Lower Nihotupu Dam

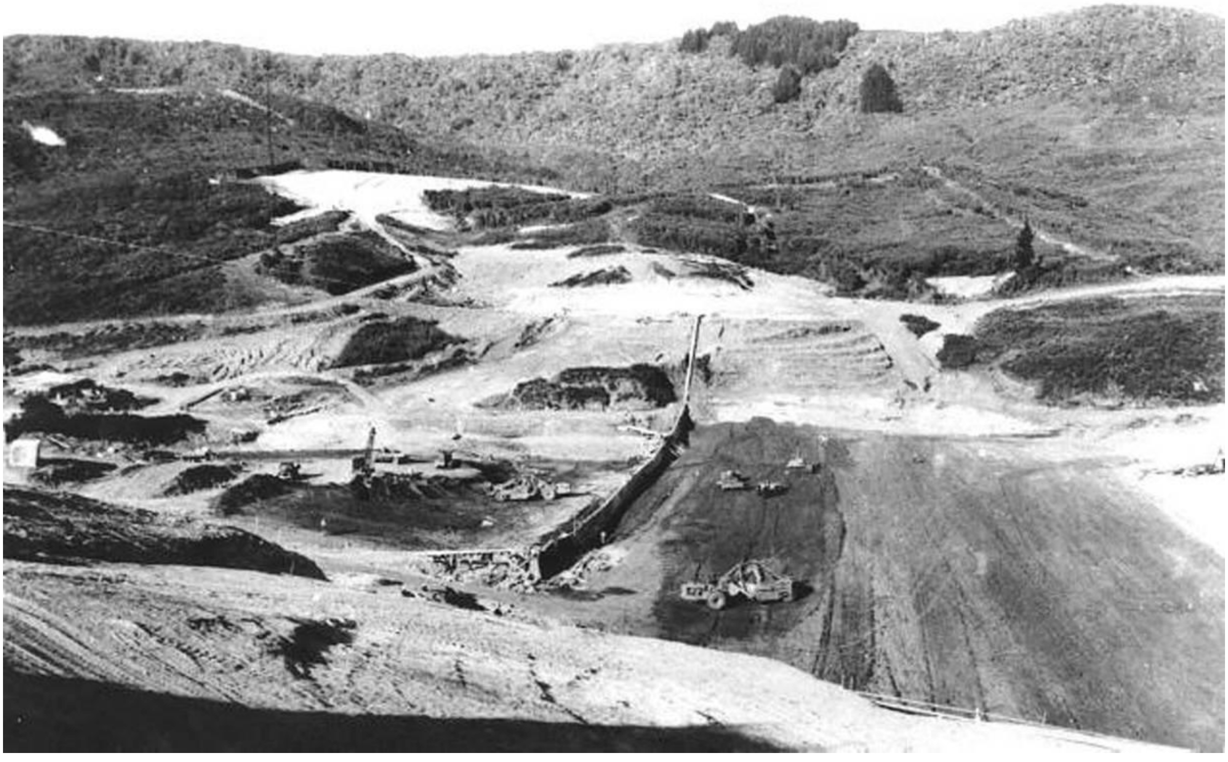
Background and Summary Details

Watercare
An Auckland Council Organisation

Aerial View of the Lower Nihotupu Dam

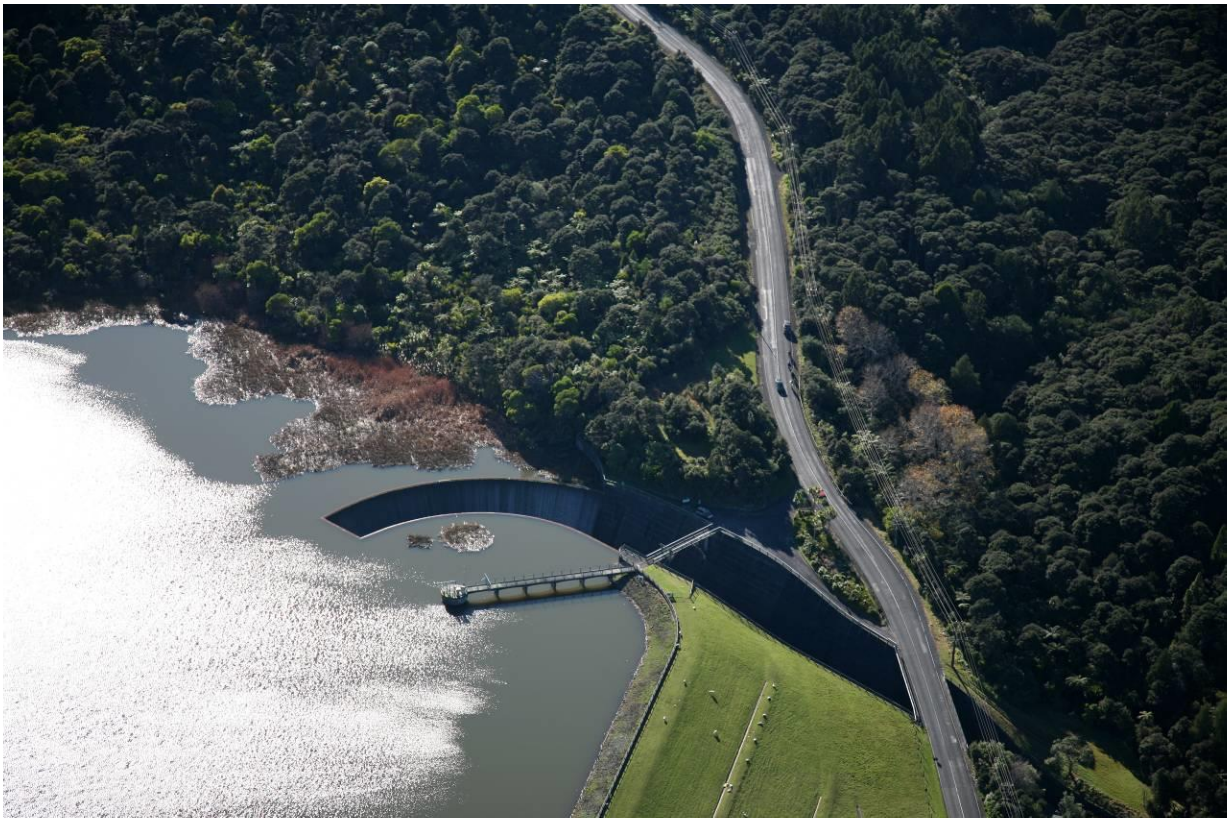
Lower Nihotupu Dam Spillway in Operation

View out to the Manukau harbour from over the lake



Looking south west along the line of the dam with the cut off wall already constructed

View of embankment, spillway and pump station



Valve tower and side trench spillway

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Version history		
Original version 1.0	April 2005	W McQuarrie
Version 1.1	September 2018	W McQuarrie

Description of the Dam and Appurtenant Structures

The Lower Nihotupu Dam, constructed between 1945 and 1948, is located 13 km southwest of Titirangi and dams the lower regions of the Nihotupu Stream. The dam is owned and operated by Watercare Services Ltd and is one of thirteen Large dams used for impounding water for Auckland's bulk water supply. The embankment is notable for being the first "scientifically controlled" rolled fill dam structure in New Zealand.

The dam and Headworks comprise the following main elements:

(a) The catchment area is some 1,300 ha and the reservoir area at top water level is approximately 71.5 ha. The dam impounds a reservoir of 4,805,000 cubic metres storage.

(b) A 24.7 m high zoned earthfill dam with a crest length of 381 m and containing a total of 359,100 cubic metres of fill. The embankment has a central clay core with transitional and shoulder zones constructed from rubble. Seepage collected by the downstream filter toe drain discharges below the spillway. Piezometers monitor pore pressures in the embankment fill and foundation of the dam, and settlement gauges monitor surface movements.

(c) A concrete cut-off wall was installed at the base of the dam, along the centreline, extending at least two meters into hard bedrock and upwards into the fill to a height of 10% of the embankment height. In addition, a grout curtain was drilled and injected into the bedrock below the cut-off wall.

(d) During construction the core zone was widened from the original design dimensions. Bore logs from recent drilling show that the boundary between core and shoulder zones varies widely. Recent trenching has shown that the core extends up to within 1 metre of the dam crest, with the remaining material being well compacted silts, clays and gravels.

(e) Drawings show the filter toe zone draining into the old streambed downstream of the dam, which had been backfilled with gravels. This extends down to manhole 4 on the seaward side of the main road. The seepage from the drain is collected in to Manhole 4, and transported via the 900 mm drainage pipe to the drainage path beneath the spillway outlet. The old streambed area has been filled in right down to the sea, from manhole 4. The drainage water is carried in a 900 mm diameter pipe whose invert discharges at about low water level. A system of surface drains and pipes is installed in the berms on the downstream side of the dam. This drainage system was upgraded in the early 1980's.

(f) Seepage is measured at drainpipes discharging into refurbished (2001/2) manholes through “V” notch weirs beside the 'drainage sump" at the downstream toe. The storm water drain pipes collecting runoff from surface drains on the berms are not monitored. The seepage past manhole 4 is not measured because of the difficulty of distinguishing between seepage water, and tidal generated flows. King tides have on occasion backed up the supply tunnel to the base of the valve tower.

(g) The valve tower is a 5.13 m diameter reinforced concrete circular structure which contains water intakes, valves and screens at various levels and two low level valves for scour and emergency dewatering. The internal scour valve discharges into the supply tunnel, and exits into the spillway channel. A standpipe within the tower, and connecting to the lake, houses the float system for the lake level recorder. The valve tower was strengthened and anchored down in 2010.

(h) A 34 m long reinforced concrete bridge provides access to the valve tower from the left abutment.

(i) An open spillway weir discharges through a spillway channel at the left abutment. The spillway discharges into an estuary known as Muddy Creek, which is an arm of the Manukau Harbour.

(j) The spillway is a free overflow spillway discharging through a spillway channel constructed in a cut trench on the left abutment of the dam. The spillway channel is lined with reinforced concrete facing of varying thickness. The walls of the channel are a skin on the cut faces, without anchorage into the rock. KRTA in their 1992-93 safety evaluation (KRTA, 1993) investigated the spillway walls in detail. Damwatch Services reviewed the seismic capacity and uplift capacity of the spillway in 2008.

(k) The spillway walls extend higher than original ground level and the top of the earth dam abuts the channel walls. The dam core had to be placed against an overhanging wall with counterfort supports. A film of the dam's construction indicates care being taken in trying to compact fill in these restricted areas.

(l) A pedestrian and road bridge, 34m in length, crosses the spillway channel.

(m) A diversion tunnel became the 3.35 m diameter supply tunnel. It is 130 m long and leads from the base of the valve tower to the spillway channel south of the main road. It contains a 1270 mm diameter supply pipeline and a 760 mm diameter scour pipeline.

(n) The dam has two operational free discharge valves for rapid draw down of the dam. One is directly connected to the scour valve in the valve tower and discharges into the supply tunnel. The second is located just outside the supply tunnel, and discharges from the supply main into the spillway channel.

(o) A pump station immediately downstream of the dam, and located on the south side of Huia Road, boosts the mains pressure up to the Huia water treatment plant.

Geology

The dam is located south of the Parau faults and west of the Waiatarua fault. One arm of the Waiatarua fault is inferred to pass close to the left abutment while another arm of the fault passes close to the right abutment. The dam is founded on Waitemata group of interbedded sandstone and mudstone consisting of moderately thin beds of weak to moderately strong sandstone of Andesite, quartz and feldspar grains.

Foundation preparation work included excavating alluvial soils and weathered rock to intact bedrock over a wide width in the central part of the dam. Clean stream gravels were left in place under the upstream and downstream shoulder zones. One larger fault, defined by a wet, plugged shatter zone, required special attention in foundation preparation work.

Lower Nihotupu Dam site geology and structure are relatively complex and thorough investigation and site preparation was undertaken prior to embankment construction leading to a good understanding of the site geology (Safety Evaluation Study Stage 1, 1992, refer Appendix G). An adaptation of Appendix G, rewritten with dimensions metricated, is attached as **Appendix A**.

The dam is underlain by Waitemata Group sedimentary rocks, comprising tight mudstones to medium grained sandstones. A band of semi-volcanic material (Parnell Grit) interbedded with the Waitemata Group series, was found in the vicinity of (and to the right of) the Nihotupu Stream bed.

At the dam site the strata are curved into an anticline with a north-south axis. Numerous minor steeply dipping faults and closely jointed and shattered zones were found in the dam foundation. The most significant fault in the foundation, on the western side of the valley, is associated with a 0.5m wide gouge zone.

The only known area of significant instability is the area of surface movement in the Nihotupu valley at the head of the reservoir, through which the Huia supply pipeline passes. The ground movements appear to be slow. The area is referred to as the 40 acre Slip, and lays between the head of the reservoir, and a cliff located just below the Arataki Parks Centre.

The 40 Acre Slip landslide study area consists of an older landslide complex that was most likely formed by a retrogressive mechanism starting at the toe and working upward. Evidence indicates that the larger landslide complex has always been a slow moving landslide and has mostly stabilised in recent times. However, one active lobe within the larger slide mass has been noted since the early 1900’s. The location of historical slide movement has been documented and is currently being observed visually and with instrumentation measurements.

The existing landslide is a remnant of a much larger original slide mass.

The landslide complex is classified as a ‘slow moving translational slide.’ Based on case history information and the characteristics of the landslide geometry and the weak materials, it is considered to have a very low likelihood of rapid failure. It is most likely to move at creep rates, with different parts of the slide mobilising at different times. Currently the larger landslide complex is in a relatively stable condition.
Some surface drainage improvements were undertaken at approximately 1988. These had the effect of slowing down the movement.

While reservoir blockage by a landslide dam cannot be eliminated as a potential risk it is considered an extremely low risk. The primary reason for this assessment is the slow creeping progressive nature of the slide, its low slope angle and its low potential for sudden rapid movement of sufficient displacement to block the reservoir.

The Parau fault passes through this zone of slope movement in a NE-SW direction. The poor drainage and seepages in the vicinity could conceivably be associated with this fault zone.

Crest materials

The cover fill (upper fill capping layer) extends to between 0.9m – 1.2m depth below the crest of the dam. It is underlain by dam core material. The cover fill consists mainly of a fine to coarse gravel in a brown silt, clay and sand matrix, with some boulders and cobbles. The fill appears to be well compacted, moist to dry and brittle, with a trace of rootlets and building materials (e.g. wood).

Downstream Fill

The downstream fill was encountered only in TP1, because this was the only pit which could be extended onto the downward slope, due to the presence of underground services elsewhere. The downstream fill consists of a fine to coarse gravelly silt with some clay and minor sand, cobbles and boulders; it was moist, stiff and had a trace of rootlets in the matrix. The downstream fill differs from the cover fill visibly by colour and a smaller volume of larger materials (cobbles and boulders).

Core material

The core material was encountered between 0.9m – 1.2m depth below the crest of the dam i.e. at a shallower depth to that inferred from the results of the 1992 bore investigation. The upper up to 1m thickness of core material encountered in the test pits typically consisted of a clay or silt with significant sand and a trace of gravel. It is mostly brown or grey, stiff to very stiff, moist to wet and plastic, with a trace of rootlets.

Average Permeability Values for Material Types

Material	n	Permeability (m/s)	Comment
Core material	8	1×10^{-9}	k_h
Cover fill	3	2×10^{-8}	Recompacted
Downstream fill	1	1×10^{-9}	Recompacted

Upper Nihotupu dam breach

In the event of a dam breach of the Upper Nihotupu dam, the following consequences will be felt at the Lower Nihotupu dam:

Lower Nihotupu dam	Peak water elevation	Flood arrival time	Depth over spillway	Depth over dam crest
	23.5 mRL	60 minutes	2.3 m	0.05 m

PMF passing through the Lower Nihotupu dam

The following consequences will result from a probable maximum flood at Lower Nihotupu dam:

Lake level scenario	Peak inflow (m^3/s)	Peak outflow (m^3/s)	Peak lake level (m)	Starting lake level (m)
Lower Quartile	611.13	591.4	22.949	21.086
Mean	611.13	591.5	22.949	21.128
Upper Quartile	611.13	592.0	22.950	21.447

The dam will be able to pass the PMF flood with a minimum freeboard of 0.05m. Wind surge and wave activity will be remediated with the wave break structure.

Surveillance and Instrumentation

The dam is subject to Watercare's Dam Safety Management System that generally follows the requirements of the New Zealand Society of Large Dams (NZSOLD) Dam Safety Guidelines 2015. This comprises field observations, instrumentation data, data processing and assessment, with annual and five yearly dam safety audits and inspections.

A Comprehensive Dam Safety Review is undertaken at 5 yearly intervals and the Intermediate Dam Safety Reviews are undertaken annually.

Routine surveillance undertaken by the caretakers follows the procedures in the Headworks Operations Manual and the Lower Nihotupu Dam Procedures Manual.

Non routine inspection is normally undertaken by the Dam Safety Engineer and potentially staff from Damwatch Services Ltd.

The dam was first instrumented with standpipe piezometers in October 1992. In August 2001, vibrating wire instruments were installed to provide real time data and to determine if a more responsive system could be provided. The vibrating wire instruments are connected to the Watercare SCADA system with data collected at hourly intervals and stored in the PI data base. A representative recording is selected for each day and forwarded weekly on a Saturday to Dam Safety Intelligence for checking and archiving on their database.

Manual surveillance records are collected monthly on a Psion data recorder, and forwarded electronically to Dam Safety Intelligence. These are for instruments that do not have an automated surveillance capability (e.g. visual inspections, standpipe piezometers and seepage flows)

Deformation survey monitoring of the dam was commenced in 1991 by the Auckland Regional Council. This has been undertaken by Energy Surveys Ltd since 2004 with the resulting reports forwarded to Watercare and Dam Safety Intelligence.

Seepage monitoring is undertaken at several locations. The assessment of the seepage flows is complicated by the tidal inflows that back up the drainage systems, flooding the weirs. Recordings are not taken at high tides. Additionally, the seepage collected and measured in manhole 1 is often lost on the transmission to the next manhole, as the connecting drain has several connections of subsoil pipe leading back into the toe drain along the eastern (left) portion of the downstream shoulder. At high tides, the water drains back into the toe drain, instead of out through the drains to sea.

The lake level recorder and the local raingauge are monitored continuously on Hydrotel, and the data forwarded to the PI data base, with daily summaries forwarded weekly to Dam Safety

Intelligence. Data placed on the PI database is not verified. The hydrometric data is also forwarded to a Hydstra database where it is verified before archiving. The Hydrometric Analyst attends each lake level recorder and automated raingauge on a monthly routine, collects the data, ensures the calibration of the shaft encoder monitoring the lake levels, and checks the automated total of rainfall against the volume of rainfall in a check gauge. This data is verified, before being entered into the Hydstra database. In the event of any lake level or rainfall data being questioned, the Hydstra data is provided. These two sources of data are stored on site in data loggers.

The Instrument technicians have a logger that will monitor the frequency recordings of the vibrating wire instruments, so that the on-site records can be verified against the data on PI. The dam engineers have a further instrument that will not only record to frequency reading, but also establish the performance of the signal from the vibrating wire instrument. This is implemented annually.

Many of the standpipe and vibrating wire piezometers have a very slow response, due to the impermeable nature of both the core material and the grouted sedimentary rock beneath the dam.

Issues

Issues associated with the Lower Nihotupu Dam include:

- The dam toe can absorb tidal backflows that come back up the drains during King Tide events. During the Spring tide, the tide can back up the supply tunnel to the base of the valve tower. It then has to discharge both tidal water as well as any seepage water.

- The dam has a concrete cut off wall connecting the core to the foundation to act as a seal along the core/foundation contact.

- There is no chimney drain or filter laid against the core of the dam.

- Eight excavated test pits were undertaken along the crest of the Lower Nihotupu Dam in February 2013. There is a clear demarcation between the silty clay core and the overlying gravelly cover materials with the contact occurring at 0.9m to 1.2m below the crest of the dam at the eight test sites.

Failure Mechanisms

A failure modes workshop was held on 10 & 11 March 2010 facilitated by Damwatch Services Limited to establish potential failure modes relevant to Lower Nihotupu Dam and to assess whether surveillance monitoring, operations and maintenance practices & procedures satisfactorily address these failure modes.

During the workshop seven potential failure modes were identified as being relevant to Lower Nihotupu Dam. Two of these scenarios were discounted on the basis that they were either so remote as to be non-credible or would not lead to failure of the dam. The following five scenarios were considered potential failure modes for Lower Nihotupu Dam:

Potential Failure Mode 1: Overtopping Failure due to Landslide into Reservoir

Potential Failure Mode 2: Overtopping Failure due to Crest Erosion

Potential Failure Mode 3: Overtopping Failure due to Failure of Upper Nihotupu Dam

Potential Failure Mode 4: Piping of the Core through the Dam Body

Potential Failure Mode 5: Piping of the Core along the Abutment Contact with the Spillway

Benchmarks

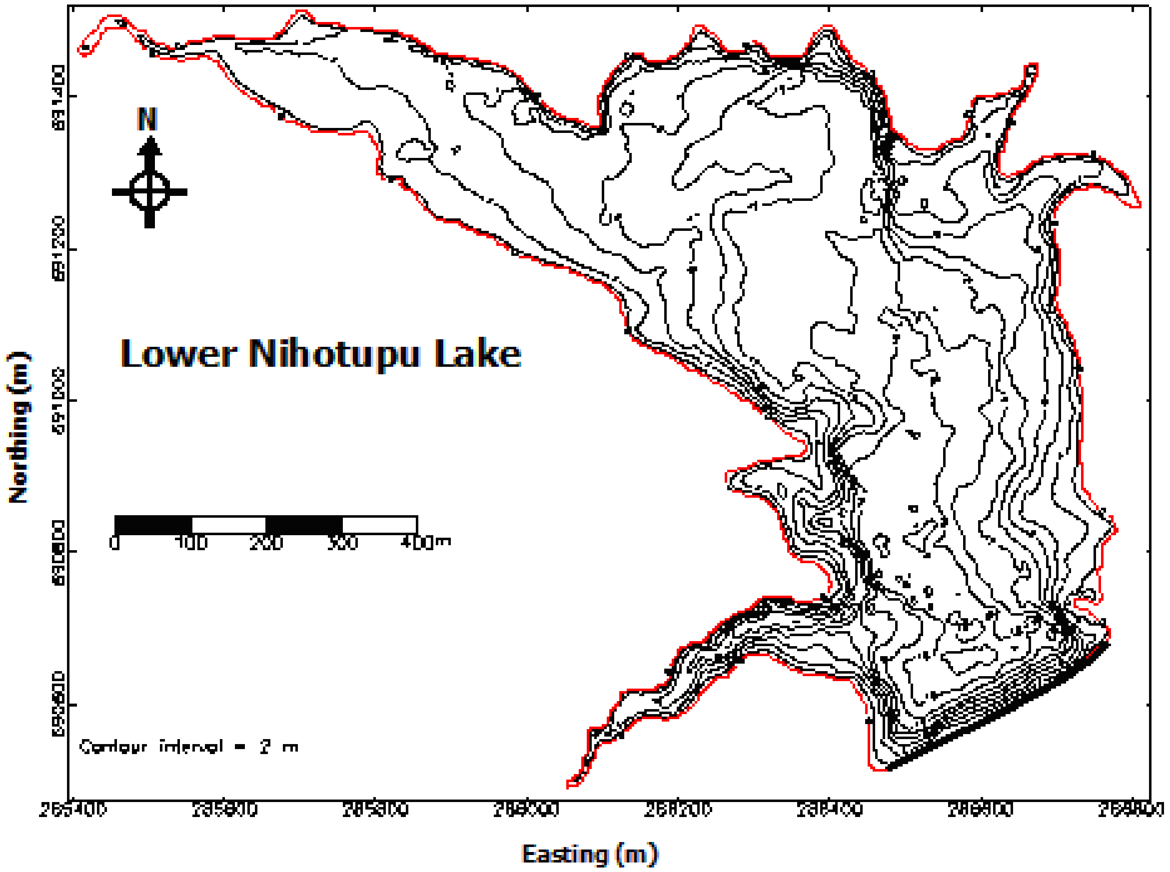
The following table describes the location and Reduced Level of benchmarks at the Lower Nihotupu Dam.

Benchmark Location	Reduced Level (RL)
Valve tower floor by lake Level recorder.	23.540 (Mt Eden 1946)

Continued

Bathometric Plans

The following bathometric plan shows the area and contours of Lower Nihotupu Lake.



Lower Nihotupu Dam Hydraulic Structures Hydrological Data

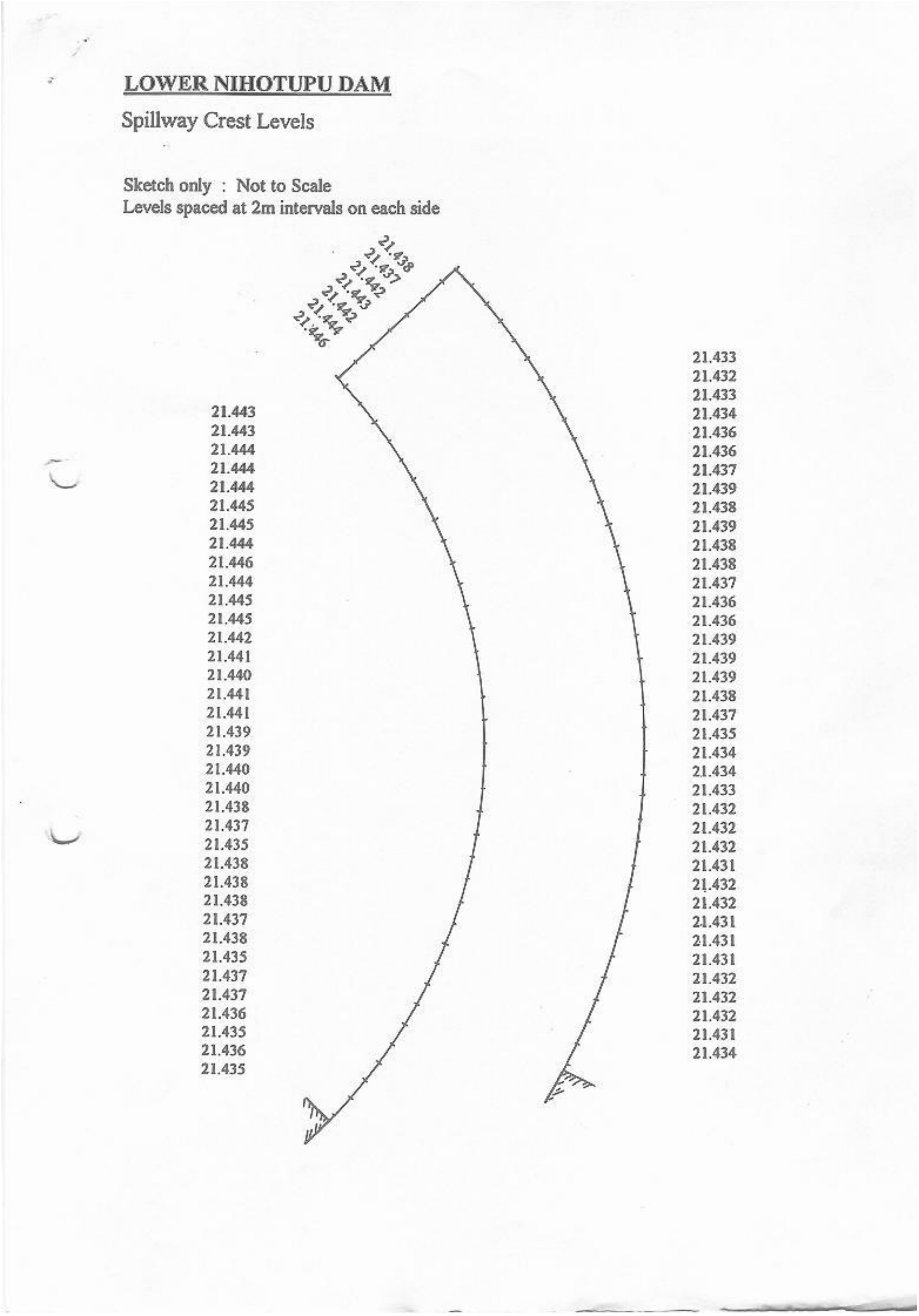
Hydraulic Structure	**Lower Nihotupu Dam**	
Structures		
Levels	**Lands & Survey Datum**	
Top of dam core	22.360	
Dam crest	23.480	
PMF peak level	22.95	
Top of spillway crest	21.430	
Permeability of the material above the core	2 x 10^{-8} m/s	
Permeability of core material	1 x 10^{-9} m/s	
Top of auxiliary spillway crest		
Intake No 1	18.300	
Intake No 2	12.800	
Intake No 3	5.838	
Intake No 4		
Intake No 5		
Intake No 6		
Scour intake	3.026	
Discharges		
Maximum Capacity of supply to treatment	54,000 m^3/day	
Maximum flow capacity spillway crest		
Maximum capacity IFDV	5.25 m^3/s	
Maximum capacity EFDV	3.8 m^3/s	
Catchment		
Water Source	Nihotupu Stream	
Catchment area (ha)	1240	
Surface area of full lake (ha)	52.9	
Live Storage at full volume (m^3)	4,605,000	
Storage between spillway and crest (m^3)	1,044,000	
Hydrology		
Return Period	**Q (m^3/s)**	
	Inflow	**Outflow**
Mean Annual Flow		
5year		
10 year		
20 year		
100 year		
500 year		
1000 year		
PMF	611	592

Benchmark	
Valve Tower	RL 23.450
Hydraulic Structure	**Lower Nihotupu Dam**
Background	
Location	Nihotupu Valley at Parau
Water source	Nihotupu Stream
Purpose	Water supply
Date Built	1945 - 1948
Dam engineering	Auckland City Council
Dam construction	Downer & Company
Dam Construction	
Structure	Earth/rock fill embankment with no filter
Height (m)	24.7
Crest length (m)	381
Crest width (m)	4.572
Dam volume (m^3)	359,000
Valve tower	Free standing
Spillway type	Uncontrolled double sided trough weir
Auxiliary spillway	
Scour Valves	
Valve type located outside dam	FDV (Sleeve valve)
Size (mm diameter)	760
Installation date	2000
Condition	Good
Method of operation	Automatic/manual
Maximum scour flow rate (m^3/sec)	
Maximum scour rate off the intakes (m^3/sec)	3.8
Valve type located within dam	FDV (Sleeve valve)
Size (mm diameter)	760
Installation date	2000
Condition	Good
Method of operation	Automatic/manual
Maximum scour flow rate (m^3/sec)	5.25
Maximum scour rate off the intakes (m^3/sec)	
Notes	

Lower Nihotupu Dam Full Range Spillway Rating

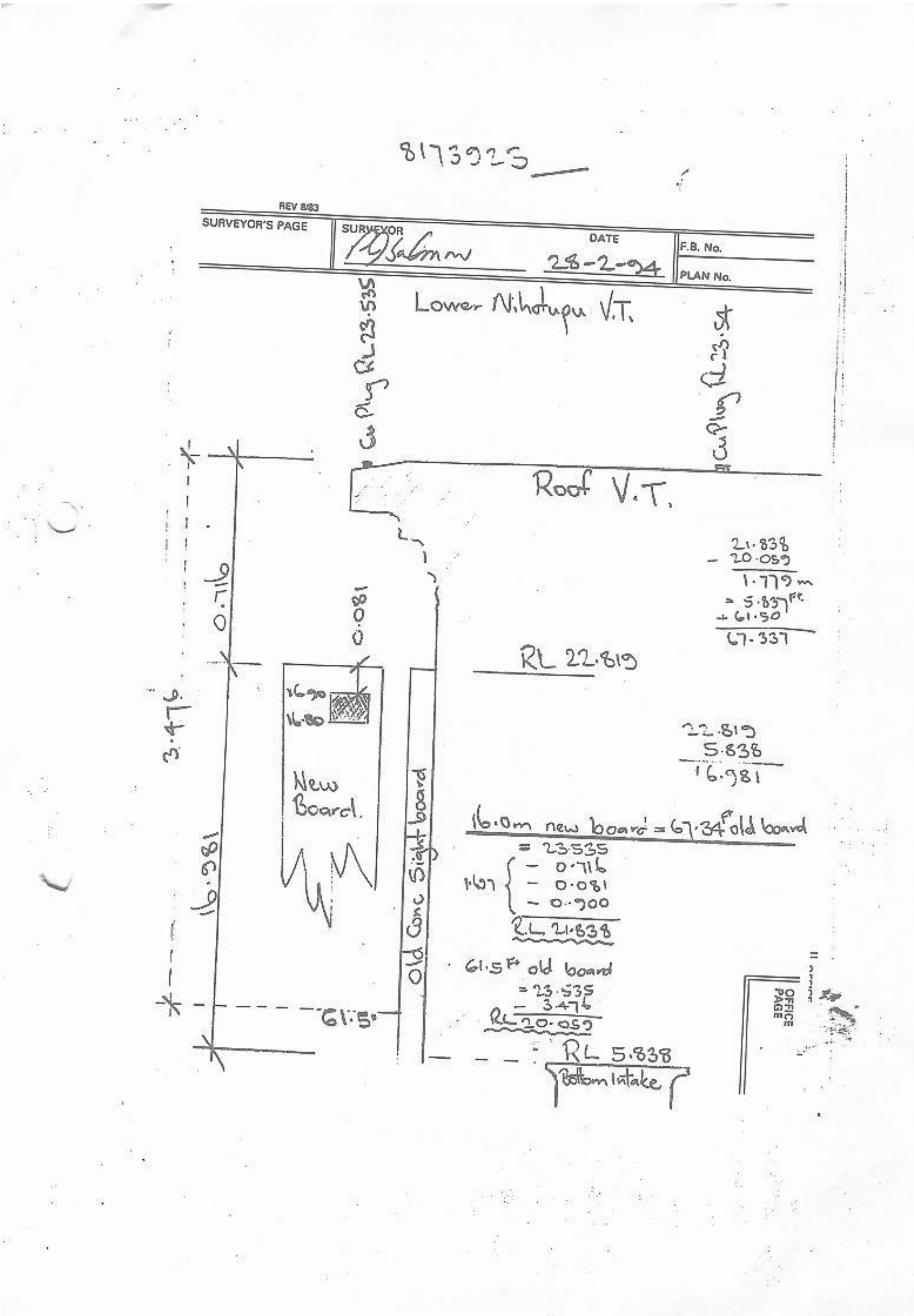
Table 3.4.2 Lower Nihotupu Dam Spillway Rating
Full Range Rating Table

Water Level (m RL)	Gauge Level (m)	Head Over Crest (m)	Total Discharge (m^3/s)
21.431	15.593	0.000	0.0000
21.456	15.618	0.025	0.6011
21.481	15.643	0.050	2.176
21.506	15.668	0.075	4.355
21.531	15.693	0.100	7.035
21.556	15.718	0.125	10.17
21.581	15.743	0.150	13.72
21.606	15.768	0.175	17.67
21.631	15.793	0.200	22.01
21.656	15.818	0.225	26.73
21.681	15.843	0.250	31.82
21.706	15.868	0.275	37.28
21.731	15.893	0.300	43.11
21.756	15.918	0.325	49.20
21.781	15.943	0.350	55.55
21.806	15.968	0.375	62.15
21.831	15.993	0.400	69.02
21.856	16.018	0.425	76.24
21.881	16.043	0.450	83.79
21.906	16.068	0.475	91.64
21.931	16.093	0.500	99.79
21.956	16.118	0.525	108.2
21.981	16.143	0.550	117.0
22.006	16.168	0.575	126.0
22.031	16.193	0.600	135.4
22.056	16.218	0.625	145.0
22.081	16.243	0.650	155.0
22.106	16.268	0.675	165.2
22.131	16.293	0.700	175.8
22.181	16.343	0.750	197.8
22.231	16.393	0.800	221.0
22.281	16.443	0.850	245.6
22.331	16.493	0.900	271.1
22.381	16.543	0.950	296.4
22.431	16.593	1.000	322.1
22.481	16.643	1.050	349.2
22.531	16.693	1.100	376.8
22.581	16.743	1.150	403.7
22.631	16.793	1.200	430.2
22.681	16.843	1.250	457.2
22.731	16.893	1.300	485.0
22.781	16.943	1.350	513.4
22.831	16.993	1.400	542.4
22.931	17.093	1.500	601.8



Lower Nihotupu Dam Spillway Levels

Lower Nihotupu Dam Tide Gauge



Lower Nihotupu Dam Valve Tower Levels

water Care
services limited

Memorandum

TO:	Richard Chandler, Brian Park, Peter Griffiths, Trevor Moore, Ross Sleeman
FROM:	Wal McQuarrie
SUBJECT:	**Lower Nihotupu Valve Tower level Recordings**
CC:	Alan McPike, Harvey Stewart
DATE:	14 May 1996 FILE: 11/18/13

BACKGROUND

The Lower Nihotupu Valve Tower was the only structure where the lake level was not based upon the RL of the bottom intake. Instead, it was based loosely upon the mean high tide level.

ACTION

At approximately 1.30 pm yesterday, the instrument measuring the lake level at Lower Nihotupu was re-calibrated to read levels above the bottom intake (RL 5.838).

There was found to be a discrepancy of approximately 50 mm between the tide gauge and the level recorder, when the changeover occurred. There is a 4.525 mm difference in level between the former data and the new lake level.

RECORDINGS

The caretakers will continue to use the lake level recording for their recordings.

Ian Marshall will correct the archiving procedures at Church Street by the end of this week.

VOLUME/DEPTH TABLES

A new set of volume/depth tables will be produced by the end of next week for issue.

Regards

William McQuarrie

Wallace McQuarrie
Senior Systems Engineer

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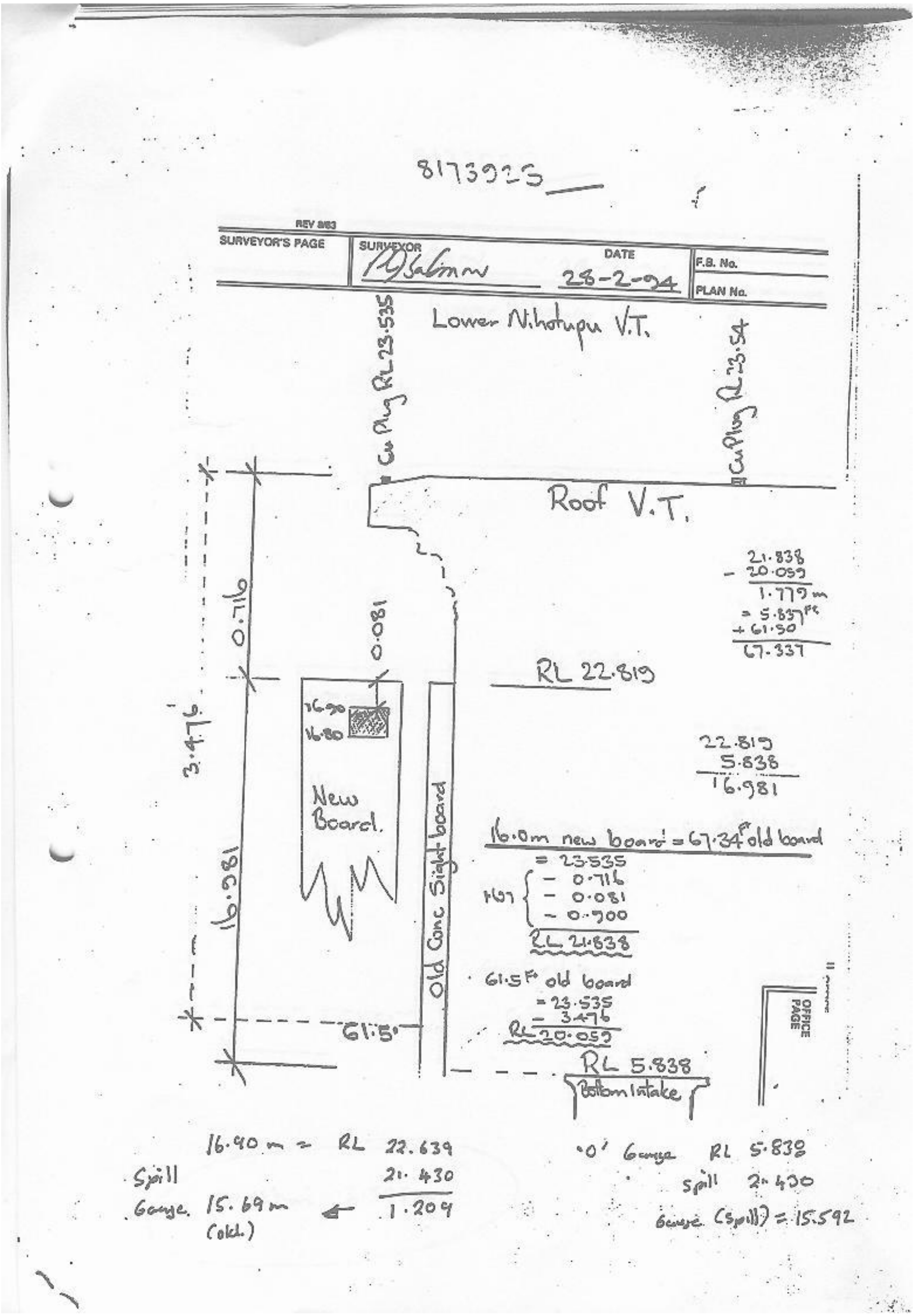
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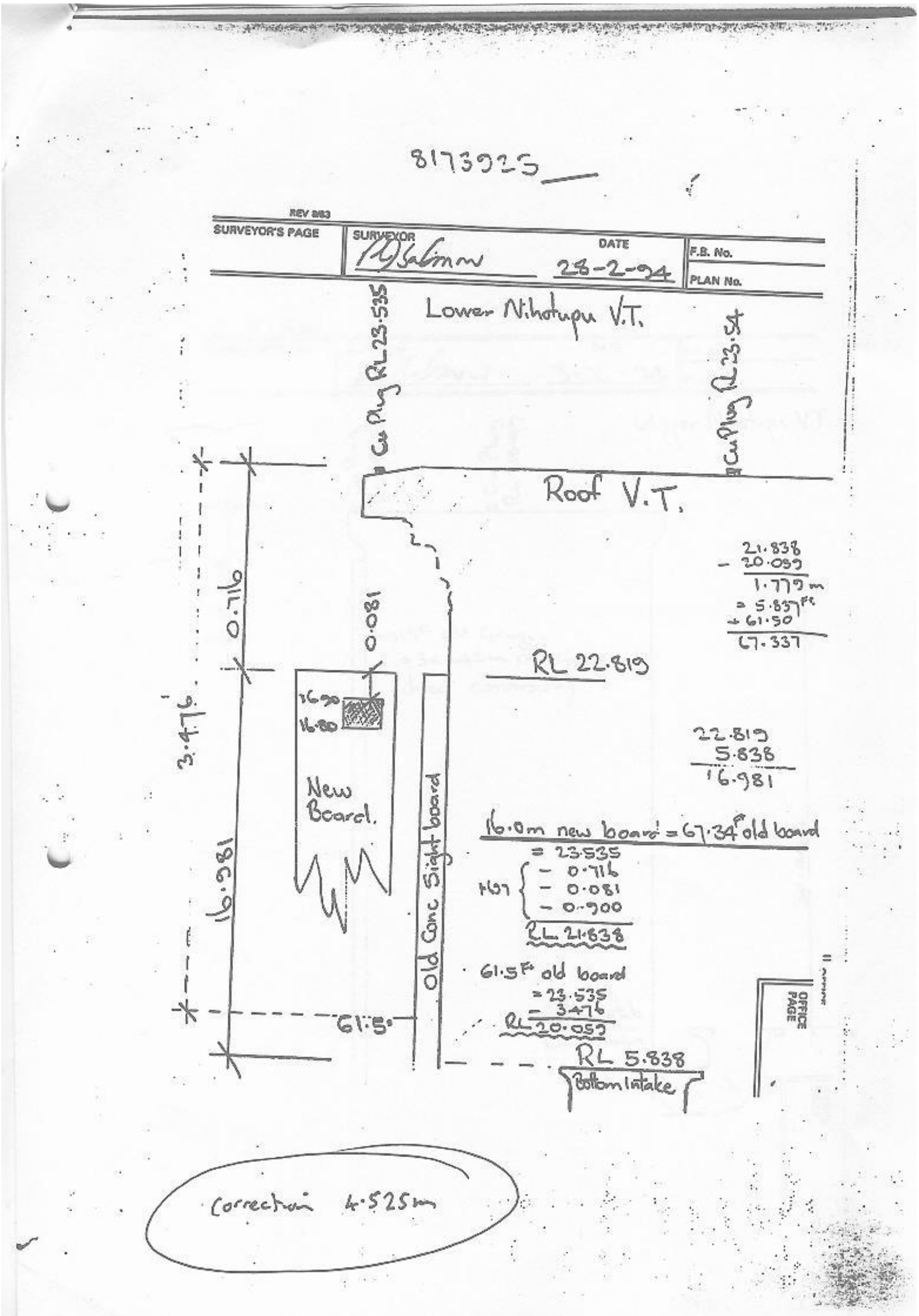
SURVEYOR
DATE
28-2-94
F.B. No.
PLAN No.
Lower Nihotupu V.T.
Cu Plug RL 23.535
Cu Plug RL 23.54
Roof V.T.
21.838
- 20.059
1.779 m
= 5.837 ft
+ 61.50
67.337
RL 22.819
22.819
5.838
16.981
New Board.
16.90
16.80
Old Conc Sight board
16.0m new board = 67.34 ft old board
= 23.535
- 0.716
- 0.081
- 0.900
RL 21.838
61.5 ft old board
= 23.535
- 3.476
RL 20.059
RL 5.838
Bottom Intake
0.716
0.081
3.476
16.981
61.5'
OFFICE PAGE
16.90 m = RL 22.639
Spill 21.430
Gauge 15.69 m 1.209
(old)
'0' Gauge RL 5.838
Spill 21.430
Gauge (Spill) = 15.592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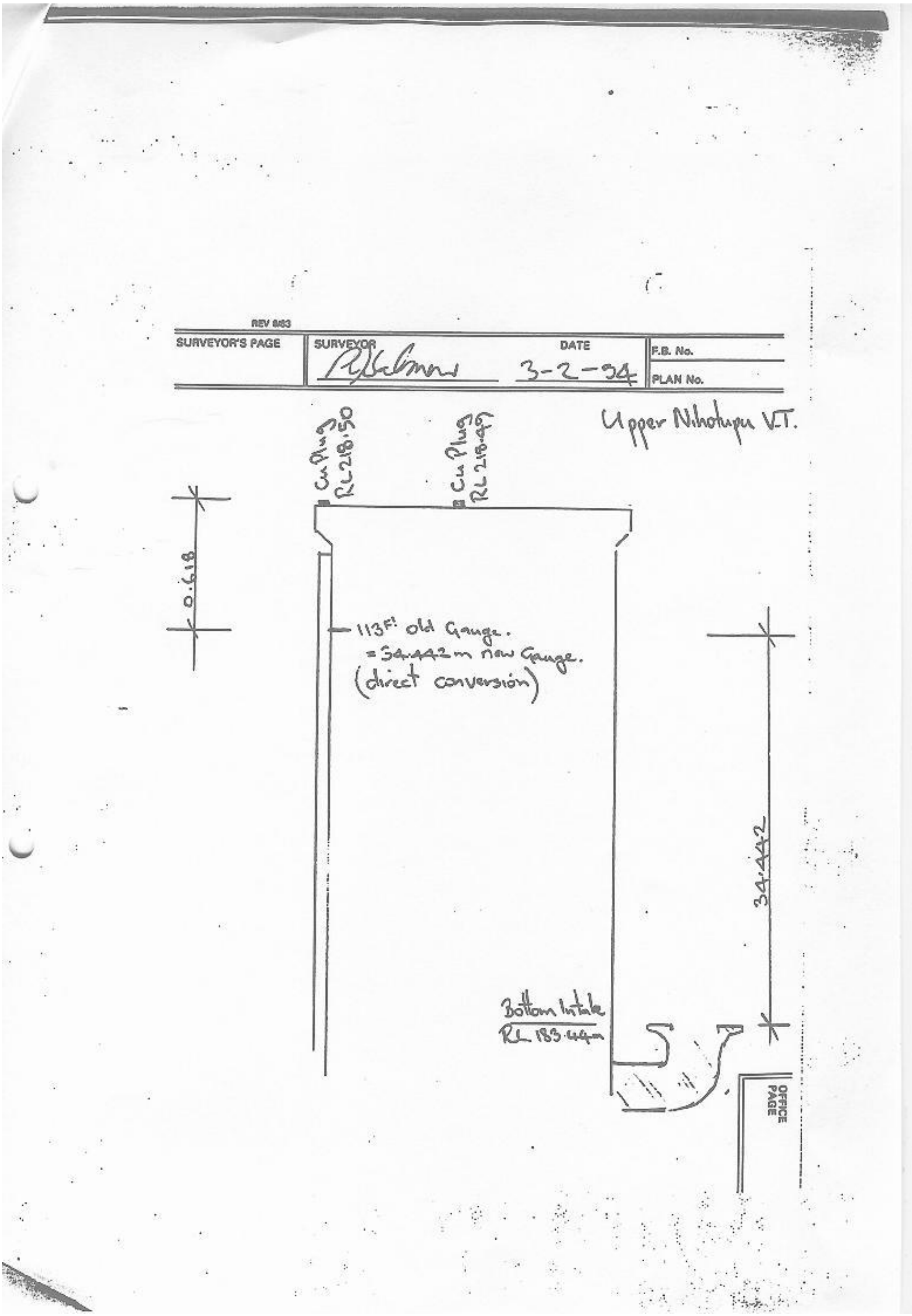
- 20.059
1.779 m
= 5.837 Ft
+ 61.50
67.337
0.716
0.081
RL 22.819
3.476
16.90
16.80
New Board.
22.819
5.838
16.981
Old Conc Sight board
16.0m new board = 67.34 Ft old board
= 23.535
- 0.716
- 0.081
- 0.900
RL 21.838
16.981
61.5 Ft old board
= 23.535
- 3.476
RL 20.059
61.5
RL 5.838
Bottom Intake
OFFICE PAGE
Correction 4.525m


REV 8/83
SURVEYOR'S PAGE
SURVEYOR
DATE
3-2-94
F.B. No.
PLAN No.
Upper Nihotupu V.T.
Cu Plug RL 218.50
Cu Plug RL 218.49
0.618
113Ft old Gauge.
= 34.442 m new Gauge.
(direct conversion)
34.442
Bottom Intake
RL 183.44m
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Potential Impact Classification Lower Nihotupu Dam

Background

Determination of the PICs has been based on dam break studies previously carried out on the dams. These studies considered both “sunny day” and “rainy day” dam failure scenarios. A “sunny day” failure is a failure with no associated rainfall induced flooding resulting from an event such as a large nearby earthquake or internal erosion of dam embankment or foundation materials leading to dam failure. A “rainy day” failure is associated with an extreme flood event which overwhelms the spillway, overtops the dam leading to failure. Cascade dam failures, when the failure of an upstream dam causes the subsequent failure of a dam of dams downstream, were also considered.

It is common for a dam to have a higher PIC for the ‘sunny day’ failure case than for the “rainy day’ failure case, as the incremental damage above the Dam Crest Flood may be less than the damage for the ‘sunny day’ failure. Also, the warning and flooding provided by the large flood preceding the dam breach requires evacuation and the potential for loss of life may be reduced below the ‘sunny day’ case. The highest PIC determined from the relevant dam failure scenarios is used for the purposes of Section 134 of the Building Act (2004). The “rainy day” PIC sets the required standard for the maximum flood magnitude that is expected to be passed safely by the dam. The “sunny day” PIC sets the required standard for the maximum seismic event that the dam can withstand.

Introduction

The Lower Nihotupu Dam is a zoned earthfill/ weathered rock fill embankment dam, 25m high and with a crest length of 365m. It has a free overflow spillway on its left abutment. The dam was commissioned in 1948.

The PIC determination utilises information from the Safety Evaluation and Dam Break Study Report by Tonkin and Taylor3, the Western Area Dams Emergency Preparedness Plan, published by Watercare, dated August 2001 and the Upper Nihotupu Dam 2002 Five-Yearly Safety Review.

The area downstream of the Lower Nihotupu Dam has been inspected to identify the topography, property and infrastructure. The effects on population and property identified in the above reports have been considered in conjunction with the field inspection to determine the Potential Impact Category for the Lower Nihotupu Dam using the Proposed Regulations to the Building Act 2004 and NZSOLD Dam Safety Guidelines.

Table C1: Summary of Failure Scenario

Scenario	Peak Breach Outflow (m^3/s)	Dam Failure Mode
1	760	Earthquake induced failure of the dam
2	700	Piping failure of the dam

The breach outflows were modelled for the short 4 km downstream channel to the sea.

The location of both Lower and Upper Nihotupu Dams are shown on Figure C1.

The biggest outflow from earthquake induced failure was used to determine the PIC as discussed in detail below.

"Sunny day" Assessment of PAR and Fatalities

Downstream of the lower dam, the only building at risk from a lower dam break is the Lower Nihotupu Dam pump station which is normally unmanned. There is a car parking area at the roadside. There is one house downstream of the dam between Huia Road and the true right bank of Big Muddy Creek which is at the edge of the flooded area (a house nearer to the dam on the aerial photo has been removed). The single house is not expected to be inundated to a depth greater than 300 mm. Huia Road, which runs below the Lower Nihotupu Dam will be inundated. Moderate traffic was observed along the road that serves several communities beyond Nihotupu. The Population at Risk is therefore assessed as 1 to 10 based on road traffic and itinerant trampers at the car park.

Fatalities downstream of the lower dam are possible but not expected.

"Sunny day" Assessment of Socio-Economic and Financial Impacts

The value of damage to Huia road and bridge approaches is estimated between $1m and $10m. Since the Lower Nihotupu Dam and pump station are owned by Watercare their loss is excluded from the consequence analysis for Lower Nihotupu dam failure. The incremental impact of the socio-economic is therefore assessed as Major.

"Sunny day" Assessment of Environmental Impacts

The environmental impact in the reach downstream of the dam is considered significant but recoverable and is hence assessed as Moderate.

Determination of "sunny day" PIC

With a PAR of 1 to 10 and a Moderate environment impact the "sunny day" PIC would be Low.

However the PIC is determined by the socio-economic impact of Major which leads to the PIC as Medium in terms of the Proposed Regulations.

The PIC determination in accordance with NZSOLD Guidelines, with respect to fatalities, is also Low as no fatalities are expected, but a Major socio-economic and financial impact leads to a Medium PIC.

"Rainy day" Scenario

As noted previously, "rainy day" scenarios were not modelled for Lower Nihotupu.

Although the spillway has capacity to safely pass the PMF the "rainy day" scenario should be determined in the future.

SUMMARY

The assessment of PIC for Lower Nihotupu Dam is summarised in Table C2.

Table C2: PIC Assessment for Lower Nihotupu Dam

Dam	Socio-economic, financial and environmental impacts				Impact on People				Overall Dam
	NZSOLD and Building Act				NZSOLD		Building Act (Composite Socio + PAR assessment)		
	Facilities	$ Value	Environment	PIC	Fatalities	PIC	Population at Risk (PAR)	PIC	PIC
Lower Nihotupu	Huia Road Bridge approaches	$1m - $10m (excl. Lower Nihotupu Dam and Pump house) $1m - $10m	Significant but recoverable	Medium	No fatalities expected	Medium	1 to 10	Medium	Medium

The "sunny day" PIC for Lower Nihotupu Dam is determined as Medium.

Potential Failure Mechanisms for the Lower Nihotupu Dam

Potential Failure Mode 1: Overtopping Failure due to Landslide into Reservoir

Potential Failure Mode 1 is an overtopping failure of Lower Nihotupu Dam due to increased water levels associated with the failure of the 40-Acre pipeline slip.

40-Acre Slip

The “40-Acre Slip” is located at the head of the Lower Nihotupu Reservoir and is the only the only known area of significant instability. The slip comprises colluvial debris and is mainly underlain by the Waitemata Formation, and the Cornwallis Formation and breccia-conglomerates at the top of the slope.

The foreshore slip investigation (Watercare 2000) reported that ground movements appear to concentrate towards the eastern end of the above ground pipeline and that high groundwater levels, evidenced by the presence of two water courses, play a major role in triggering intermittent movements.

Current deformation survey data indicates that the current rate of movement is 50-60mm/year over the past 10 years, which does not present a dam safety concern. Drainage at higher ground, in the form of open trenches, appears to be effective at reducing the rate of movement.

A 20° ground slope does not suggest rapid failure, however accelerated movement could be expected in very wet (storm) events. The slip is located at the head of the lake and wave action is not expected to directly impact on the dam. The dam is also serviced by a concrete wave wall.

The foreshore slip investigation estimated the volume of displaced water as 160,000m^3 and lake area at top water level as 540,000m^3, which would result in a widespread increase in lake level of 300mm (RL21.73) which would overtop the core by up to 1.23m, refer Section 5.2.1 for detailed explanation, but is expected to be of short duration before the spillway lowers the extra water level.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 1 are:

- Deformation surveys,
- Damage to or movement of the Huia Pipeline,
- Severe weather warnings,
- Deterioration of drainage conditions,
- Failure of the Huia Pipeline

Possible actions to reduce risk include:

- Geological reconnaissance by a landslide geologist,
- Engineering interpretation of slide information,
- Regular inspection and maintenance of the drains,
- Improved drainage,
- Warning instruments for land deformation and pipeline failure.

Potential Failure Mode 2: Overtopping Failure due to Crest Erosion

Potential Failure Mode 2 is an overtopping failure of Lower Nihotupu Dam due to high lake levels flowing through the fill material above core and resulting in crest erosion.

Depth to the Core

Three boreholes were drilled through the embankment crest to determine soil properties as part of the Stage II SEED Investigations. Two additional boreholes were drilled in the crest in 1995. Borehole logs indicate that the depth to the core varies from 1.5m to 3.5m, up to 1.5m below full supply level, refer Figure 2. Borehole logs indicate that the material above the core is a moderately to well compacted non-plastic mixture of clay silts and silty gravels.

In view of the high uncertainty about the depth, extent and nature of materials overlying the core it is not possible to judge the likely dam performance for lake levels above the flood of record. The 2010 CSR reviewer reports that the flood on record was in July 1994, with a peak discharge of $108m^3/s$ corresponding to a reduced level of 21.96m, 1.54m below the crest but overtopping the core by 530mm. The Lower Nihotupu Dam “rainy day” dam break study (Damwatch 2010) determined that the floodwave resulting from the PMF, assuming failure of the Nihotupu Auxiliary Dam and controlled overtopping of Upper Nihotupu Dam would overtop the Lower Nihotupu spillway crest by about 1.6m. The PMF would not overtop the crest, with a freeboard of about 0.6m, however the core of the dam could be overtopped by up to 3.0m at BH2.

On 3 November 2009 during the CSR inspection the reviewer noted a damp patch on downstream face above top berm near centre of dam. On the day of the inspection the lake was at spillway crest level (RL21.43m). During the workshop on 11 March 2010 the Caretaker was requested to inspect the downstream face at this location and reported that there were no damp patches evident. The lake level on 11 March 2010 was RL21.01m, 420mm below spill level.

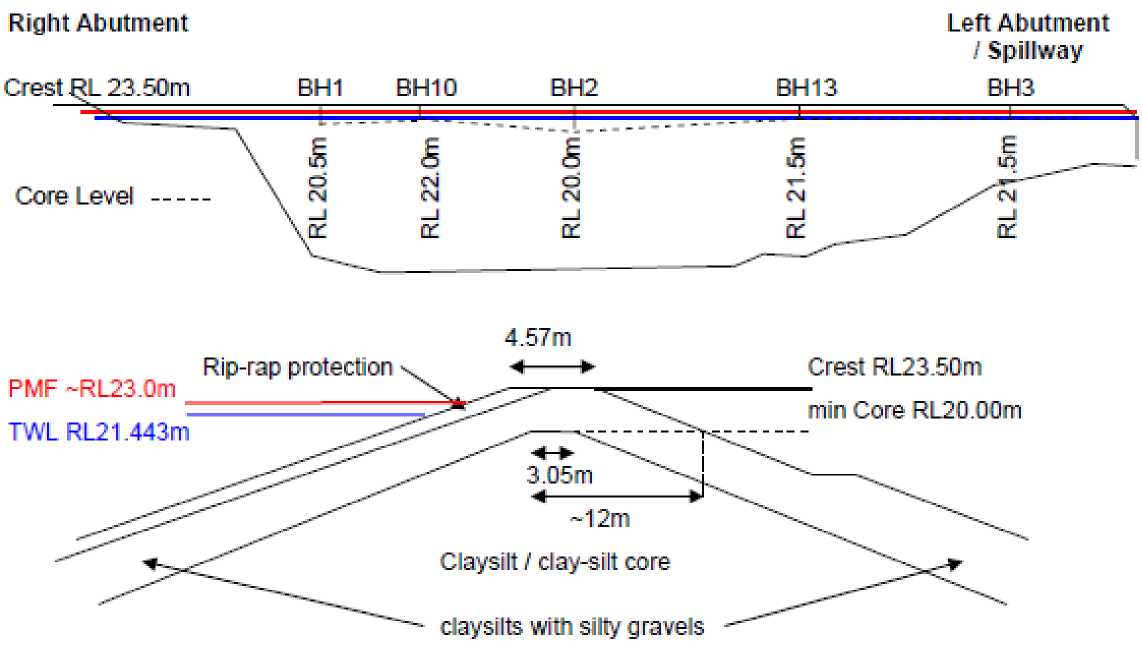


Figure 2: Approximate Borehole Locations and Depth to Core

Potential for Failure Mode to Develop

The Lower Nihotupu Dam Stability Assessment (Damwatch 2007) noted that there is uncertainty as to whether the material above the core would remain stable under seepage head.

The hydraulic flow path is estimated as 12m from the upstream edge of the core. At top water level (RL21.43m) this equates to a hydraulic gradient of about 0.1 and under the PMF a hydraulic gradient of about 0.2, based on the lowest level of the core.

For erosion to initiate at a hydraulic gradient of 0.2 would require a uniformity coefficient of 2, which is equivalent to a layer of uniform sized sand across the entire crest. The presence of a coarse gravel lenses on the upstream side would increase the critical gradient.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 2 are:

- High lake level (above flood of record),
- Observation of seepage on the downstream face,
- Settlement of the crest.

Possible actions to reduce risk include

- Establishing the true level of the core and material properties,

- Undertaking specific inspection of the downstream face at the top berm at high lake levels,
- Setting alarm parameters for lake level based on the flood of record,
- Increasing the level of the core.

Potential Failure Mode 3: Overtopping Failure due to Failure of Upper Nihotupu Dam

Potential Failure Mode 3 is an overtopping failure of Lower Nihotupu Dam due to failure of the Nihotupu Auxiliary and Upper Nihotupu Dams.

Dambreak Analysis

The *Potential Impact Category Assessment for Watercare's Dam* carried out in 2006 by Damwatch commented that there had been no 'sunny day' cascade failure because the lower dam has freeboard capacity to absorb the upper reservoir volume and would not fail, and that previous dam break analyses are inadequate for 'rainy day' PIC determination.

Rainy day dam break scenarios were modelled as part of the *Upper & Lower Nihotupu Dams Rainy Day Dam Break Study* as input to the *Classification of 15 Dams Owned by Watercare Services Limited* by Damwatch in July 2010, and incorporated the PMF developed in the *Probably Maximum Flood Study for Watercare Dams* carried out by Opus in 2008.

The Upper and Lower Nihotupu Dam "rainy day" dam break study determined that the flood wave resulting from the failure of the Nihotupu Auxiliary and Upper Nihotupu Dams under the PMF would overtop the Lower Nihotupu spillway crest by about 2.3m and the crest of the dam by about 5mm.

Borehole logs indicate that the depth to the core varies from 1.5m to 3.5m, up to 1.5m below full supply level, refer Section 5.2.1, therefore the core would be overtopped by up to 3.5m.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 3 are:

- Failure of the Nihotupu Auxiliary and Upper Nihotupu dams, and
- High lake level at Lower Nihotupu Dam.

A cascade failure scenario should be managed in the surveillance, operations & maintenance procedures and emergency preparedness at Upper Nihotupu Dam.

Possible actions to reduce risk include:

- Incorporating an auxiliary spillway
- Establishing the true level of the core and material properties,

- Increasing the level of the core,

- Reducing the top operating level of the dam, and

- Planning to draw down the reservoir if there is evidence of impending failure of Upper Nihotupu Dam.

Potential Failure Mode 4: Piping of the Core through the Dam Body

Potential Failure Mode 4 is piping of the core into the toe drain or clean gravels underlying the downstream shoulder due to lack of filter protection.

Core Details

The dam was originally designed to comprise a central clay-silt core 88ft (26.8m) wide at the base tapering to 15ft (4.6m) at the top, shoulder material of generally a one-to-one mix of silt and gravel grading outwards to straight gravel at the upstream and downstream faces, and a toe drain of selected clean gravels. Early designs included provision for a concrete core wall however laboratory testing of the core material negated the need for a concrete core wall as the material had satisfactory low permeability.

In the early stages of construction the core material consisted of in-situ clay silts, however part way through the second season this was changed to a one-to-one mix of clay and silt due to diminishing supply of clay silts, and was achieved by inter-trailing a layer of clay with an above layer of silt using giant disks. Due to construction difficulties trailing at the near-full height of the core, clay silts were interchanged for the final few feet of the core (Hutchinson, 1951).

Due to the favourable properties of the core and abundance of clay and silt the core was extended into the shoulders without the addition of gravels. The shoulders, reduced to a quarter of their original size, consisted of a one-to-one mix of clay silts with silty gravels by inter-trailing, due to the unavailability of suitable plant to extract favourable gravels from the floodplain (Hutchinson, 1951, Mead).

Seepage Control

The dam does not incorporate an internal filter or chimney drain system; seepage is controlled by a gravel zone at the downstream toe of the embankment. There is uncertainty about the grading of the toe drain and whether a filter was installed between the downstream shoulder and toe drain or between the downstream shoulder and the underlying clean gravels.

Potential for Piping to Occur

The average annual probability of failure for a homogenous dam constructed in the 1940's is less than $<5 \times 10^{-3}$, refer Table 1. Most piping failures arise in the first five years of operation, after which the probability of failure is 1.9×10^{-4}. In addition, the body of the Lower Nihotupu dam is considered to be better than the average homogenous dam constructed in the 1950s with respect to piping.

Table 1: Piping Failure Mode Statistics for Homogenous Dams (Foster 1998)

Construction period	Average annual probability of failure
1900 – 1930	3.6×10^{-2}
1930 – 1950	$<5 \times 10^{-3}$(0 failures)
1950 – 1970	7.7×10^{-3}
All years	1.4×10^{-2}

Soil test results during construction indicate the following mean Atterberg-limits

- Liquid Limit LL 79-84
- Plastic Limit PL 33-40
- Plasticity Index PI 43-48

Under the Unified Soil Classification (USC) the embankment core material is classified as CH, an inorganic clay with a liquid limit higher than 50% (Terzaghi 1996). If a filter is not present, there is nothing to prevent the erosion of fines into the toe gravels or underlying clean gravels. However, the core is highly plastic and therefore the likelihood of backwards erosion and suffusion is negligible under the seepage gradients which occur in a conventional dam.

Piezometric pressures at three sections through the embankment indicate that the phreatic surface is contained in core and there are low hydraulic gradients from the core into the foundation downstream of the cut-off wall. Measured seepage flow is low; however total seepage is not being accurately monitored as seepage flows are affected by the tides and surface water run-off.

The specification called for stripping and benching of the site due to the irregular weathering profile. Due to the favourable cross valley foundation profile there is an extremely low likelihood of transverse cracks being present at foundation level and extending through the dam to connect with the lake, refer Drawing 0315313-05. Drawings 0315313-10&11 indicate steep benching beneath the downstream shoulder towards the right abutment, which is unfavourable with respect to longitudinal cracking.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 4 are:

- Monitoring seepage on downstream face,
- Seepage detection in toe area,

- Observation of cracks or slumping on the crest,
- Changing piezometric pressures.

Possible actions to reduce risk include:

- Investigations to provided confidence in materials and levels,
- Implementing measures to enable robust toe seepage monitoring,
- Upgrading with a filter buttress,
- Dam crest monitoring.

Potential Failure Mode 5: Piping of the Core along the Abutment Contact with the Spillway

Potential Failure Mode 5 is piping of the core along the abutment contact with the spillway.

Spillway Details

The spillway is located on the left abutment and consists of a free overflow weir and concrete lined spillway channel discharging to Big Muddy Creek. The spillway weir extends around the outer end and both sides of the intake channel and is cut into the left abutment with the exception of approximately 50m on the inner side weir which is supported by a cellular structure due to a low foundation level at this location (Auckland City Council 1945).

Drawing 0315316-07A indicates that a vertical face wall was constructed between counterforts at the core-spillway abutment. This is more favourable with respect to crack formation than compacting around exposed counterforts but less favourable than a sloping face. The specification called for the embankment materials to be thoroughly compacted against any concrete features of the spillway using the same method specified for compaction against the cut-off wall.

The field tile drain installed during construction to intercept seepage and control pressures behind the spillway channel lining is monitored at Manhole 1 (WRD13). Potential low stress regions may be present in core at the junction between cut-off and vertical wall, however seepage paths from low stress area potentially track to the tile drain.

Potential for Failure Mode to Develop

The hydraulic gradient across the wall is estimated as 0.5 at top water level (RL21.43m) and 0.75 under the PMF. The likelihood of erosion initiating from a crack in a CH soil is give in Table 2, with values interpolated for a gradient of 0.75.

Table 2: Likelihood of erosion initiating in a CH soil (Foster 1998)

Crack width (mm)	Hydraulic gradient		
	0.5	.075	1.0
2	0.01	0.06	0.1
5	0.05	0.23	0.4
0	0.1	0.45	0.8
25	0.4	0.70	1.0
50	0.95		

For the failure mode to develop would require an earthquake event to generate a crack followed by a low probability flood event increasing the hydraulic gradient across the wall. A potential space for eroded fines exists if the concrete walls are cracked and a void exists behind the vertical wall. However, the core is highly plastic and therefore the likelihood of backwards erosion and suffusion is negligible under the seepage gradients which occur in a conventional dam.

The probability of piping along the abutment contact with the spillway is considered unlikely.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 5 are:

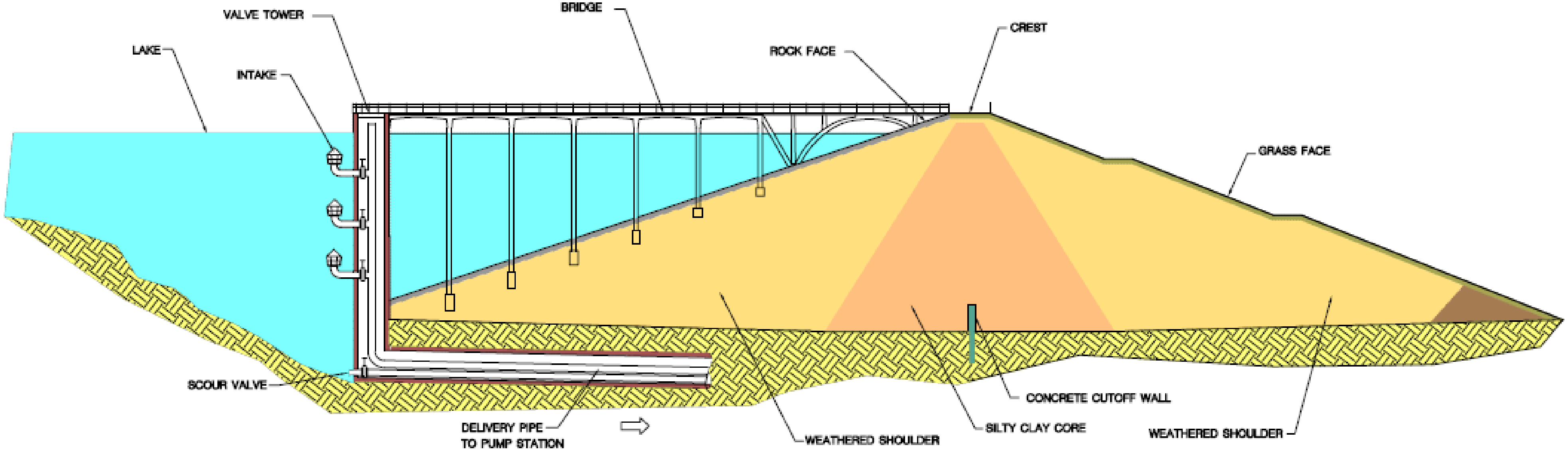
- Monitoring of the tile drain,
- Deformation monitoring,
- Post-earthquake inspections.

Possible actions to reduce risk include

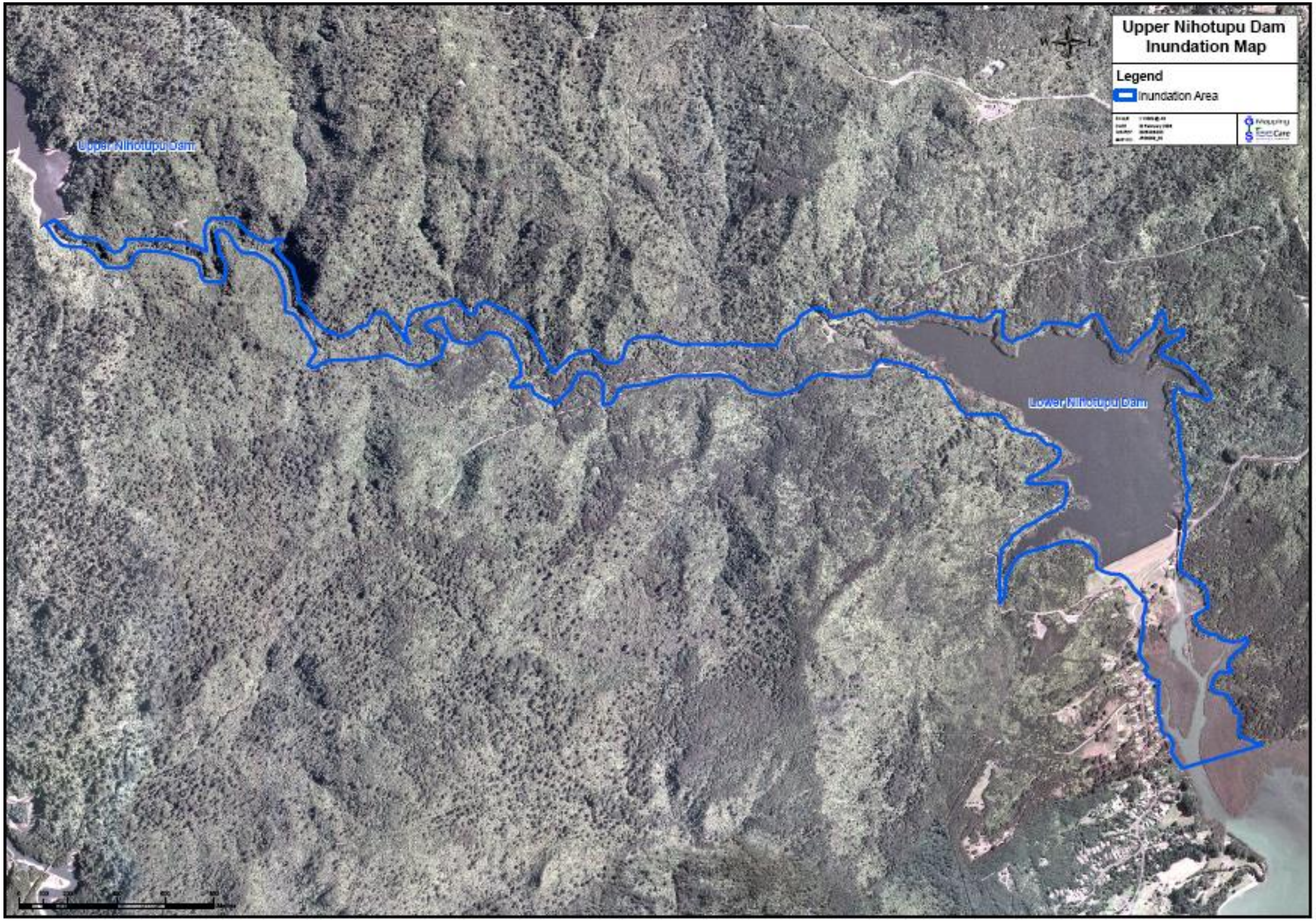
- Reviewing the location of deformation markers,
- Investigating the conditions inside the cavity behind the vertical wall,
- Installing piezometers through the wall into the core contact,
- Undertaking specific post-earthquake inspection of key areas,
- Undertaking specific post-flood response inspections key areas including the tile drain and weirs,
- Developing an isometric sketch of the spillway,
- Backfilling the void behind the vertical wall.

Site Location Lower Nihotupu Dam

Cross Section Lower Nihotupu Dam



Lower Nihotupu Dam Inundation Map



Upper Nihotupu Dam
Inundation Map
Legend
Inundation Area
Upper Nihotupu Dam
Lower Nihotupu Dam

Lower Nihotupu Dam Spillway Rating – Low Range Rating Table

Water Level (m RL)	Gauge Level (m)	Head over Crest (m)	Total Discharge (m^3/s)
21.431	15.593	0.000	0
21.432	15.594	0.001	0.0005
21.433	15.595	0.002	0.0021
21.434	15.596	0.003	0.005
21.435	5.597	0.004	0.0092
21.436	15.598	0.005	0.0147
21.437	15.599	0.006	0.0221
21.438	15.600	0.007	0.0316
21.439	15.601	0.008	0.0435
21.440	15.602	0.009	0.0586
21.441	15.603	0.010	0.076
21.442	15.604	0.011	0.0958
21.443	15.605	0.012	0.1179
21.444	15.606	0.013	0.1423
21.445	15.607	0.014	0.1694
21.446	15.608	0.015	0.1988
21.447	15.609	0.016	0.2312
21.448	15.610	0.017	0.2653
21.449	15.611	0.018	0.3018
21.450	15.612	0.019	0.3394
21.451	15.613	0.020	0.3794
21.453	15.615	0.022	0.4638
21.456	15.618	0.025	0.6022
21.461	15.623	0.030	0.8607
21.466	15.628	0.035	1.151
21.477	15.633	0.040	1.469
21.476	15.638	0.045	1.811
21.481	15.643	0.050	2.177
21.486	15.648	0.055	2.574
21.491	15.653	0.060	2.989
21.496	15.658	0.065	3.424
21.501	15.663	0.070	3.886
21.506	15.668	0.075	4.36
21.511	15.673	0.080	4.861
21.516	15.678	0.085	5.376
21.521	15.683	0.090	5.915
21.526	15.688	0.095	6.467
21.531	15.693	0.100	7.043
21.541	15.703	0.110	8.243
21.551	15.713	0.120	9.513
21.561	15.723	0.130	10.86
21.571	15.733	0.140	12.28
21.581	15.743	0.150	13.74

Lower Nihotupu Dam Spillway Rating – Full Range Rating Table

Water Level (m RL)	Gauge Level (m)	Head over Crest (m)	Total Discharge (m^3/s)
21.431	15.593	0.000	0
21.456	15.618	0.025	0.6011
21.481	15.643	0.050	2.176
21.506	15.668	0.075	4.355
21.531	15.693	0.100	7.035
21.556	15.718	0.125	10.17
21.581	15.743	0.150	13.72
21.606	15.768	0.175	17.67
21.631	15.793	0.200	22.01
21.656	15.818	0.225	26.73
21.681	15.843	0.250	21.82
21.706	15.868	0.275	37.28
21.731	15.893	0.300	43.11
21.756	15.918	0.325	49.20
21.781	15.943	0.350	55.55
21.806	15.968	0.375	62.15
21.831	15.993	0.400	69.02
21.856	16.018	0.425	76.24
21.881	16.043	0.450	83.79
21.906	16.068	0.475	91.64
21.931	16.093	0.500	99.79
21.956	16.118	0.525	108.2
21.981	16.143	0.575	117.0
22.006	16.168	0.575	126.0
22.031	16.193	0.600	135.4
22.056	16.218	0.625	145.0
22.081	16.243	0.650	145.0
22.106	16.268	0.675	165.2
22.131	16.293	0.700	175.8
22.181	16.343	0.750	197.8
22.231	16.393	0.800	221.0
22.281	16.443	0.850	245.6
22.331	16.493	0.900	271.1
22.381	16.543	0.950	296.4
22.431	16.593	1.000	322.1
22.481	16.643	1.050	349.2
22.531	16.693	1.100	376.8
22.581	16.743	1.150	403.7
22.631	16.793	1.200	430.2
22.681	16.843	1.250	457.2
22.731	16.893	1.300	485.0
22.781	16.743	1.350	513.4
22.831	16.993	1.400	542.4
22.931	17.093	1.500	601.8

Estimation of Earthquake Spectra

Reference Report 4181
Estimation of earthquake spectra for Watercare dams
M. C. Gerstenberger & G. McVerry
GNS Science Consultancy Report 2006/048
May 2006

Report No 4128 – Estimation of Earthquake Spectra for Watercare Dams by GNS Science Consultancy Report, May 2006

Refer to report 4128 when considering the potential seismic loading on dams. The minimum design earthquake motions for the Auckland region in NZS1170.5:2004 were taken into consideration.

This study was commissioned to develop appropriate acceleration response spectra for Watercare dams in the Hunua and Waitakere Ranges near Auckland. For the Waitakere Ranges, a single site has been selected for calculating the hazard: the Upper Nihotupu dam (36.97ºS, 174.57ºE).

Using the current New Zealand national seismic hazard model, we calculated the results using both a probabilistic seismic hazard analysis (PSHA) and a deterministic analysis. For PSHA, the hazard model consists of two components: 1) major faults that describe earthquake location, magnitude and recurrence interval of primarily magnitude M>7 events; and 2) the smoothed regional background seismicity based on the entire historical earthquake catalogue.

Return periods of 150, 500, 1,000, 2,500 and 10,000 years were considered in the analysis.

For both ranges, a deterministic study was performed using a magnitude 6.5 earthquake at a distance of 20 km. The spectrum for ⅔ of the 84-percentile motions for this event corresponds to the minimum design earthquake (MinDE) motions for the Ultimate Limit State in the Auckland region as designated in the new standard NZS1170.5:2004 (Standards New Zealand, 2004) for earthquake actions in New Zealand.

Based on the interpretation of the NZSOLD (2000) guidelines by Mejia et al. (2001) we recommend the following spectra as the Safety Evaluation Earthquake (SSE) motions:

- Smoothed 10,000 year motions for High Potential Impact Category (PIC) dams
- Smoothed 2,500 year motions for Medium PIC dams
- Smoothed MinDE motions for Low PIC dams

In Figure 1 we show the smoothed spectra for each location, and in Table 1 we present the recommended smoothed values for each period. The spectra are presented for rock site conditions i.e. site class B in terms of NZS1170.5.

Table 1 Recommended Class B smoothed spectra for SEE[1] Motions

	Hunua			Waitakere		
Period (s)	High PIC 10,000 years	Med. PIC 2,500 years	Low PIC MinDE motions [2]	High PIC 10,000 years	Med. PIC 2,500 years	Low PIC MinDE motions [2]
0.0	0.28 (g)	0.19 (g)	0.12 (g)	0.19 (g)	0.12 (g)	0.12 (g)
0.075	0.66	0.43	0.29	0.44	0.31	0.29
0.1	0.66	0.43	0.29	0.44	0.31	0.29
0.15	0.66	0.43	0.29	0.44	0.31	0.29
0.2	0.66	0.43	0.29	0.44	0.31	0.29
0.25	0.66	0.43	0.29	0.44	0.31	0.29
0.3	0.66	0.43	0.29	0.44	0.31	0.29
0.35	0.59	0.39	0.26	0.40	0.27	0.26
0.4	0.53	0.35	0.23	0.36	0.25	0.23
0.5	0.45	0.30	0.20	0.31	0.21	0.20
0.75	0.33	0.22	0.15	0.23	0.16	0.15
1.0	0.27	0.18	0.12	0.18	0.13	0.12
1.5	0.20	0.13	0.09	0.13	0.09	0.09
2.0	0.15	0.10	0.07	0.11	0.07	0.07
3.0	0.10	0.07	0.04	0.07	0.05	0.04

[1] SEE Motions - Safety Evaluation Earthquake (or Maximum Design Earthquake) motions

[2] MinDE motions – Minimum Design Earthquake motions (⅔ of 84-percentile motions for magnitude 6.5 earthquake at 20 km) govern rather than 500 year return period motions

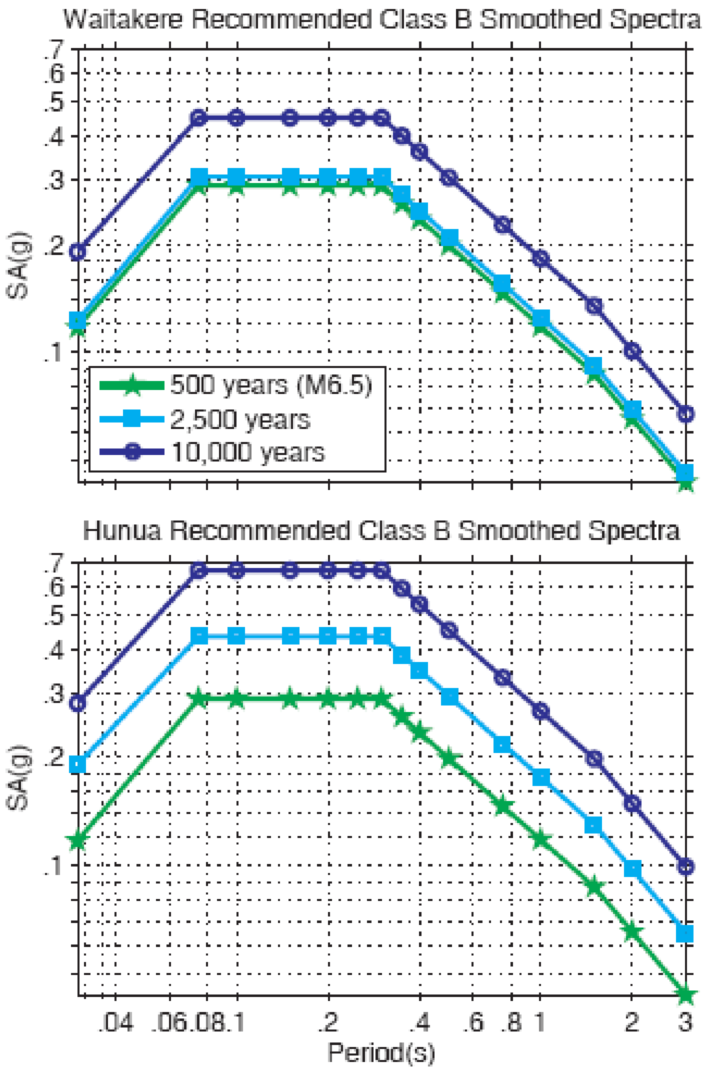


Figure 1 Recommended smoothed spectra for the Safety Evaluation Earthquake motions for the Waitakere and Hunua Ranges dams.

WAITAKERE RANGES RESULTS

For the Waitakere Ranges all calculations were performed for a single representative site corresponding to the location of the Upper Nihotupu Dam (36.97ºS, 174.57ºE). This was possible because the Waitakere dams are remote from active faults, and the distributed seismicity governing the seismic hazard varies only slightly between the dam sites. As it is more than 40 km distant, no deterministic calculations based on the Wairoa North Fault were done for this site, however probabilistic calculations were done using five different return periods: 500 years, 1,000 years, 2,500 years, 5,000 years and 10,000 years. Also, as described in section 2.2.1, in the probabilistic calculations the Wairoa North Fault was allowed to rupture in either a magnitude 6.6 or a magnitude 7.1 event. Lastly, the results were compared to the minimum design earthquake (MinDE) motions for the Auckland region in NZS1170.5:2004. The MinDE motions are derived from the minimum allowable hazard factor, Z, of 0.13. This is a greater value than has been estimated for the Auckland area (Z=0.1) but is set to ensure compliance in low seismicity areas with the performance objective to withstand the most severe shaking that the structure is likely to be subjected to with a small

margin against collapse. The Z value corresponds to ⅔ of the 84th percentile motions of a magnitude 6.5 normal faulting earthquake at 20km from the site; the largest event likely to occur in low seismicity regions in New Zealand without previous surface expression of the fault. The factor of ⅔ in NZS1170.5 comes from assuming that the design level motions incorporate a margin against collapse of 1.5, a commonly used value for building structures. The appropriateness of this margin may need to be assessed for the Watercare dams.

The raw spectra for all calculations are shown in Figure 2.

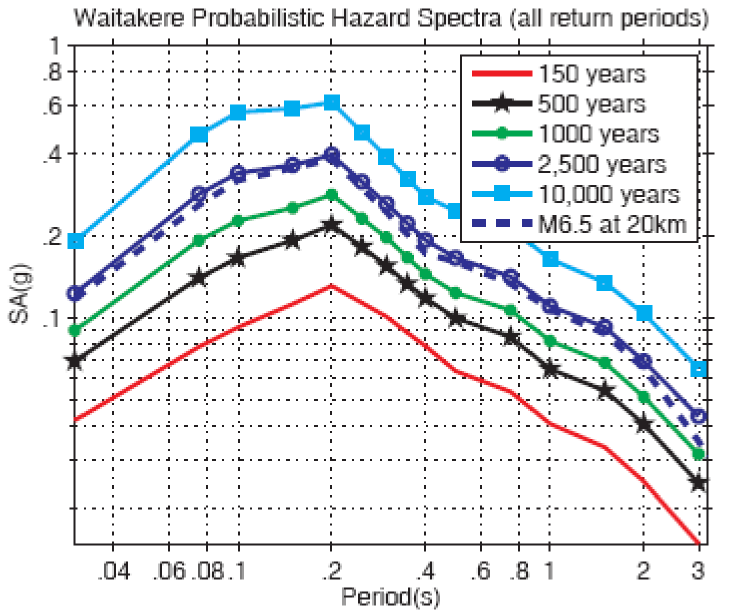


Figure 2 Raw spectra for all calculations done for the Waitakere Ranges site for a Class B rock site.

Dam Potential Impact Category

The decision about which spectra to recommend is primarily based upon the Dam Potential Impact Categories (PIC) as defined by NZSOLD (2000). The criteria given in the Mejia et al. (2001) interpretation of the NZSOLD Guidelines are shown in Table 4. The hazard in the Waitakere Ranges is dominated by the background seismicity; therefore the only deterministic study we performed was the evaluation of a magnitude 6.5 event at a distance of 20 km to satisfy the minimum design earthquake requirements from NZS1170.5:2004.

Following the code designation, the spectra for the MinDE are shown as 2/3 of the 84th percentile motions for this event. As can be seen in Figure 2, both the 10,000- and 2,500- year motions are higher than the MinDE motions; therefore the required raw spectra for high and medium PIC dams are the 10,000- and 2,500-year motions. However the 500-year motions are less than the MinDE motions and based on NZS1170.5:2004, we therefore recommend the MinDE spectra for low PIC dams.

Table 4 Summary of seismic load evaluation criteria Adapted from Mejia et al. (2001) interpretation of NZSOLD (2000) guidelines

Dam PIC	High	Medium	Low
Safety Evaluation Earthquake Motions	84th percentile for Controlling Maximum Earthquake (CME). Need not exceed 10,000-year motions	50th – 84th percentile for CME. Need not exceed 2,500-year motions	CME. Need not exceed 500-year motions
Operating Basis Earthquake Motions	150-year motions	150-year motions	150-year motions

3.2 Smoothing of the raw spectra

Smoothed design envelopes were developed to largely envelope the raw spectra from the hazard analyses for the recommended SEE motions: MinDE, 2,500 years, and 10,000 years.

The construction of these envelopes followed procedures similar to those used in developing code spectra, although different from the specific procedures used for NZS1170.5:2004.

Each spectrum comprises a segment rising linearly with period T from the 0s value to period T1, a constant spectral acceleration plateau at the peak of the smoothed spectrum, and a descending branch in which the spectral acceleration reduces with increasing spectral period T. The smoothing procedure involves defining an appropriate amplitude and period band for the constant acceleration plateau, and approximating the descending branch by segments proportional to T -γ, where the exponent γ takes values such as 2/3, 3/4, 1 or 2 in various segments. The smoothed spectral shape adopted for Class B sites in the New Zealand code has a branch proportional to T -0.75 in the period range from the corner period T2 to 1.5s, with

T2 taken as 0.3s, a constant-velocity branch proportional to T -1 between 1.5s and 3s, and a constant-displacement branch proportional to T -2 at periods beyond 3s. The smoothed spectra recommended in this study have been guided by the draft code spectra. However, the corner periods T1 and T2 and the amplitudes of the descending branches have been varied to more appropriately reflect the site-specific study results established from the hazard analyses.

The smoothing parameters are defined in Table 5.

Table 5 Recommended spectral parameters for the Waitakere ranges.

Return Period	T_1 (s)	T_2 (s)	Sa(0) (g)	Sa(T) for $T_1<T<T_2$ (g)	Sa(T) for $T_2 <T <1.5s$	Sa(T) for $T>1.5s$
MinDE motions	0.075	0.3	0.12	0.29	$0.087 *(1.5/T)^{.75}$	0.087*1.5/T
2,500 years	0.075	0.3	0.12	0.31	$0.092*(1.5/T^{.)75}$	0.092*1.5/T
10,000 years	0.075	0.3	0.19	0.45	$0.134*(1.5/T)^{.75}$	0.134*1.5/T

A comparison of the raw to the recommended smoothed spectra for the Waitakere Dam sites is show in Figure 3. The smoothed values for each period are listed in Table 1.

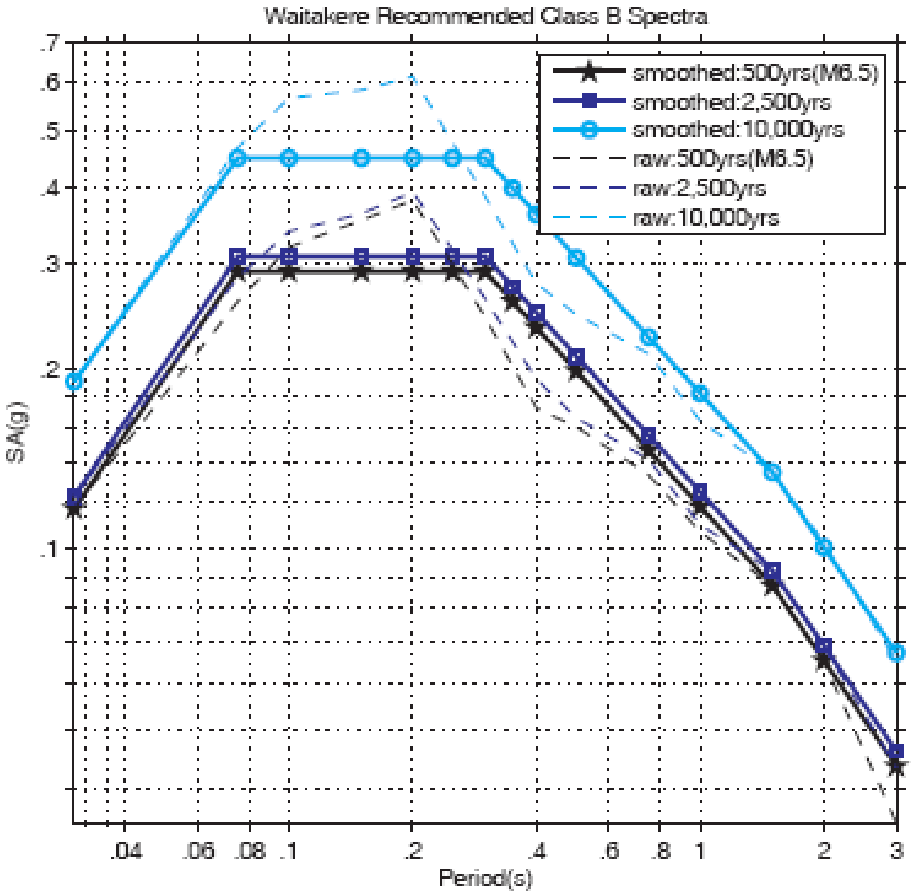


Figure 3 Recommended raw and smoothed spectra for the Waitakere Ranges for a Class B rock site.

Shown are calculations for 10,000 years (PIC: high); 2,500 years (PIC: med); and a magnitude 6.5 event at 20km (PIC: low).

OPERATING BASIS EARTHQUAKE

Following the NZSOLD guidelines, we have also developed spectra for the Operating Basis Earthquake (OBE). In the guidelines, the OBE is recommended to be based on the 150 year motions for all PICs, as shown in Table 4. The NZS1170.5:2004 Standard approximates the 150-year spectrum by 0.6 times the 500-year spectrum. When the 500-year spectrum is less than the MinDE motions, an approach consistent with that of NZS1170.5:2004 is to use 0.6 times the MinDE spectrum in place of the 150-year spectrum for the OBE spectrum. In Figures 8 and 9 we show the raw and smoothed spectra based on both of these approaches for determining the OBE motions (the 150 year motions). The smoothing has been done in the same way as for all other spectra presented in this report. Table 7 shows the smoothed values for both approaches.

For both locations, the spectrum for 0.6 times the MinDE motions exceeds the 150-year spectrum. The difference is around 10% for the Hunua Dams, but is about 50% for the Waitakere sites.

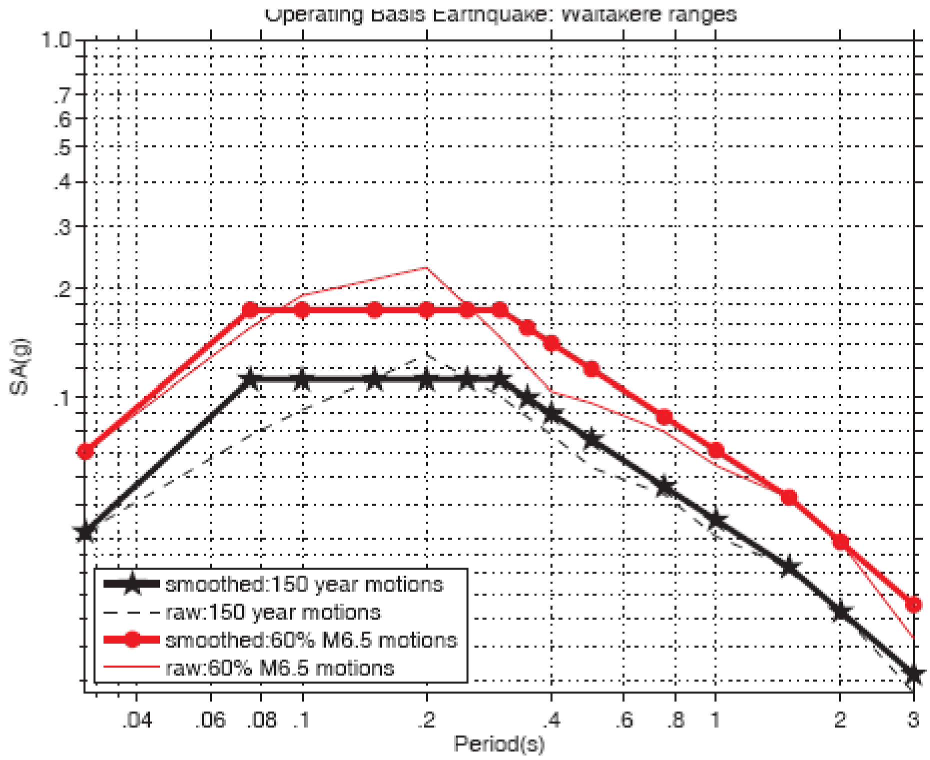


Figure 8 Candidate raw and smoothed OBE spectra for the Waitakere Ranges

Table 7 Recommended OBE spectral parameters for both ranges

Period (s)	Waitakere 150 year smoothed spectra	Hunua 150 year smoothed spectra	60% MinDE smoothed spectra
0.0	0.042	0.067	0.070
0.075	0.111	0.159	0.175
0.1	0.111	0.159	0.175
0.15	0.111	0.159	0.175
0.2	0.111	0.159	0.175
0.25	0.111	0.159	0.175
0.3	0.111	0.159	0.175
0.35	0.099	0.141	0.156
0.4	0.090	0.128	0.141
0.5	0.076	0.108	0.119
0.75	0.056	0.080	0.088
1.0	0.045	0.064	0.071
1.5	0.033	0.047	0.052
2.0	0.025	0.036	0.039
3.0	0.017	0.024	0.026

Appendix A – Geological Exploration

Geological Investigation (extracted from Appendix V of the contract documents, with dimensions metricated)

Soil Survey and Foundation Exploration at Site of Proposed Lower Nihotupu Dam

Being part of the Report to Auckland City Council by City Engineer, Dated 20 March 1944

Note: This was an interim report only, supplied merely for the general information of contractors.

Investigations carried out subsequently to the tabling of this report have led to revisions of certain test results, quantities or other information. Wherever any differences are noted, contractors must understand that the information supplied in the contract documents supersedes that given herein.

Introduction

Investigation work at Lower Nihotupu to determine foundation conditions at the dam site and the extent, nature, and suitability of the materials available for use in the earth embankment is now sufficiently advanced to permit the making of close generalisations as to the conditions obtaining. As a result, it is now possible to state definitely that an earth dam is a feasible proposition for the site.

This investigation was conducted by Mr. C W Firth, M.Sc., A.M.Inst.C.E.

Field Work

Field work, including additions to the original programme put in hand to cover the possible substitution of a multiple arch concrete structure for the earth dam primarily under consideration, is now complete. In this connection, routine outcrop geological investigation has been supplemented by the sinking of 71 test shafts, of an aggregate depth of 322 metres, and by carrying out of other works spread over the various phases of the work in general, as follows;-

Dam Site:

Shafts	39;	average 4.19m deep	162.8m
Tunnels	2;	15.24m and 27.212 m long	42.4 m
Bores	56;	average 3 m deep	170.7 m

Western Borrow Pit Area: (Spurs west of dam site)

Shafts	21	average 6.1 m deep	115.8m
Crosscuts	9	average 38.1 m long	342.9 m

Upstream Borrow Pit Area: (stream-flats upstream from dam site)

Shafts	11	average 4 m deep	43.6 m
Trenches	16 lines 2.1 m deep		755.9 m

Laboratory Work

To test the suitability of the materials available for use in the work has been done in part in location and in part by the valued co-operation of the Soil Survey Division of the Department of Scientific and Industrial Research. In general, the policy adopted in this connection has been to use the Departmental laboratory for the determination of grading analyses and moisture characteristics of the many individual soil types which it was early recognised would have to be dealt with, and with this “standard” information as a basis, to concentrate, in the laboratory established by the Council in the field, on tests more directly connected with the engineering design and analysis as applied to such admixtures of the several types which will come within the limits of economic practice on the job.

22 different soil types have been submitted to the Department and dealt with by them. Much of this material is of exceptionally fine grain, and therefore has involved considerable routine work, of a particularly tedious nature. The time alone required to conduct tests of this character has rendered the assistance of the Department particularly valuable to the Council under the ruling conditions of urgency and shortage of staff.

The field laboratory has put through complete engineering tests on seven soil types and mixtures which have direct application to the job, and has undertaken the necessary stability analysis to check the adopted design. Much routine testing has also been put in hand.

Detailed Report

In the following section of this report, details are given, under appropriate headings, of the various phases of the investigations on which the conclusions quoted are based.

Geology

"Country rock" on location is the mid-Tertiary (Miocene), so-called Waitemata Series of sedimentary rocks of the type popularly known as "papa". Generally speaking, the series comprises alternating series of approximately 75% of tight mudstone and siltstone in either thin or heavy beds, and 25% of fine to medium-grained sandstones in beds ranging from mere threads t a thickness of approximately 3 m. Interbedded, locally with these "normal" strata of the series on location, there is a notable variant in the form of a thick bed of semi-volcanic material to which the name "Parnell Grit" horizon is applied. The basal of this bed is very coarse grained, and is sharply defined from the various underlying normal strata. The succeeding 6 m of the horizon varies from coarse to fine tuffaceous sandstone and is largely gradational up into the overlying beds.

In the hills to the west and north of location, Waitemata strata disappear under the great pile of andesitic volcanic rocks which form the Waitakere ranges. The contact between the two formations, Waitemata and Waitakere, is probably transitional and apparently occurs, locally, at approximately RL 93 m. The latter, therefore, does not come directly within the scope of the work.

Throughout the lower part of the valley, the Nihotupu stream, in achieving its present position, has cut terraces at various levels up to RL 38 m in the Waitemata strata which here form the valley sides, and has deposited thereon extensive accumulations of andesitic silts and gravels derived from the neighbouring hills. The modern and most recent members of this terrace sequence form fairly extensive flood-plain flats bordering the stream throughout the valley floor. The central 122m of the dam site crosses such a flat.

Structure of the "Country" Rock

Waitemata strata on location are disturbed in a manner typical of the series everywhere. Generally speaking, the dam site is located a north-plunging anticline, so that strata dip into the country in both abutments. In the case of the east abutment, where there are several strong beds of water-bearing sandstones, this feature will involve special attention in the matter of drainage, and depths of stripping and foundations. Average attitudes of strata are: -

East abutment; strike slightly north of west, dip 15^0 north. West abutment, strike meridional, dip $10\ ^0$ west.

The east-crestal section of the above anticline, i.e. that section occupied by the stream flat in the central apart of the dam site, is down faulted between a strong fault near and along the fold-axis across the face of the west abutment, and a series of sub-parallel smaller step-faults across the inner end of the eats abutment. The fault across the west abutment cuts off the "Parnell Grit" which dominates this locality, and down faults it to the east, below the scope of the work in this direction. It has a throw of from 9 m to 15 m, and is defined by a wet, pugged shatter-zone 600 mm to 900 mm wide which will require special attention during construction of the foundations.

The planes of the faults in the east abutment, and those of the many minor fractures which have been located during survey, are clean, close breaks which should not impose any constructional difficulties on the work. It should be borne in mind, however, that faulting in this country is so prevalent that there are probably further fractures in the unproven gaps between the test shafts. This is suggested, in some cases, by the anomalies in the positions of recognisable strata between adjacent shafts or bores.

While on the subject of faults, it may be noted that 366 m beyond the dam site to the east, "Parnell Grit" is exposed in road cuttings at RL 62 m. A synclinal axis lies 91 m beyond the dam site in this direction, but this folding alone is not sufficient in itself to account for the presence of the "Grit" in the position observed. Strong faulting is again suggested. There is evidence, however, that this must be safely clear of the dam site.

Weathering Characteristics – Foundation Conditions

Old Terrace Deposits

All the older, andesitic terrace deposits which so extensively blanket Waitemata "country" rock on location are now extensively decayed to "heavy" red-mottled clays and clay-grits of very high colloidal content. They therefore have low density, low shear strength, and such high shrinkage and swell values that not only are they quite useless for embankment purposes, but also they should be removed from the embankment site. A thickness of 6 m of material of this type caps the spurs in which the west abutment of the dam and the spillway will be located, down to RL between 18 and 26 m respectively.

Modern Terrace Deposit:

The modern stream terrace which occupies the central portions of the dam site differs from the older terraces in that its constituent materials, though still andesitic, are generally clean and fresh. Also, it rests everywhere on a hard, unweathered floor of Waitemata strata. The section developed here is as follows:-

Levels of Contacts.		Average thickness & range

Average RL of Range		
240 mm (- 500 mm to 850 mm)	Sandy silts. Grade to fine sandy grits or silty loams.	1.9 m (1.1 m to 2.9 m)
	Coarse, ill-sorted gravels, boulders up to 320 mm diameter, clean, free grit matrix.	1.85 m (0.4 m to 3.14 m)
-1.7 m (-0.6 mm to – 3.3 m)	Hard Waitemata strata	

NB Highest recorded H.W.St. 1.5 m (27/01/1944)

The sandy silts have practically no cohesion, and have such high capillarity and such detrimental compaction characterisations that they must be removed from the site. The gravels are very porous and permeable, but although uncemented they are compact, and their grading is so coarse that they undoubtedly have high shear resistance. There is no need, therefore, foe them to be stripped from the site of the cross-section of the embankment to ensure water tightness of the structure. Excavated materials will provide ideal material for embankment toe-drains.

In the marginal areas around the stream flat, both or either the sandy silts or the gravels grade locally into zones of dark puggy carbonaceous muds or sand, usually with timber or considerable vegetable matter. All material of this type must be removed and dumped. Along the embankment site north of the stream, the modern stream flat has cut back in some measure into the earlier low level terrace in which the sandy silts are largely decayed to silty or semi-plastic “rusty” clays. These also must be removed, but will be usable later in the bank itself.

The cut-off wall across the stream flat need to be extended only 1.2 m to 2.4 m into the hard Waitemata strata on which the gravels rest, depending on the relative perm abilities of the several members of the series encountered.

Normal Waitemata Country

In both abutments, Waitemata strata, whether under old terrace gravels or exposed at the surface, are completely leached or softened by weathering to depths ranging from 1.5 m to even 6 m before firm “papa” is encountered.

The average section in normal Waitemata country is transitionary through the following range of soil types:-

150 mm to 225 mm	Surface soil
600 mm to 1800 mm	Heavy plastic, yellow residual clays
900 mm to 2400 mm	Leached country; structure and texture of parent rock still apparent, but all original mineral matter converted wholly or in part to plastic yellow clays.

600 mm to 2400 mm	Soft “blue” country; typical “papa” softened but not oxidised or extensively kaolinised. Grades insensibly down into hard, completely unweathered “blue” country.

Normal Waitemata strata are highly foldspathic, with the result that their residual clays and leached zones are highly colloidal, and have such low shear strength and such detrimental swell and shrinkage characteristics that they should be removed from the site of the embankment. Cut-off walls should be extended through soft “blue” country, again for from 1.2 m to 2.4 m, into hard “blue” country.

The incidence of sandstone members in the Waitemata series has considerable effect on the depth of weathering. Thus in the east abutment in particular, the presence of several strong, water bearing beds of sandstone, badly drained by reason of their dipping back into the country , has induced particularly deep, local weathering. This will involve considerable stripping for embankment foundations, and close attention to details of drainage and cut-off wall foundations. In one test shaft, a band of soft wet, leached sandstone is located 4.8 m below the top of the “blue” country, 7.5 m below the surface.

Parnell Grit Country

Parnell Grit country dominates the west abutment, and is entirely different from normal Waitemata country. It’s typical ‘soil” profile is as follows:-

450 mm to 900 mm	Soil and residual clay
4600 mm to 6000 mm	Leached, rusty, oxidised grit, generally friable, soft and rubbly at top, but becoming progressively firmer, more compact, but still jointed in depth.

The underlying, unweathered “grit” is dark green or blue-grey in colour, very hard, tough, and compact, but generally more permeable than average “papa”. Water therefore tends to accumulate in its base, with the result that the underlying tight mudstones are usually softened or even thoroughly leached well beyond the profile of weathering in the “grit” itself.

Cut-off walls must be carried down into sound “blue-grit’ for approximately 1.2 m, but for the embankment foundations, only the relatively thin surface soil and clay need to be removed.

The dip of the country carries the “grit” well below the surface to the west, so that it will be encountered extensively in the bed of the spillway channel. The volcanic constituents of the unweathered “grit” are highly chloritised and serpentinised, with the result that after quarrying, this material breaks down to a gritty rubble which tests have shown to be a suitable constituent for portions of the main embankment. Excavated “grit” should therefore be retained with this in view.

Materials for the Embankment

It was early realised that none of the several materials available on location were suitable by themselves for use in an embankment of the proposed design, but that reliance would have to

be placed on proper mixing of certain selected types. Tests and economic considerations show that there are two soil-type groups available which are suitable for these purposes, and that those are conveniently located in two well defined areas, the potentialities of which have now been thoroughly tested. These areas and types are:-

1 Western borrow-pit area situated in the spurs immediately beyond (west) the spillway channel and dam site generally. Waitemata residual clays and leached materials.

2 Upstream borrow-pit area - the modern and more recent low level terraces forming the stream flats bordering Nihotupu stream above the dam site. Gravels, sandy silts, and clay silts.

It is estimated that, after allowing for the stripping of approximately 69,000 m^3 of undesirable material from the site embankment, the quantity of selected fill required in the bank will be of the order of 335,000 m^3. In the design analysed, this total is divided between a core-wall section requiring 61,000 m^3, and outer sections of different quality material involving the remaining 274,000 m^3. These quantities are available from the borrow-pit areas examined.

Western Borrow Pit Area – Core Material

As previously stated, normal Waitemata strata give rise to heavy, colloidal clays which in themselves are quite unsuitable for dam embankment purposes. Even the sandstone members which comprise 25% of the average Waitemata sequence give rise to residual clays which are practically devoid of material coarser than the fine sand group. It was hoped that this obvious deficiency in coarse fractions could be made good by mixing Waitemata residuals with high-level terrace deposits. The unexpectedly high degree of decay of these latter in depth has been found not only to rule out this possibility, but also renders their retention in any capacity in the work dangerous.

Test have shown, however, that use can be made of Waitemata residuals by mixing them with the sandy-silts which have to be removed in any case from the stream flat in the centre portion of the embankment site. Averaged samples from test shafts sunk through the Waitemata residuals in the western borrow-pit area, mixed with 40% of these sandy silts, gives a mixture with a grading within the limits prescribed for core-materials, and of such low permeability that the use of a concrete core in the structure would be redundant. Shear strength characteristics of this mixture are satisfactory (see No 24, table 2 appended).

A suitable core of the above mixture will call for approximately 38,222 m^3 of Waitemata residuals, and 26,756 m^3 of sandy silts. These quantities are available the former from the western borrow pit area, and the latter from selected strippings from the stream flat on the embankment site. In the case of the former, it will be necessary, in order to obtain the required quality of material, to prepare the site by stripping, and dumping in the nearby gullies, form 600 mm to 900 mm of heavy, colloidal surface clays and soils. The average haul from the borrow pit to site will be of the order of 460 m.

Upstream Borrow-Pit Area – Outer Section Material

The only source of material suitable for the semi-permeable and permeable outer sections of the embankment of the side slopes proposed, is the flats bordering the stream upstream from the dam site. Those comprise, in general, a relatively narrow modern flood plain adjacent to the stream banks, which is backed by, or merges into a more extensive older

terrace at a somewhat higher level. In the modern flood plain, the section through the terrace deposits is the semi-relatively fresh sandy-silts on fresh, ill-sorted gravels –as in the stream flat at the embankment site. In the older terraces, however, although the general section is the same, the silts over the gravels are more or less decayed to silty or sandy clays, or even to heavy plastic clays in marginal areas, and the grit-matrix of the upper parts of the gravels themselves sis usually also decayed.

Test shafts and trenches have been sunk throughout these stream flats to a distance of approximately 915 m u-stream from the dam site. It appears from these investigations that the quantities tabulated below are available form this area, it being assumed as a basis for estimation for these that a mixture containing at least 50% gravels will be required for stability. To attain this standard, an appreciable quantity of “heavy” clay silts and clays, particularly from the older terraces, will have to be stripped and dumped. Stripping also includes an allowance for excluding from the dumped mix certain local zones of puggy, carbonaceous muds and sands and timber debris which are always present in some measure in deposits of this nature.

Table 1

Zone	Area (m^2)	Average thickness (m)			Gross Quantities (m^3)		
		Silts	Gravels	Stripping	Silts	Gravels	Stripping
Flood plain and better parts of older terrace	54,343	1.8	1.8	0.45	99,378	99,378	24,462
Poorer parts of older terrace – marginal	52,670	0.9	0.9	1.20	40,515	40,515	53,511
Stream Bed	6,688	-	0.9	-		6,116	
Totals					139,893	146,009	77,973
					285,902	m^3 gross	

Tests carried out on this gravel / clay-silt mixture indicate that in spite of the porosity of the former (25% $\pm$), the latter are so compact in situ that the loss in gross bulk on working, mixing and compaction is only about 3 ½ %. Allowing 5% to cover variations in grading, it appears that the area tested can be expected to yield approximately 271,377 cubic metres of fill, in place in the embankment. This apparent deficiency of 2293 cubic metre on the estimated requirement of 273, 670 m^3 for outer section work can be readily obtained by extending the borrow-pit still further upstream, where similar, though more limited conditions are known to exist. It has already been mentioned, also, that Parnell Grit spoil from the spillway channel is expected to be suitable for incorporation in the outer sections of the embankment.

The average haul from the upstream borrow-pit area to the embankment is 610 m.

Certain inherent, disadvantageous working conditions hold throughout the stream flats. Ground level varies from 1.5 m to 4.6 m above the present stream level, and generally speaking, ground water-level is at the gravel-silt contact, between 1.5 m and 2.7 m below ground level. Wet conditions are bound to exist when quarrying the gravels, but these are free-draining, and there is sufficient grade in the valley floor to permit control of the stream to prevent flooding of the working face under normal conditions. Throughout the lower half of the borrow-pit area, however, this generalisation will hold good for the entire section only if a measure of control is applied to the storage lake above the temporary supply dam. Weir level of the latter is RL 3.286 m. The gravels rest on an irregular floor of hard “papa” which rises from an average level of RL 1.810 m at the dam to weir level 304m upstream. In the lower part of the area, therefore, working of the gravels will be restricted to such times as it may be permissible to empty the temporary supply lake. In any case, working any part of the borrow-pit during times when the temporary supply is being drawn upon is bound to seriously affect the quality of this supply. It should be noted that optimum moisture content for both gravels and clay-silts is high (35% to 40%), and therefore wet working conditions are far less serious than may appear at first sight. Mixing and compaction should be no more difficult than will be the case with the core-wall material, the Waitemata residual components of

which occur in situ, even during Summer months, with moisture contents well above their "optimum" of 28%, and will therefore have to be dried out before being compacted in place.

While working in the stream flats no particular difficulty should be encountered in generalising on a digging and placing programme to ensure a desirable, uniform increase in permeability from the core-section outwards, yet it must be admitted that the known and expectable variations in character of material, and the occurrence of lenses of undesirable material from place to place, will call for very close supervision if the aim is to be obtained. It is possible also, that in view of the high percentage of boulders over 150mm in diameter in some parts of the gravels, means may have to be taken to remove all such to facilitate proper compaction. These boulders are of hard, clean andesite, and it is suggested that in any case their removal from the mix for use for concrete aggregate of for other phases of the work may be economic.

The grading of the gravels is variable, particularly with regard to fractions of cobble or boulder size. The following analysis appears to be fairly typical.

Sieve Size	**% Retained**	**% Passing**
2 ½ "	60%	
2 ½ "		40%
1"		25%
3/8"		20%
¼"		15%
1/8"		10%
30 mesh		3.5%
50 mesh		3.0%
100 mesh		1.0%
200 mesh		0.2%

Test results:

Departmental reports on tests carried out by the soil survey laboratory, together with comments on those reports by my staff, are already on file.

Test results on samples and mixtures dealt with in the field laboratory are listed in Table 2 appended. Chief interest lies in samples No 24 and No 29, which are mixtures designed as closely as possible to simulate average conditions which can be expected, with the materials available, for core-wall and outer sections of the bank respectively. All results are the average of a considerable number of individual tests. This is necessary because of the many anomalies which are inherent in work of this nature, particularly when dealing with material with such relatively high and variable colloidal-content as is the case in this instance.

In order to expedite analysis, some tests are incomplete, only those required to evaluate "worst conditions" being finalised at this stage.

Shear-strength results for samples numbers 27 to 29 are considered to be very conservative, for, because of the limiting size of the shear-testing machine, it has to be assumed that only those fractions of the gravels which pass 1/8" mesh are effective in the soil mass, all coarser fractions being of the nature of "plumbs" only. It would be necessary to attempt tests with a

shear-box large enough to permit the incorporation in the test mix of fractions up to 1”, at least, to achieve results more consistent with actual practice.

The noticeable falling off of both cohesion and angle of internal friction in the case of saturated samples is particularly important and instructive. Here again, however, results are considered to be very conservative. Samples of compacted material are saturated by unrestrained uptake of eater from water saturated sand, in which they are imbedded to prevent crumbling. In practice, fill material in the body of an embankment will be under load, and swell and capillarity will be controlled, water intake will be limited, and such extreme shear-strength reductions will not be achieved. Practical difficulties in the way of carrying out tests to cover the varying conditions which will eventually obtain are, however, too great to warrant further refinements at this stage.

Slope Stability Analysis

The Swedish Method of analysis has been applied to an embankment comprising a core of mixture of Waitemata residual clays and sandy silts, and in the outer section of averal stream-flat material finished to an upstream slope of 1 in 3. In later analyses, allowance has been made for a measure of graduation in grading of outer-section material in conformity with the conditions which will be aimed for in practice.

It is apparent that the critical case in such a bank is at the maximum draw-down (RL 7.3 m). The clay content in the outer section remains fairly high for this type of structure, and therefore it must be assumed that it is quite possible for maximum draw-down to be attained with this outer material still essentially in a saturated condition due to its relatively low permeability.

On this basis, the bank is found to have a factor of safety of 1.4. In view of the conservative nature of the test results discussed in the previous section, and the extreme nature of the conditions assumed to apply at all stages in the analysis, the design of the embankment is considered to be safe, and within accepted limits for this type of structure.

Conclusions

As stated at the beginning of this report, the results of extensive investigation show that although much of the material on location is unsuitable and that in consequence appreciable revision of details of the original design for this work have been imperative, suitable materials have been located, and the work can be put through at cost of the order of the original estimate.

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Reliability Centred Maintenance (RCM)

Watercare Services Ltd is working to have a reliability centred maintenance programme covering all of the company's assets working for most assets by July 2009. From this will be developed the company's maintenance strategy and programme, and provide the input into the asset management plan. The project is currently working on the Headworks assets, having completed the foundation work at the treatment plants.

RCM is a procedure for determining maintenance strategies based on reliability techniques and encompasses well-known analysis methods such as:

- Failure Mode Effects and Criticality Analysis (FMECA).
- IEC 60300-3-11 Nov 1999
- SAEJA1011 Aug 1999 & SAE JA1012 Jan 2002

It is a framework based on international standards that facilitate a maintenance plan development to:

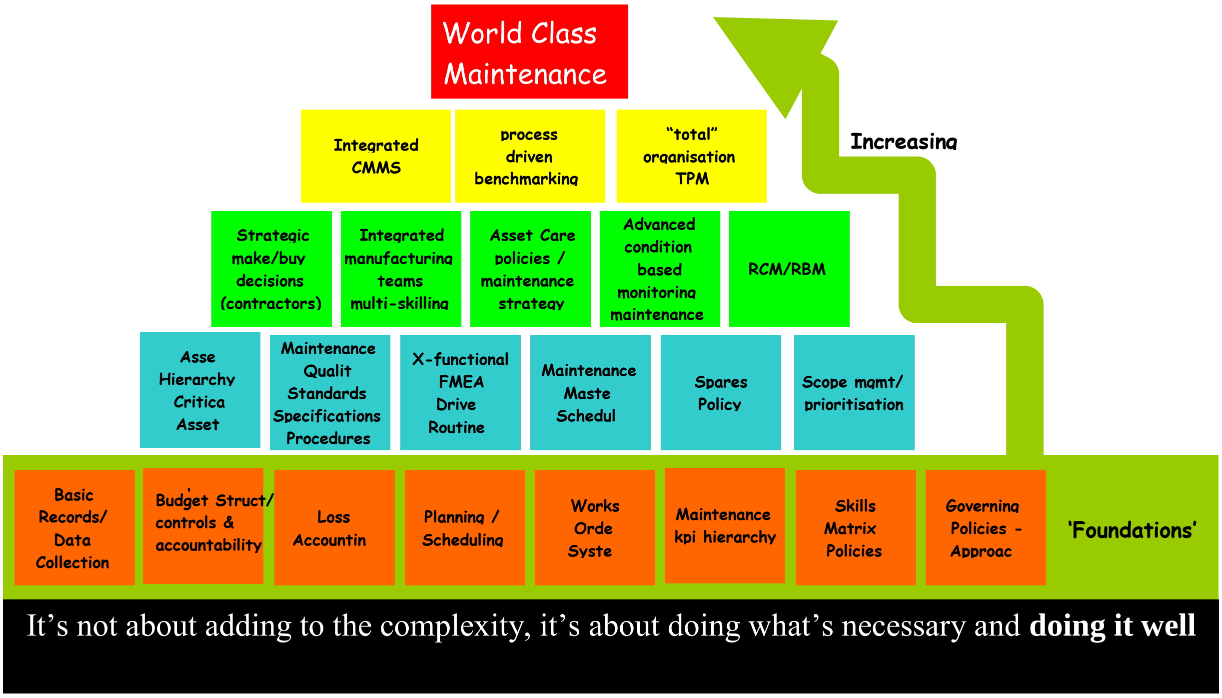
- Meet Health & Safety requirements;
- Meet environmental requirements;
- Meet operational requirements;
- Improve asset reliability and availability; and
- Minimise costs.

Watercare wanted a "blue print" based on best practice

Using the Kepner Tregoe decision analysis, Watercare determined that its maintenance strategy required:

- Consistent maintenance strategy;
- Critical assets identified and ranked in priority;
- Maximised long term sustainability of assets;
- Optimised cost of maintenance vs. cost of failure;
- Alignment to an international standard;
- Appropriate maintenance plans assigned to each asset (condition, performance, time based, run to failure); and
- Address Health & Safety, Environmental and Operational issues

Watercare's maintenance pyramid focus's on the foundations, first and then on building the infrastructure to regain control


World Class Maintenance
Increasing
Integrated CMMS
process driven benchmarking
"total" organisation TPM
Strategic make/buy decisions (contractors)
Integrated manufacturing teams multi-skilling
Asset Care policies / maintenance strategy
Advanced condition based monitoring maintenance
RCM/RBM
Asse Hierarchy Critica Asset
Maintenance Qualit Standards Specifications Procedures
X-functional FMEA Drive Routine
Maintenance Maste Schedul
Spares Policy
Scope mgmt/ prioritisation
Basic Records/ Data Collection
Budget Struct/ controls & accountability
Loss Accountin
Planning / Scheduling
Works Orde Syste
Maintenance kpi hierarchy
Skills Matrix Policies
Governing Policies - Approac
'Foundations'
It's not about adding to the complexity, it's about doing what's necessary and **doing it well**

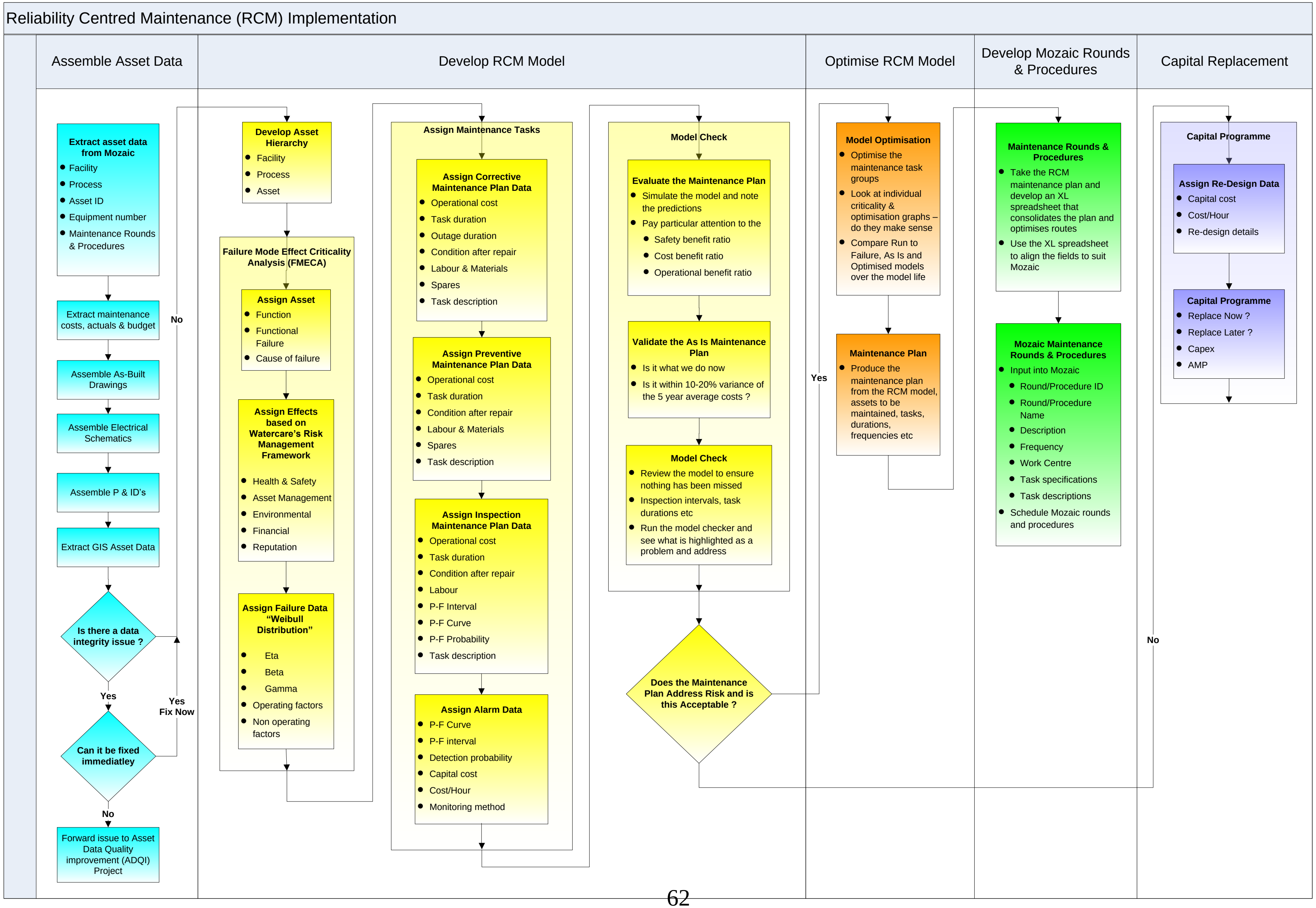

Reliability Centred Maintenance (RCM) Implementation
Assemble Asset Data
Develop RCM Model
Optimise RCM Model
Develop Mozaic Rounds & Procedures
Capital Replacement
Extract asset data from Mozaic
Facility
Process
Asset ID
Equipment number
Maintenance Rounds & Procedures
Extract maintenance costs, actuals & budget
Assemble As-Built Drawings
Assemble Electrical Schematics
Assemble P & ID's
Extract GIS Asset Data
Is there a data integrity issue ?
Yes
Can it be fixed immediatley
No
Forward issue to Asset Data Quality improvement (ADQI) Project
No
Yes Fix Now
Develop Asset Hierarchy
Facility
Process
Asset
Failure Mode Effect Criticality Analysis (FMECA)
Assign Asset
Function
Functional Failure
Cause of failure
Assign Effects based on Watercare's Risk Management Framework
Health & Safety
Asset Management
Environmental
Financial
Reputation
Assign Failure Data "Weibull Distribution"
Eta
Beta
Gamma
Operating factors
Non operating factors
Assign Maintenance Tasks
Assign Corrective Maintenance Plan Data
Operational cost
Task duration
Outage duration
Condition after repair
Labour & Materials
Spares
Task description
Assign Preventive Maintenance Plan Data
Operational cost
Task duration
Condition after repair
Labour & Materials
Spares
Task description
Assign Inspection Maintenance Plan Data
Operational cost
Task duration
Condition after repair
Labour
P-F Interval
P-F Curve
P-F Probability
Task description
Assign Alarm Data
P-F Curve
P-F interval
Detection probability
Capital cost
Cost/Hour
Monitoring method
Model Check
Evaluate the Maintenance Plan
Simulate the model and note the predictions
Pay particular attention to the
Safety benefit ratio
Cost benefit ratio
Operational benefit ratio
Validate the As Is Maintenance Plan
Is it what we do now
Is it within 10-20% variance of the 5 year average costs ?
Model Check
Review the model to ensure nothing has been missed
Inspection intervals, task durations etc
Run the model checker and see what is highlighted as a problem and address
Does the Maintenance Plan Address Risk and is this Acceptable ?
Yes
No
Model Optimisation
Optimise the maintenance task groups
Look at individual criticality & optimisation graphs – do they make sense
Compare Run to Failure, As Is and Optimised models over the model life
Maintenance Plan
Produce the maintenance plan from the RCM model, assets to be maintained, tasks, durations, frequencies etc
Maintenance Rounds & Procedures
Take the RCM maintenance plan and develop an XL spreadsheet that consolidates the plan and optimises routes
Use the XL spreadsheet to align the fields to suit Mozaic
Mozaic Maintenance Rounds & Procedures
Input into Mozaic
Round/Procedure ID
Round/Procedure Name
Description
Frequency
Work Centre
Task specifications
Task descriptions
Schedule Mozaic rounds and procedures
Capital Programme
Assign Re-Design Data
Capital cost
Cost/Hour
Re-design details
Capital Programme
Replace Now ?
Replace Later ?
Capex
AMP

Table 2 Shear Strength Determinations

Samples		Moisture Content		Maximum Density		Shear Strength			
						Cohesion		Angle Ø	
No.	Description	At Optimum	At Saturation	Dry	Wet	Optimum	Saturation	Optimum	Saturation
23	Average Waitemata residual material. Test shafts B2 – B9	27		93	119	*2200	600	*14^0	9^0
24	60% average Waitemata residuals & 40% sandy silts	29	35		116	*1910	500	*14 ½ ^{0}	13^0
25	Clay Silts – average type from stream flats	40	42		110	*1730	*1030	*11^0	9^0
26	Residual clay – leached mudstone 1205 on dam C.L.		40				*1000		8^0
27	Stream flats, shaft T.6. 14% gravels below 1/8" 85% silts and clay silts	39			111		230		10^0
28	Stream flats, shaft T5 5% gravels below 1/8" 95% sandy to clay silts	35			112	1590	590	22^0	12^0
29	Stream flats average sample all shafts 17% gravels below 1/8" 83% silts and clay silts.	38			108	1880	675	20^0	12^0

* Subject to alteration. Further test are in hand.