notes

* 12% error on cylinder marked volumes.

No air seen

Excess water rteurn.

Job no. 24480.73 Matai Dam 8.10.13

Piezometer sheet

P 17

m. H^2O	L	R	Both
Initial	17.4	17.4	17.4
Final	17.5	17.5	17.5

Tube vol. 1780ml Tip ht. 12.1m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	10.12	10.50	38min.	58
Vol.water 1	2800	6000				
Vol. water 2	500	8300				
Used ml.	*2200	2300				

Notes * 12% error on cylinder marked volumes.

No air seen

Job no. 24480.73 Matai Dam 8.10.13

Piezometer sheet

P 18

m. H^2O	L	R	Both
Initial	10.5	10.5	10.5
Final	10.5	10.5	10.5

Tube vol. 1470ml. Tip ht. 7.4m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	11.05	11.35	30min.	58
Vol.water 1	10000	650				
Vol. water 2	8000	1900				
Used ml.	*1750	1250				

Notes

* 12% error on cylinder marked volumes.

No air seen

Water loss.

Job no. 24480.73 Matai Dam 8.10.13

Piezometer sheet

P 19

m. H^2O	L	R	Both
Initial	12.9	12.9	12.9
Final	12.5	12.5	12.5

Tube vol. 1120ml. Tip ht. 12.8m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	11.35	11.58	23min.	58
Vol.water 1	8000	1900				
Vol. water 2	6500	3300				
Used ml.	*1325	1400				

Second attempt;9.10 13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	150	0	10.17	10.30	13min.	35
Vol.water 1	3300	5600				
Vol. water 2	2800	6000				
Used ml.	*450	400				

Notes

* 12% error on cylinder marked volumes.

First;

Sporadic air from 1000ml return.

Second; No air seen

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Piezometer sheet

P 20

m. H^2O	L	R	Both
Initial	6.4	6.4	6.4
Final	6.4	6.4	6.4

Tube vol. 540ml. Tip ht. 5.9m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	100	0	12.00	12.15	15min.	65
Vol.water 1	6500	3300				
Vol. water 2	5400	4000				
Used ml.	*975	700				

Notes

* 12% error on cylinder marked volumes.

No air seen

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Piezometer sheet

P 21

m. H^2O	L	R	Both
Initial	5.3	5.9	5.1
Final	2.3	2.3	2.3

Tube vol. 410ml. Tip ht. 2.9m

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	50	0	12.15	12.22	7min.	114
Vol.water 1	5400	4000				
Vol. water 2	4500	4450				
Used ml.	*800	450				

Notes

* 12% error on cylinder marked volumes.

Air at 400ml return

Water loss

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Piezometer sheet

P 22

m. H^2O	L	R	Both
Initial	30.3	30.3	30.3
Final	30.3	30.3	30.3

Tube vol. 1250ml. Tip ht. 25.8m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	350	0	12.22	12.50	28min.	54
Vol.water 1	4500	4450				
Vol. water 2	2800	6900				
Used ml.	*1500	2450				

Notes

* 12% error on cylinder marked volumes.

No air seen

Excess return

Job no. 24480.73 Matai Dam 8.10.13

Piezometer sheet

P 23

m. H^2O	L	R	Both
Initial	26.4	26.3	26.4
Final	26.3	26.3	26.3

Tube vol. 980ml. Tip ht. 26m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	12.52	1.12	20min.	70
Vol.water 1	2800	6900				
Vol. water 2	1200	9000				
Used ml.	*1400	2100				

Notes

* 12% error on cylinder marked volumes.

No air seen

Excess return

Job no. 24480.73 Matai Dam 8.10.13

Piezometer sheet

P 24

m. H^2O	L	R	Both
Initial	28.5	28.4	28.5
Final	27.5	27.3	27.4

Tube vol. 1790ml. Tip ht. 25.8m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	325	40	1.30	2.38	68min.	32
Vol.water 1	10000	1000				
Vol. water 2	7500	4700				
Used ml.	*2200	3700				

Second attempt;9.10 13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	325	40	11.51	12.41	50min.	39
Vol.water 1	8600	1900				
Vol. water 2	6400	4950				
Used ml.	*1950	3050				

Third attempt;9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	325	40	2.15	2.55	40min.	58
Vol.water 1	3300	5700				
Vol. water 2	700	8800				
Used ml.	*2300	3100				

Notes

* 12% error on cylinder marked volumes.

First;

Air at 500ml. return.Sporadic 1200ml to 1800ml.ret.

Then occasional with bursts at 1800ml.and 2600ml. return.

Second; minor air at 200ml. ret. Sporadic to end.

Third. Still minor air at increasing intervals.

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Piezometer sheet

P 25

m. H^2O	L	R	Both
Initial	18.1	0.2	8.8
Final	21.3	21.3	21.3

Tube vol. 1550ml. Tip ht. 25.9m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	2.39	3.42	63min.	43
Vol.water 1	7500	4700				
Vol. water 2	4400	7000				
Used ml.	*2725	2300				

Second attempt;9.10 13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	10.30	11.01	31min.	60
Vol.water 1	2800	6000				
Vol. water 2	700	7900				
Used ml.	*1850	1900				

Third attempt;9.10 13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	11.24	12.00	36min.	39
Vol.water 1	10200	600				
Vol. water 2	8600	1900				
Used ml.	*1400	1300				

Notes

* 12% error on cylinder marked volumes.

First; Air at 400ml. return 500ml,600ml.Sporadic to 1500ml. Return. then occasional to 2300 return.

Second; Air at 300ml and 600ml return then sporadic to 1900ml.

Third; still occasional minor air, at increasing intervals.

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Piezometer sheet

P 26

m. H^2O	L	R	Both
Initial	28.3	28.0	28.2
Final	28.1	28.1	28.1

Tube vol. 1640ml. Tip ht. 25.5m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	40	3.43	4.43	60min.	32
Vol.water 1	4400	6000				
Vol. water 2	2300	9800				
Used ml.	*1925	3800				

Second attempt;9.10.13

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	300	40	12.44	1.34	40min.	64
Vol.water 1	6400	4950				
Vol. water 2	3500	8900				
Used ml.	*2550	3950				

Notes

* 12% error on cylinder marked volumes.

First;

Air at 1200ml.return.

Air at 2400ml. return.

Sporadic,minor, to 3200ml.

Second; minor air at 200ml,500ml and 700ml. Then small bursts at increasing intervals.

Nothing seen from 3700ml to 3950ml return.

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Piezometer sheet

P 27

m. H^2O	L	R	Both
Initial	26.8	26.7	26.7
Final	26.7	26.7	26.7

Tube vol. 1400ml. Tip ht. 25.5m.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	4.45	5.25	40min.	44
Vol.water 1	2300	6000				
Vol. water 2	300	8400				
Used ml.	*1750	2400				

Second attempt 9.10.13.

	In kPa	Vac.kPa	Time st.	Time fin.	Elapsed	Rate ml/min
Pressure	250	0	1.36	1.45	9min.	58
Vol.water 1	3500	8900				
Vol. water 2	2900	8400				
Used ml.	*525	500				

Notes

* 12% error on cylinder marked volumes.

First; Air at 600ml,1000ml,1300ml,1400ml. Return
Occasional air to 2100ml.

Second; Minor air at 100ml. then clear to 500ml.

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