

Lower Huia Dam



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Lower Huia Dam

Summary of Dam and Main Elements

The Lower Huia Dam was constructed between 1967 and 1971, is located 16 km southwest of Titirangi and dams the lower regions of the Huia Stream. The dam is owned and operated by Watercare Services Ltd and is one of ten dams used for impounding water for Auckland's bulk water supply.

Embankment

The embankment is a 39.6 m high zoned earthfill dam with a crest length of 366 m and containing a total of 916,800 cubic metres of fill. The embankment has a central clay core with transitional and shoulder zones constructed from rubble.

The dam was designed as a rock fill embankment with a clay core and constructed between 1967 and 1971.

Materials

The 40 m high embankment contains a central core with transitional and shoulder zones employing rockfill, and filter and a drainage layers constructed downstream from the core.

Core

The core comprises stiff moist brown slightly gravelly silty clay, PI 30. $k \sim 1 \times 10^{-9}$ m/s. The core material was sourced from alluvial clays within the dam basin near the river.

Shoulders

The shoulder materials comprise a broad size range up to 300 mm with extensive zones of weathered tuff. The rock shoulder fill consists of andesitic conglomerate sourced locally from a quarry above the western abutment and. The “rock fill” was specified to be less than 300mm in size. Softer layers of sandstone were encountered during the quarrying operation and this material broke down to form a finer matrix.

Filter & Drainage layers

Formed by raking scoria delivered to site 0.3 to 20 mm graded to the filter and 20 mm to 35 mm graded to the drainage layer.

The 3.05m wide filter zone downstream of the core is described as graded gravel scoria or crushed hard rock. Gravel used in the 1.4m wide “drain” zone downstream of the sand filter zone was extracted from the riverbed upstream of the site.

Downstream face

A layer of clay and topsoil about 300 mm deep was applied to the face and vegetated with grass.

Upstream face

On the upstream face, the dam is protected by a layer of basalt rip rap.

The dam core finishes 600 mm approximately above the spillway. Higher flows may exceed the capacity of the drainage system.

The riprap layer is approximately 4.6m horizontal wide, comprising selected hard conglomerate from the quarry below 30masl and basalt boulders of approximately 60cm diameter above this level.

Embankment Vegetation

The downstream face of the dam and the refaced section of the left abutment are graded, and established with an appropriate grass cover that can be maintained by grazing sheep, augmented by mowing of the dry grass stalks in January/February each year if necessary. The grass is kept cropped to aid identification of any abnormalities.

Abutment Vegetation

The abutments are generally covered with dense native bush.

On the left abutment, the downstream face of the embankment was trimmed to match the configuration of the dam face.

Being natural ground excavated back to the current profile, this area is sometimes subject to minor areas of subsidence.

This area has the same grass cover as the dam face, and is generally indistinguishable.

Embankment Internal Drainage

A pervious drainage layer formed using 20 mm to 35 mm scoria was installed on the downstream side of the filter blanket to transport any seepage entering the filter blanket.

A reinforced concrete collector drain pipe 380 mm I.D. and perforated with 12 mm diameter holes at 250 mm centres runs along the base of the downstream chimney drainage zone, collecting seepage flow intercepted by the chimney drain and foundation drains. The flow is transferred to a 900 mm discharge pipe discharge into Manhole 1. A manhole chamber is located at each end of the collector drain, in the abutments.

A rock drain runs from abutment to abutment along the dam foundation interface beneath the downstream shoulder to intercept seepage not picked up by the collector drain. This rock drain discharges into a lateral "rubble rock" drain 4.5 m wide by 2.4 m deep. The rock drain collector meets a sump formed from 12 to 50 mm scoria. The sump measures approximately 10 m x 2.3 m high x 1.8 m wide. The water is collected into a chamber by a series of 10 mm

perforated concrete pipes 1.8 m long, laid at 530mm intervals along the chamber. The seepage flows into Manhole 1

A collector drain pipe 380mm I.D. and perforated with 12mm diameter holes at 250mm centres running along the base of the downstream chimney drainage zone transfers seepage flow intercepted by the chimney and foundation drains to a 900mm I.D. discharge pipe.

The discharge pipe conveys the water to a manhole just beyond the toe of the dam near the piezometer house, where this flow and flow from the underground dam in the old riverbed is discharged.

A rock drain runs from abutment to abutment along the dam /foundation interface beneath the downstream shoulder to intercept seepage not picked up by the collector drain. This rock drain discharges into a lateral “rubble rock” drain 4.5m wide by 2.4m deep. The lateral drain also appears to pick up any overflow from the main collector drain system. At the downstream end the lateral rubble drain terminates at a pipe discharge system known as the “West Abutment Drain”.

Embankment External Drainage

Runoff from the dam face is collected in shallow berm drains and transported into concrete sumps, and then down the face of the dam in RCRRJ pipes. The sumps were checked, and areas of porous concrete repaired during 2008.

Where seepages appear on the face of the dam, draincoil is laid in a bed of filter sand, and the water discharged to the berm channels.

There have been small areas of seepage appear from time to time on the face of the dam. In most cases, these have been put down to rainwater entering the downstream shoulder, and exiting at a perched water table within the downstream material.

There has been a consistent seepage exiting from the left abutment for a number of years. This is considered to originate from within the abutment and be driven by natural groundwater release, and not affected by rainfall or lake level.

CCTV inspection shows that a number of the closed pipe drains have cracked surfeits, which might allow water to escape during very high flows. So far, it appears that the repairs to the sumps has contained the loss of any surface water from the drainage system

Other Elements of the Dam

Spillway

An ungated free flow bellmouth spillway (morning glory) discharges through a spillway tunnel passing through the left abutment into a spillway dissipator

A bellmouth (morning glory) spillway is sited alongside the left abutment. The spillway is capable of passing the PMF flow without the dam overtopping.

The structure is a 21.4 m diameter reinforced concrete bellmouth spillway flowing into a 6.1 m diameter concrete lined tunnel laid through the left abutment into a spillway dissipater.

The spillway tunnel is located beneath the left abutment. The 6.1 m diameter tunnel leads from the base of the vertical shaft of the bellmouth spillway to the downstream stilling basin.

Spillway Dissipater

A stilling basin has been installed at the end of the spillway tunnel to dissipate the energy stored in the spill water. It discharges into the Huia Stream.

Valve Tower

A free standing 37m high valve tower is sited in the lake. This is a 5.13 m diameter reinforced concrete valve tower which contains water intakes, valves and screens at varying levels and two low level valves for scour and emergency dewatering. The scour valves discharge into the diversion supply tunnel. The water level recorder is also installed in the tower. An 88.5 m long reinforced concrete trestle bridge provides access to the tower from the right abutment.

The tower can be accessed by either a bridge extending from the crest and across the morning glory spillway, or by walking up the supply tunnel. The valve tower contains the four intake screens, intake valves, standpipe, and lake level recording system. The SCADA system is located on top of the valve tower.

The intakes feed into a 900 mm diameter standpipe, which in turn feed the 900 mm diameter supply main in the diversion tunnel.

Diversion / Supply Tunnel

The 3.05 m diameter diversion / supply tunnel leads from the valve tower to the downstream toe of the dam. It contains a 914 mm diameter supply pipeline and a 610 mm diameter scour pipeline.

Valve Tower Access Bridge

An 88.5 m long reinforced concrete trestle bridge provides access to the valve tower from the right abutment. The bridge crosses over the bellmouth spillway opening.

Free Discharge Valve

The supply intakes discharge into a 910 mm main and the scour intake into a 610 mm main. The two mains are interconnected by valves when they exit the supply tunnel.

The 610 mm main leads across to finish alongside the outlet from the spillway dissipater. The main is sealed off by a plate welded to the end of the main.

A 450 mm pipe connected off the scour main feeds a 450 mm diameter free discharge sleeve valve discharging to the stream bed below the spillway outlet.

The 450 discharge valve has limited capacity. In the event of a major hazard occurring to the dam, the end of the scour main would be gas axed off. Once opened, the scour main could not then be closed until the lake drained out.

The valve can deliver between 2.4 and 2.7 m³/s, depending upon where the water is drawn from. It is also dependent upon the lake level being full.

Upstream Rip Rap Facing

The upstream face is covered with an outer layer of basalt over the top of rip rap sourced from the Huia Quarry

Piezometers

Hydraulic piezometers were installed within the embankment during the construction of the dam. These are reliable instruments, provided they are deaired whenever they show signs of trending away from normal. No instrument is deaired unless necessary, as the process does stress the tubes.

Standpipe piezometers were installed to supplement the hydraulic piezometers in the early 1990's. The impermeable nature of the core material and the foundation result in their not being responsive.

In 2002, vibrating wire piezometers were installed within the dam to provide real time data that will be more responsive than the standpipe instruments.

Survey Monuments

Survey monuments have been installed to monitor vertical and horizontal movement of the dam.

Road Access

The only road access is along Huia Road, past the Lower Nihotupu Dam. In the event of the road access being blocked for any reason, access can be achieved by boat to Huia Bay, off loading at the wharf installed for the construction of the Upper Huia Dam.

Power Supply

The primary power for Huia is sourced from Scenic Drive, taken overland to Parau, and then transported across country to Huia.

There is only the one source of network power to Huia.

There is no back-up power supply to the Huia Dam. The valve tower is fitted with a connection to allow a portable generator to be fitted to the supply in an emergency.

If there is a major loss of power, then a containerised power generator will be fitted up to the pump station, capable of running the pump station, treatment plant, and the dam SCADA.

In the event of a total loss of power supply, trained caretakers are available and on call to manually take over the operation and surveillance of the dam.

Spillway Dissipater

The spillway dissipater has been built on compacted fill, following the removal of all peaty material.

Beyond the tunnel exit, the spillway apron widens out to 12.2 m while the invert falls away steeply for 10.4 m over a distance of 27.5 m. The base of the apron contains a non standard baffle arrangement.

The high sill at the end of the deep stilling basin precludes the need for adequate tail water to form a jump within the basin.

Communication

The local status reports and control commands are transferred to and from the site by SCADA using a radio bearer system. The radio bearer requires a power supply for operation. In the event of a loss of power supply, the caretaker will take over the surveillance, reporting and manual operation at the facility.

There is a telephone land-line connected to the site in the Huia Pump Station.

Cellphone coverage is generally nil except for an enhanced capacity inside the Huia Village water treatment plant.

Communications can be maintained with the radio telephone network. Radios installed in the Operations vehicles are self sufficient in a power outage. The repeaters have solar powered battery systems.

Catchment and Reservoir

The catchment area is some 1,430 ha and the reservoir area at top water level is approximately 55.7 ha. The dam impounds a reservoir of 6,660,000 cubic metres storage from a catchment area of 1,430 ha.

Geology

The dam is founded on soft weak sandstone of the Cornwallis formation. On the right abutment, lenses of Piha formation (an andesitic conglomerate) were detected in boreholes in the shoulder of the abutment.

Geological Setting

The Lower Huia Dam, located in the lower reaches of the Huia Valley approximately 1 kilometre upstream of the Manukau Harbour, is founded on interbedded volcanoclastic sandstone and siltstone of the Cornwallis formation (Waitemata Group) at lower Miocene age which dip at a low to moderate angle to the west. These fine grained sediments grade up into coarsely stratified andesite breccia-conglomerates with minor grit, sandstone and

siltstone mapped as Piha formation of the Manukau subgroup by Hayward (1983) (Ref 1). At the dam site the Cornwallis/Piha Formation contact occurs just above the dam crest level on the right abutment.

The Lower Miocene rocks in the Huia Valley are divided into a number of tilted or gently folded blocks produced by a network of ENE and WNE striking faults. Generally, many of the blocks are tilted to the NW with low overall west and northwest dips. However, steeper dips occur near fault boundaries where bedding is strongly deformed by faulting.

Dam Foundations

Investigation records show that the dam is founded on soft weak sandstone of the Cornwallis Formation, which in construction photographs, appears to have a moderate to steep dip near the intake tower. Investigation boreholes on the right abutment located lenses of the Piha Formation, an andesitic conglomerate, on the shoulder of the abutment.

Inspection of aerial photographs SN 1924/9 and 10 flown prior to construction show uneven and hummocky topography indicative of past land stability high on the left abutment ridge extending towards the stream on the downstream side of the ridge. During construction of the downstream portal of the spillway tunnel, these areas became unstable when undercut, as did areas higher on the abutment.

Excavation of the dam foundation was apparently trouble free with the exception of a “soft” area at the base of the ridge forming the left abutment. This “soft” area required extensive sub excavation to remove weathered material leaving a substantial sized hole and steepened batters down from the left abutment.

Left abutment

Geological mapping associated with this study shows that the sandstone with interbedded siltstone dips between 20°W to 40°W on the left abutment (out of the slope) north to the intake tower with much of the slope above the dam and reservoir level apparently formed by bedding surfaces. Undercutting of this surface could lead to localised planar failure involving this slope. Earthquake shaking would have the same effect and this is a threat to the intake tower.

Localised instability (creep) has occurred high on the left abutment along the embankment/shoulder contact where regrowth vegetation (manuka) is deformed. This creep appears to be shallow seated and may be related to the pre construction instability.

More significant creep has occurred lower on the left abutment necessitating local repair and installation of horizontal bored drains at about the level of the crown of the spillway tunnel to drain the abutment. This area of instability is likely to coincide with the area of the hummocky surface recognised in the pre construction photographs. The remedial works appear to have been successful and the drains were dry on the day of inspection.

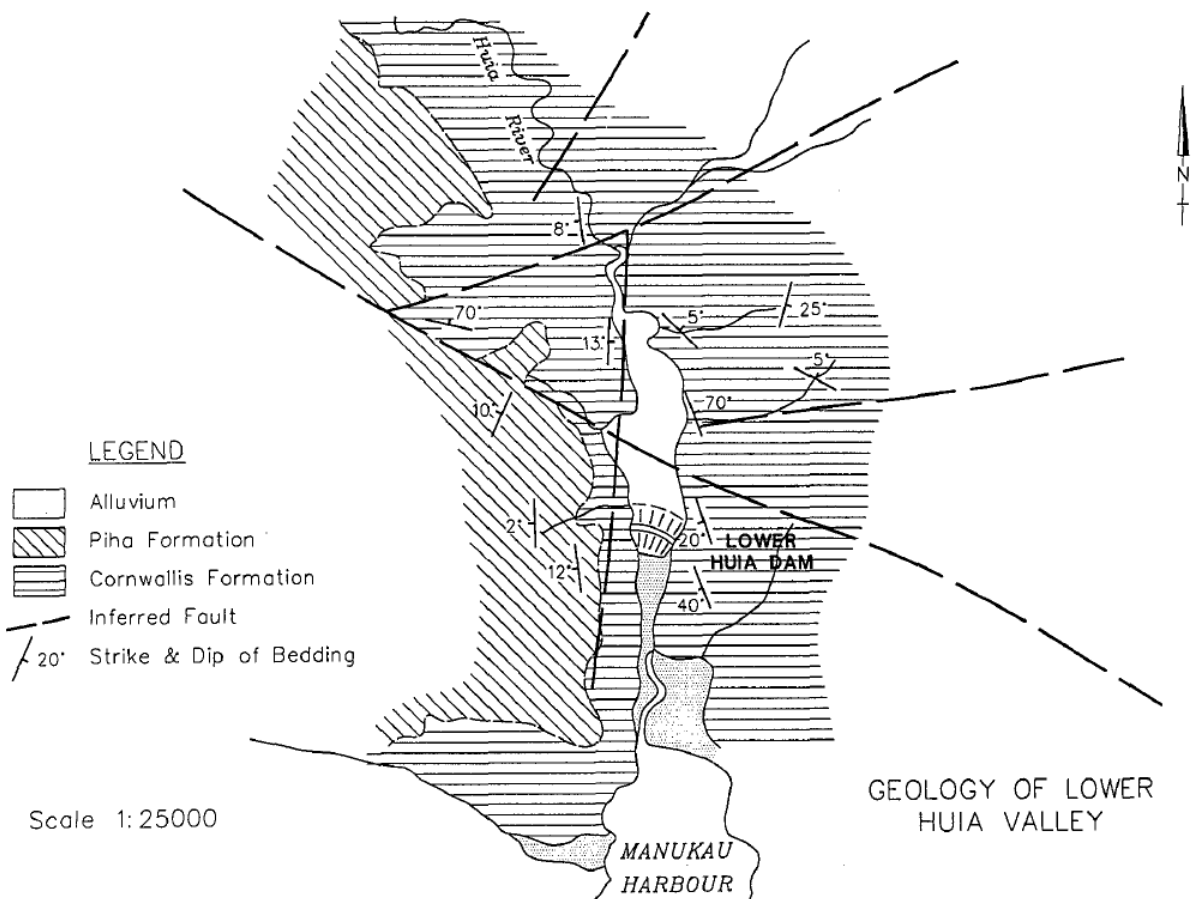
Right Abutment

There is no evidence of instability on the right abutment with both topography and vegetation being under formed. The embankment/abutment contact appears dry and stable. However, at the base of the dam, a large fill area on the west side of the valley floor appears to be very wet. There is no clear indication of where the water is coming from, although it is possible that the lenses of Piha Formation mentioned previously could be a source. Options

include run-off from the old quarry area above the abutment or from seepage under the dam foundation along the course of the original stream bed, which may be the cause of the “soft” areas mentioned above.

Reservoir Basin

An inspection was made of the reservoir margins at a time when the reservoir was almost full. As a consequence, little could be seen of the drawn-down slopes. Generally, forest vegetation extends down to the water line with the exception of the right (west) side of the reservoir where a drowned old road is located near the reservoir edge. Batters cut for this drowned road has fretted locally causing small, localised, areas of drop outs. Elsewhere along the reservoir margins the slopes above the water line are stable with the exception of the steeper slopes on the left side near the intake tower and bellmouth spillway, as described previously. The slopes above the reservoir adjacent to the intake tower/bellmouth spillway more or less reflect the dip of the underlying sandstone/siltstone bedding and are showing signs of minor planar instability along the reservoir margins due to wave undercutting. These slopes should be inspected when the reservoir is drawn down to assess stability. Apart from localised areas of potential instability (e.g. near left abutment) little risk is seen of substantial slope failure displacing large volumes of water and impacting on dam safety.



Surveillance and Instrumentation

Introduction

Lower Huia Dam is instrumented by hydraulic, standpipe and vibrating wire piezometers, deformation survey markers, inclinometers and seepage weirs. The vibrating wire piezometers and seepage weirs are read by the SCADA system and entered into the PI data base at 8 hourly intervals, or earlier if changes occur. The standpipe and hydraulic piezometers are currently read on a monthly basis, however their reading frequency has varied from weekly to fortnightly over their history their recordings are put onto a Psion data logger, and sent via the internet to Damwatch. Deformation surveys and inclinometer readings are undertaken on an annual basis by sub contractors and forwarded to Damwatch. Reservoir level and rainfall data are scanned at 15 minute intervals and any changes are entered into the PI monitoring database. Seepage is monitored at 15 minute intervals and entered into the PI database through SCADA.

Instrumentation	Beginning of Records
Hydraulic Piezometers	24 March 1972
Standpipe piezometers	10 February 1994
Vibrating Wire Piezometers.	11 January 2002
Manual weir flow recordings	13 June 1977
Automated weir flow recordings	24 May 2002

Rainfall Data

Rainfall data has been added to the dam database since 26 September 1978. earlier data is stored on Watercare's Hydrol database. The data used in the dam safety database is currently non verified data from the PI database, using data collected through SCADA. Where there is issue with the data provided, verified data can be downloaded from the Hydrol database and entered into Damwatch's database.

The data forwarded from the PI database to Damwatch is the daily total recorded at midnight. This information is available in real time on Watercare's In Touch system, and from the PI database at Watercare.

Obtaining verified data takes more time as a the Data Technician has to travel to site, download the data logger attached to the raingauge, and verify the data recorded against the check gauge installed on site.

Piezometer Monitoring

Twelve twin tube hydraulic piezometers were installed during the construction of the dam. Recorded monitoring began in 1977.

Twenty standpipe piezometers were installed in 1999.

Fifteen vibrating wire piezometers were installed on site in 2001/2002. These instruments were installed with grout. An earlier review considered that the lack of response from some

instruments was due to the grouting method. Further review by Damwatch and Watercare concluded that the lack of response is due to the instruments being located in low permeability materials.

Seepage Monitoring

Seepage is measured from two drainage systems, known as “West Abutment” and “Main Drain”. The West Abutment drainage system is the pervious rock drain that lies in the foundation between the left and right abutments. Another pervious rock drain intercepts the east-west lying drain and conveys water to a V-notch weir. The Main Drain is the 900 mm diameter pipe connecting the dam’s chimney drain to a separate V-Notch weir.

Three seepage weirs are located in a manhole at the toe of the dam adjacent to the old river channel. The reading of these weirs is automated by the SCADA system, and flows are recorded at 15 minute intervals, with the average daily recording being forwarded to Damwatch on a Saturday.

Two drains are located within the diversion tunnel. These are positioned on the left and right side of the tunnel, near the downstream portal. The drains measure leakage through the tunnel lining. The monitoring was automated, and the data collected on SCADA until the flows almost ceased, and now a qualitative system of measurement is used.

Deformation Monitoring

Dam deformation monitoring consists of six monuments on the embankment and four on the left abutment that are observed annually for vertical and translational movement. Forty crest markers are observed annually for vertical only movement. There are two inclinometers installed in the left abutment that are observed annually.

Potential Impact Classification of the Lower Huia Dam

Because of the consequent consequences resulting from a failure of the Upper Huia Dam, the consequences of the two dams are considered together..

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.

For the dams with high PIC listed above there is not immediate need to determine ‘rainy day’ PIC as it will not influence the initial requirement of the Building Act (2004) dams.

Introduction

The following text documents the basis for assessing the potential impact classifications of the Upper and Lower Huia dams.

The Upper Huia is a concrete gravity dam, with a straight crest 166m long and a maximum height of 37m. The dam was completed in 1929 as a monolithic structure without contraction joints.

The Lower Huia Dam is a zoned earthfill embankment dam with a chimney drain. It is 39.6m high and was completed in 1971.

The PIC determination utilises information from Huia Dams, ‘Report on Dam Break Analysis’ by Technical Services, Treatment Process Section, Watercare Services dated April 1993.

The area downstream of the Huia Dams 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 Upper and Lower Huia Dams in accordance with the Proposed Regulations to the Building Act 2004 and also the NZSOLD Dam Safety Guidelines.

Dam Break Study

Complete or partial collapse of the Upper Huia Dam during a major seismic event (‘sunny day’ failure), was considered the most likely failure mode for the dam by Watercare in their study.

This was assumed to occur with the reservoir at the maximum operational level of RL 166.22m.

Overtopping and piping failures were considered the most likely failure modes for the Lower Huia Dam. They were assumed to occur with the reservoir at the maximum operational level of RL 42.1m. A cascade overtopping failure of Lower Huia Dam following the failure of Upper Huia Dam was also modelled.

All together six failure scenarios were considered in the report which are summarised in Table D1. All are “sunny day” scenarios. No “rainy day” scenarios were included. Both a total collapse and partial collapse of Upper Huia Dam were considered, with and without subsequent cascade failure of Lower Huia Dam. One scenario considered a failure of the Lower Dam on its own.

Four scenarios assumed upper dam failure with subsequent failure of the Lower Dam by overtopping and erosion.

Table D1: Summary of Failure Scenarios

Scenario	Upper Huia Dam		Lower Huia Dam				Roughness coefficients adopted
	Failure Mode	Peak Breach Outflow (m ³ /sec)	Failure Mode	Peak Breach Outflow (m ³ /sec)	Breach Dev time (mins)	Time of Peak Flow (mins)	
1	Total Collapse	10,000	Overtopping & erosion	8,000	25	38	Lower limit
2	Total Collapse	10,000	Overtopping & erosion	7,950	25	40	Upper limit
3	Total collapse	10,000	Overtopping & erosion	2,700	90	67	Lower limit
4	Partial collapse	7,000	Overtopping & erosion	8,000	25	39	Lower limit
5	Total Collapse	10,000	No Failure	N/A	N/A	N/A	Lower limit
6	No Failure	N/A	Piping	6,200	25	25	Lower limit

Scenarios 1 to 4 were chosen for detailed analysis. Scenarios 1, 3 and 4 provided the upper and lower bounds for discharge and timing of the flood. Scenario 2 was considered the most appropriate scenario for the assessment of flood inundation.

A summary of the modelling results for Scenario 2 is shown in Table D2.

Distance from dam (km)	Time of max. flood (mins.)	Velocity of dam break flood front v (m)	Dam break flood depth d (m)	Houses flooded	d x v	Comments
Upper Dam	0		RL 152.4 d=19 m	0		
Pipe Bridge 3.9 km	6-9	5-10	RL 55.4 d=14 m	0		Bridge destroyed
Georges Bridge 4.8 km	7-10	15 m/s	RL 45.7 d=10 m	0		
Lower Dam 7.0 km	39-68	1 m/s	RL 16.2 d=10 m	0		
D/S Dam 7.7 km	40-70	10 m/s	RL 16.1 d=15 m	5	> 1	Bridge damaged, abutments destroyed.
Huia Bay 8.8 km	41-71	10 m/s	RL 4.4 d=4 m	1 to 5	> 1	Public amenities destroyed. Road damaged.

Upper Huia Dam “sunny day” Assessment of PAR and Fatalities

In scenario 2, total collapse of Upper Huia Dam, water levels in the reach immediately downstream rise up to 20m above the stream bed level. The flood water would run up McQuillan’s Stream Valley for about 500m. The water supply line from the Upper Huia Dam that crosses the Huia Stream at the Crusher Pipe Track would be destroyed. The supply line of Lower Huia and the Smiths Bridge (5 km downstream of the dam) would be severely damaged.

Significant environmental damage would occur in the Huia Valley where the flood wave would strip the lower valley vegetation. There is no habitation between the dams that could be flooded.

The dam break flood would enter the Lower Huia Dam reservoir and overtop the dam causing cascade failure. The valve house, access road bridge and some five houses along the Huia Dam Road would be washed away. The Huia Road Bridge crossing the Huia Stream near its exit into Huia Bay would be extensively damaged and its abutments destroyed. The public amenities on the beach foreshore would be destroyed and extensive damage done to the beach environment. The only house on the sea side of the road at the east side of the bay would be flooded in excess of 300m as would at least one other house of the three other low lying houses on the beach frontage as a flood wave between 3m and 6m high is dissipated in Huia Bay.

Public on the beach frontage would be at risk.

The Population at Risk is therefore assessed as 11 to 100. Fatalities are expected.

Upper Huia Dam “sunny day” Assessment of Socio-Economic and Financial Impacts

A map showing model cross sections along with a table of peak inundation levels were used to estimate impacts for Scenario 4.

Immediately downstream of Upper Huia Dam a swathe of bush and sections of walking tracks will be inundated by the dam break wave. The cost of repairing or replacing damaged access tracks is estimated at up to \$100,000. Bush along the stream banks would be swept away.

Lower Huia Dam would be overtopped and fail. Downstream of Lower Huia Dam damage to roads, two road bridges and public facilities is assessed in the range \$1 to 10m.

Upper Huia Dam “sunny day” Assessment of Environmental Impacts

There will be heavy ecological damage and environmental damage requiring years to recover with Major impact.



Figure D1: Location of Huia Dams and Downstream Floodplain.

Upper Huia Dam “sunny day” PIC Determination

With a PAR of 11 to 100 and Major environmental and socio-economic impacts the PIC of the Upper Huia Dam is determined as High in accordance with the Building Act (2004) and Proposed Regulations.

The PIC determined in accordance with the NZSOLD Guidelines is High as fatalities are expected.

Upper Huia Dam “rainy day” PIC Determination

A “rainy day” dam failure was not considered or modelled in the Upper Huia Dam break study.

Therefore no “rainy day” PIC been determined, however, this scenario will not influence or change the PIC category determined as high in Section 1.6 above.

Lower Huia Dam “sunny day” PIC Determination

The piping “sunny day”, failure scenario for the Lower Huia Dam was not modelled so the downstream effects of a single failure of the Lower Huia Dam are not known. The only scenario that considered a failure of the Lower Dam on its own (Scenario 6) had a peak breach outflow of 6,200 m³/sec; however, no inundation map was available. Compared with Scenario 2 above (7,950 m³/sec) this peak outflow is likely to result in significant impacts downstream.

Therefore, while a comprehensive assessment was not possible due to lack of detailed dam break modelling results, an intermediate assessment based on 3 or more houses being

inundated in a “sunny day” scenario with high velocity flow, PAR is in the range 1 – 10 and fatalities would be likely. Similar socioeconomic and financial cost would be caused to that assessed for the cascade scenario considered above. Accordingly the “sunny day” PIC determined for Lower Huia using an intermediate level of assessment is determined as High in accordance with both the proposed regulations and NZSOLD Guidelines.

Lower Huia Dam “rainy day” PIC Determination

A “rainy day” dam failure was not considered or modelled in the Lower Huia Dam break study.

Therefore no “rainy day” PIC can be determined. It is recommended “rainy day” scenarios be modelled and a “rainy day” PIC be determined for Lower Huia Dam.

Summary

The assessment of PIC for Upper and Lower Huia Dams is summarised in Table D3.

Table D3: PIC Assessment for Huia Dams

Dam	Socio-economic, financial and environmental impacts					Impact on people			Overall Dam
	NZSOLD and Building Act				NZSOLD		Building Act (Composite Socio – economic PAR assessment)		
	Facilities	\$ value	Environmental	PIC	Fatalities	PIC	Population at risk (PAR)	PIC	PIC
Upper Huia “sunny day”	Water supply mains (2) Lower Huia Dam bridge	\$1m - \$100m (excl. Lower Huia & pump house)	Major	High	Fatalities	High	17	High	High
Lower Huia “sunny day”	Water supply mains (2) Lower Huia Dam bridge	\$1m - \$100m (excl. Lower Huia & pump house)	Major	High	Fatalities	High	1 - 10	Medium	High

The “sunny day” PIC for Upper Huia Dam is determined as High. The “rainy day” PIC for Upper Huia Dam could not be determined as insufficient dam break modelling has been done.

Similarly, “sunny day” PIC for Lower Huia Dam, based on an intermediate level evaluation is determined as high. The “rainy day” PIC for Upper Huia Dam could not be determined as insufficient dam break modelling has been done.

Failure Mechanisms

A formal Failure Effects Modes Analysis was undertaken in March 2010. This was undertaken in conjunction with the 2009 CDSR.

Potential Failure Modes

Potential Failure Mode 1: Overtopping Failure due to Blockage of the Spillway

Potential Failure Mode 1 is an overtopping failure of Lower Huia dam due to blockage of the bellmouth spillway by a tree slide within the reservoir catchment or by a landslip on the left abutment slope above the spillway during a large flood event.

Spillway

The Lower Huia free overflow spillway is a 70ft (21.3m) diameter bellmouth leading to a 20ft (6.1m) tunnel with sharp 90° bend. With no blockage the bellmouth spillway can safely pass the Probable Maximum Flood (PMF) with 0.9m freeboard to dam crest. Overtopping would occur if the throat of the bellmouth were blocked by more than 30% or the rim of the bellmouth by more than 45% (Tonkin & Taylor 1993).

There are two threats to the performance of the spillway: tree slides within the reservoir catchment, and the slope above the bellmouth.

Left Abutment Slope Instability

Areas of instability were present above the left abutment prior to dam construction and some instability was evident during construction of the downstream portal of the spillway tunnel. Some shallow creep is evident around the left abutment and drains have been installed around the lower left abutment. Geologically mapping as part of the Stage I Safety Evaluation identified an adverse dip on the left abutment of 20°W, out of the slope and into the reservoir (Tonkin & Taylor 1993).

Two inclinometers were installed in boreholes above the areas of potential instability in 1994 as part of the Stage II Safety Evaluation (Tonkin & Taylor 1994), one in the slope left of the spillway bellmouth (BH3) and one downstream (BH4). Inclinometer reading commenced on an annual basis from 2001, with movement reference to the 2001 base reading. Movement has been limited to 46mm at BH4, in the top 2m of the borehole (Damwatch 2010).

The stability analysis of the left abutment slope undertaken as part of the Stage II Safety Evaluation indicated a marginal Factor of Safety (FOS) of 1.18 with a pore pressure ratio of 0.4, equivalent to near fully saturated soil (groundwater level 2m below ground surface). It would require an extreme weather event to fully saturate the slope and worse geology than known to produce a FOS less than 1.0.

Tree Slide within the Reservoir Catchment

The Lower Huia reservoir catchment is heavily vegetated by native trees large enough to block the throat of the bellmouth. The spillway log boom is located close to the spillway and is exposed to the longest fetch and prevailing wind. The log boom would not be expected to function under the load of a major tree slide and would contribute to the debris volume that could potentially block the spillway. The clearance between the underside of the valve tower bridge beams, which cross over the top of the bellmouth, and the crest of the spillway is approximately 7ft (2.1m), potentially increasing the risk of blockage at the rim of the bellmouth.

There have been no major tree slide events reported in 40 years of service. It would require an extreme weather event to produce sufficient debris and flood flows to overtop the dam, and the reservoir is commonly drawn down to meet summer water demand at the same time as the cyclonic flood season (February-March).

Risk Reduction Measures

The key performance parameters for the Potential Failure Mode 1 are:

- Incoming storm warnings,
- High lake level or lake level exceeding the anticipated rate of rise,
- Reservoir surveillance and vegetation management,
- Inclinator monitoring,
- Spillway surveillance and clearing of the log boom.

Possible actions to reduce risk include:

- Regularly clearing dying trees from the lakeshore,
- Regularly clearing the log boom,
- Characterising the historical generation of debris,
- Incorporating an auxiliary spillway,
- Changing the boom geometry to improve its effectiveness at diverting debris away from the spillway, taking into consideration the prevailing wind direction,
- Engineering a weak link positioned such that the chord(s) of the log boom is/are directed away from the spillway on failure,
- Developing a tell-tale monitoring procedure for the left abutment slope,
- Planning to draw down the reservoir if movement is detected on the left abutment slope.

Potential Failure Mode 2: Overtopping Failure through Crest Erosion

Potential Failure Mode 2 is an overtopping failure of Lower Huia Dam due to high lake levels flowing through the permeable gravels overlying the top of the core and resulting in crest erosion.

Crest Details

Four boreholes were drilled along the embankment crest to determine the top level of the core wall as part of the Stage II SEED Investigations. All four boreholes encountered sandy gravel transitional to gravely sand overlying the embankment core. The fill material appears to be similar in composition to the material reported to be used in the downstream shoulder. There is uncertainty about the minimum level of the core. Borehole logs indicate that the top of the core varies from 2.0m to 2.9m below crest level; however there is disagreement as to whether BH8 is the top or the sloping face of the core (Watercare 2009). This report assumes a reduced level of 41.59m. Watercare drawing 130.16-R4 indicates a wide coarse zone downstream of the gravel topping above the core, up to 40' (12m) deep, refer Figure 2.

If this coarse zone exists then there is potential for fines migration into the coarse zone impeding drainage, resulting in seepage exiting the downstream face rather than through the drainage system.

Historical Overtopping of Core

There has been one event on record when the lake level exceeded the top of the core; when the level rose 670mm above spill level on 1 January 1989 following a severe storm in the Waitakere ranges (Watercare 2009). The high lake level resulted in overtopping of the core by 620mm and subsequent overtopping of the underdrain flow monitor and deposition of sand behind the weir. In July 1998 the Caretaker observed a large flow from the rubble drain (300l/min) when the reservoir level was RL41.68, and was considered to be associated with heavy rainfall. The Caretaker also reports often removing sediment from behind the monitoring weir. The minimum freeboard to the crest during Probable Maximum Flood (PMF) will be 0.9-1.0m, however the PMF will overtop the core by 2.17m (Pickford 2010).

There is a low likelihood of exceeding Full Supply Level (FSL) and there has been no surface evidence of crest erosion from the regular surveillance of the dam to date.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 2 are:

- High lake level,
- Crest inspections,
- Seepage on downstream slope,
- Increased chimney drain and rubble drain flows.

Major uncertainties dominate the assessment, including the height of the core, the properties of core capping and the presence of a coarse semi-pervious shoulder rubble zone, the absence of historic data for the chimney drain at high lake levels, and the unknown properties of the filter and drainage zone.

Possible actions to reduce risk include:

- A records search to improve knowledge of dam materials, construction and performance history,
- Investigation of crest materials and reassessment of risk,
- Determining the depth to the core using Ground Penetrating Radar (GPR),
- Trenching along crest and raising the core,
- Alarming the chimney drain and foundation weirs.

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

Potential Failure Mode 3 is an overtopping failure of Lower Huia Dam due to failure of Upper Huia Dam and resultant high inflows to Lower Huia reservoir.

Dambreak Analysis

The *Huia Dams Report on Dam Break Analysis* was carried out by Watercare Technical Services in 1993. The dambreak study was reviewed in the *Potential Impact Category Assessment for Watercare's Dam* in 2006 and *Classification of 15 Dams Owned by Watercare Services Limited* in July 2010 by Damwatch.

The peak overtopping flow for the lower dam for a “sunny day” cascade failure was determined to be 320m³/s, equating to about 0.7m overtopping of the dam crest for about 10 minutes (Watercare 1993). The analysis assumed full lake level at the Upper and Lower reservoirs, full functionality of the Lower Huia bellmouth spillway and undermining of the wave break on the crest of Lower Huia Dam because the impervious clay core does not extend to crest level.

The Lower Huia reservoir catchment is heavily vegetated by native trees large enough to block the throat of the spillway, refer Section 5.1.1 and 5.1.3. The depth of overtopping would be expected to increase due to the likelihood of a tree slide associated with the Upper Huia Dam floodwave blocking the bellmouth spillway.

Borehole logs indicate that the top of the core of Lower Huia Dam varies from 2.0m to 2.9m below dam crest level, refer Section 5.2.1. The duration of overtopping of the core would be expected to be greater than 30 minutes based on the modelling results in the dambreak study.

Therefore the assumption that Lower Huia Dam fails as a result of a “sunny day” failure of Upper Huia Dam is reasonable, based on the current level of information on the construction of Lower Huia Dam.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 3 are:

- Failure of Upper Huia Dam, and
- High lake level at Lower Huia Dam.

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

Possible actions to reduce risk include:

- Incorporating an auxiliary spillway
- Confirming the depth to the core and the presence of a 'coarse zone' above the core,
- Trenching along the crest and raising the level of the core and reinstating the wavebreak wall,
- Reducing the top operating level of the dam,
- Planning to draw down the reservoir if there is evidence of impending failure of Upper Huia Dam, and
- Changing the spillway boom geometry to improve its effectiveness at diverting debris away from the spillway, taking into consideration the prevailing wind direction, and engineering a weak link positioned such that the chord(s) of the log boom is/are directed away from the spillway on failure.

5.4 Potential Failure Mode 4: Piping of the Core into Incompatible Filter

Potential Failure Mode 4 is piping of the core into the wide coarse zone of semi-pervious shoulder rubble indicated downstream of the gravel topping above the core.

Core Details

The core is a relatively narrow core, with an approximate top width of 15' (4.6m). Drill hole logs from the Stage II Safety Evaluation did not report any significant problems such as water loss or cavities, and describes the core material as "stiff moist brown slightly silty clay" (Tonkin & Taylor 1994).

The drill hole log for BH8 indicates gravel was encountered at the foundation contact at RL11.25m. Installation records indicate the depths of standpipe piezometers P20 and P21 are 11.1 and 9.5m respectively which conflicts with the full depth of piezometers monitored as P20 and P21 were measured in 2005 as 8.8m and 11.1m respectively. The piezometer installed near the gravel contact records a water level 7.5m below top lake level and is very slightly responsive to changes in lake level, the piezometer installed in the core has recorded dry since 2008, when it is suspected that gravel was placed down the standpipe.

Potential for Piping to Occur

Soil test results during construction indicate the following mean Atterberg-limits, based on six tests only:

- Liquid Limit LL=93

- Plastic Limit PL=57
- Plasticity Index PI=36

The report noted that there was difficulty in obtaining a consistent Liquid Limit on a given sample, and that the results would settle down to a reasonably consistent value after approximately ten tests (Watercare Data Book Volume 1). Under the Unified Soil Classification (USC) the embankment core material is classified as MH, an inorganic silt with a liquid limit higher than 50% (Terzaghi 1996).

For a cohesive soil with a $PI > 7$ the likelihood of backwards erosion and suffusion is negligible under the seepage gradients which occur in a conventional dam. The piezometers installed within the core of the dam show generally homogeneous seepage conditions along the length of the dam, stable hydraulic gradients through the core and stable seepage conditions. Pressures beneath the core are similar to pressures recorded in the core immediately above the foundation contact, indicating limited seepage across the boundary, as indicated by six piezometers located on three cross sections through the dam. Therefore piping can only occur through internal erosion through crack formation or discontinuity.

The 2006 GNS seismic hazard report recommended a peak ground acceleration of 0.19g Peak Ground Acceleration (PGA) for the Maximum Design Earthquake (MDE) equivalent to the 1 in 10,000 AEP. The probability of earthquake induced transverse cracking at the MDE is assessed as very low (1:1,000 probability with maximum likely crack width 5mm). The probability of initiation of erosion from a 1mm crack in a MH soil with a hydraulic gradient of 2 is 0.3 (in comparison, the probability of initiation of erosion from a 1mm crack in a CH soil is 0.05). The probability of erosion initiating from a 5mm crack in a MH soil is 0.9. Sandy gravel material over the top of the core reduces the risk of desiccation cracking.

The dam is constructed on a well shaped foundation with no conduit penetrations. There is no evidence of the presence of crack inducing features, such as step changes in the foundation, conduits, or abrupt changes in the foundation, with the exception of the extensive sub-excavation that was required at the base of the ridge forming the left abutment to remove weathered material, resulting in a substantial sized hole and steepened batters down from the top of the abutment (Tonkin & Taylor 1993). There is an extremely low likelihood of transverse cracks being present at foundation level and extending through the dam to connect with the lake due to the favourable cross valley foundation profile. There are some uncertainties in the exact treatment of the foundation, and borehole BH8 possibly indicates gravel at the foundation contact.

There is a limited potential for the core to erode. If erosion were to occur, the fines cannot be stored in the scoria filter/drain or the downstream shoulder and therefore the failure mode can only develop if the coarse zone is present (refer Figure 2) with an open matrix for core material to be stored, potentially leading to voids in the core. If the downstream shoulder material is a sandy gravel, then the failure mode cannot develop.

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 4 are:

- Regular inspection of the crest and downstream side of core,

- Chimney drain flow and turbidity.

Possible actions to reduce risk include:

- Trenching to determine if the 'coarse zone' is present and to recover filter, core and downstream shoulder materials for analysis and testing,
- Closed Circuit Television (CCTV) survey of drains for deposits,
- Developing a more detailed geotechnical model.

Potential Failure Mode 5: Downstream Slope Failure due to Saturated Conditions Developing

Potential Failure Mode 5 is a downstream slope failure due to saturation of the downstream shoulder resulting from sustained overtopping of the core or deterioration of the drainage system.

Saturation due to Overtopping of the Core

Four boreholes were drilled along the embankment crest to determine the top level of the core wall as part of the Stage II SEED Investigations. All four boreholes encountered sandy gravel transitional to gravely sand overlying the embankment core. The fill material appears to be similar in composition to the material reported to be used in the downstream shoulder. There is uncertainty about the minimum level of the core. Borehole logs indicate that the top of the core varies from 2.0m to 2.9m below crest level; however there is disagreement as to whether BH8 is the top or the sloping face of the core (Watercare 2009). This report assumes a reduced level of 41.59m. Watercare drawing 130.16-R4 indicates a wide coarse zone downstream of the gravel topping above the core, up to 40' (12m) deep, refer Figure 2.

If this coarse zone exists then there is potential for fines migration into the coarse zone impeding drainage, resulting in seepage exiting the downstream face rather than through the drainage system.

For overtopping of the core to result in saturation of the downstream face there would need to be low permeability horizons in the upper levels of the downstream shoulder. (Tonkin & Taylor 1994). Lower Huia Dam has a history of discrete recurrent wet patches on the downstream face, which have been noted in past Intermediate and Comprehensive Safety Reviews (T&T, Maunsell, Pickford Consulting, Damwatch Services). The most significant wet spot is the localised seepage and slumping feature towards the left abutment, which was investigated in 2008. The investigation concluded that the wet area was located close to the left abutment and that it was affected by groundwater flows from the left abutment contact.

Deterioration of the Drainage System

The "segregated" coarse fraction used in the drainage zone was achieved by spreading and levelling with a grader and is likely to vary significantly in grading and contain a significant level of scoria fines that could be acting as a transition zone rather than a filter zone (Tonkin & Taylor 1993). Scoria is not durable and can breakdown under compaction and with time. Scoria is often poorly graded and can lead to incompatibility (piping of core) or clogging of drain. The lower two to three metres of the drain may contain soft material.

Historical records indicate long term decreasing trends in the chimney (WMAIN and DRMAIN) and foundation rock drain (WWABT & DRWABT) base flows, which is likely to be abutment and foundation seepage. Spikes in the rubble drain flow relate

to rainfall, possibly indicating a highly permeable downstream shoulder. The chimney drain base flow has decreased asymptotically from 100l/min in 1985 to 22l/minute in 2010 with a significant step decrease around 1996. The foundation drain base flow has decreased asymptotically from 80l/min in 1985 to 10l/min in 2010, however an symmetrical increase occurred between 1989 and 1995. In July 1998 the Caretaker observed a large flow from the rubble drain (300l/min) when the reservoir level was RL41.68, and was considered to be associated with heavy rainfall. The Caretaker also reports often removing sediment from behind the monitoring weir.

The reduction in chimney drain and foundation base flow and the observation of sand and silt accumulated behind the weirs may indicate long term deterioration of the filter and drainage system or blanketing on the upstream face of dam. Upstream piezometers record pressures near lake level.

Saturation of the Downstream Shoulder

The lower part of the clay-silt core has reflected a steady state seepage condition since 1984, however the trend of rising piezometric levels in the upper part of the clay-silt core between 1983 and 1993 reflects steady state seepage conditions associated with a high level of saturation on the downstream side, possibly associated with the drainage filter acting as a transition zone and not being free draining as intended (Tonkin & Taylor 1993).

One standpipe piezometer (P16) and four vibrating wire piezometers (P11VW, P12VW, P14VW and P15VW) are located in the downstream shoulder of the dam. Since their installation questions have arisen with respect to the performance of the vibrating wire instruments. The borehole log for BH1 and piezometric data for P16, located on the lower berm, indicates a drained shoulder of well compacted sandy gravels and low piezometric pressures in shoulder indicate it has better drainage characteristics than first thought; however it is unknown how representative the borehole is of the shoulder in its entirety. The 2010 Comprehensive Safety Review (CSR) recommends the installation of additional piezometers in the downstream shoulder, immediately downstream of the chimney drain and in the left abutment area above the diversion tunnel, to provide an improved basis for assessing the performance of the filter/chimney drain and monitoring piezometric pressures in the vicinity of the diversion and spillway tunnels.

Stability of Downstream Shoulder Under Saturated Conditions

The downstream shoulder comprises compacted pillow lava, sourced from nearby borrows but with extensive zones of weaker tuff forming a matrix for the cobbles (Tonkin & Taylor 1993). The stability analysis undertaken as part of the Stage I Safety Evaluation made allowance for particle degradation during compaction and associated softening of the matrix yielding a high silt fraction, and that the fill behaviour could be dominated by the silt fraction with typical effective strength parameters $C' = 0$ and $\phi' = 30^\circ$, whereas laboratory testing of borrow materials indicated effective strength properties $C' = 0$ and $\phi' = 39^\circ$. The stability analysis indicates a Factor of Safety (FOS) of 1.6 with high phreatic surface and slip circle in bottom of berm. The NZSOLD guidelines suggest a minimum acceptable FOS of 1.5.

For instability of the downstream shoulder to occur a high level of saturation and weaker existing materials than assumed in the stability analysis are required. If the downstream shoulder is a sandy gravel the failure mode cannot develop.

Stability of Downstream Shoulder Under Saturated Conditions and Earthquake Loading

For the seismic case the Factor of Safety was determined to be greater than 1.0 for a range of circular failure envelopes and piezometric surface profiles, with the exception of a deep seated failure under the Maximum Credible Earthquake (MCE) event. Preliminary analysis indicates insignificant displacement would occur (Tonkin & Taylor 1993).

Risk Reduction Measures

The key performance parameters for Potential Failure Mode 5 are:

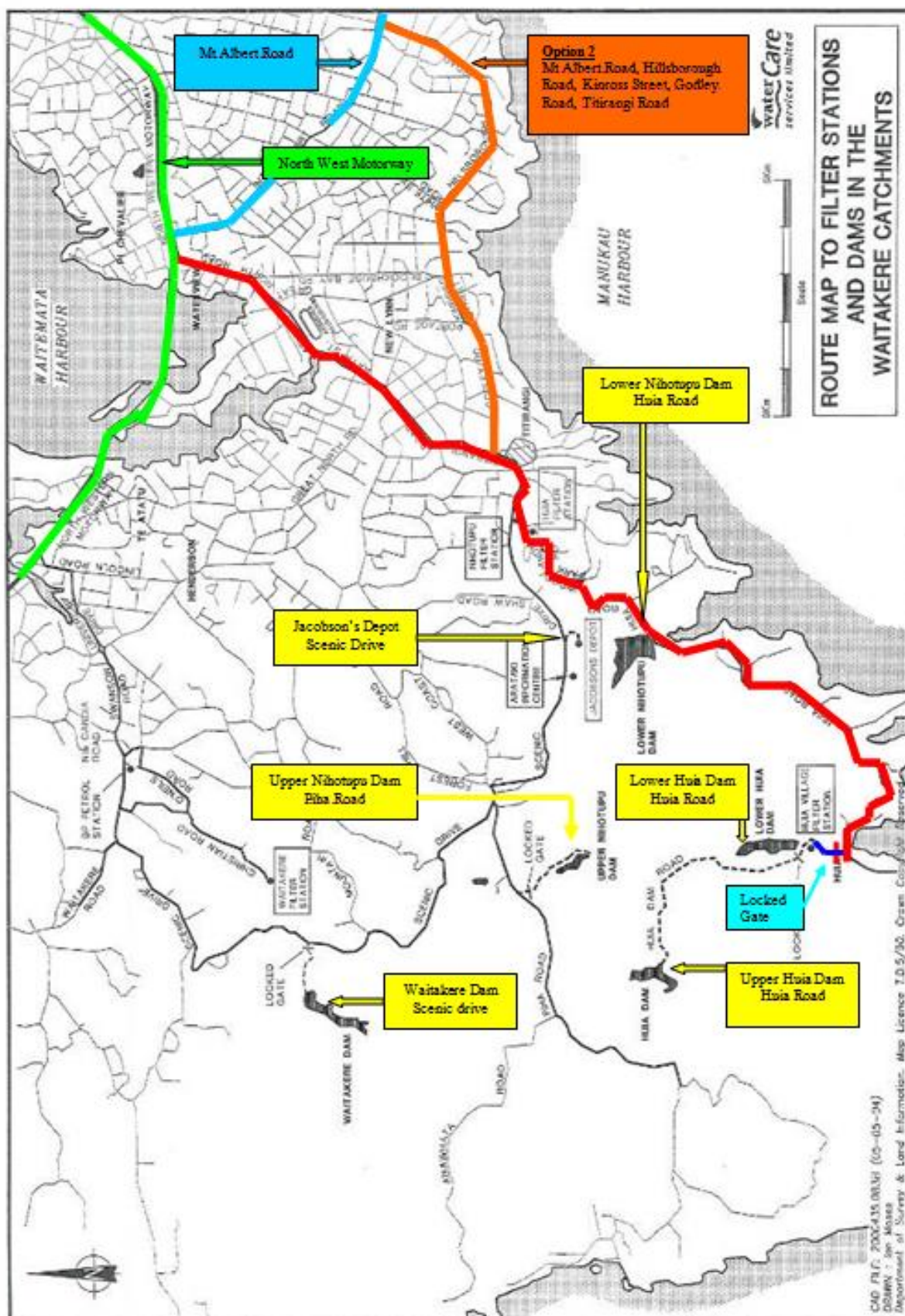
- Changes in chimney drain and foundation flows,
- Changes in seepage on the downstream face,
- Widespread increased piezometric pressures (P25, P32 and P14-16) due to back pressurisation of the drain and subsequent build up in tailwater,
- Saturation of the toe,
- Settlement of the downstream face as a result of prolonged saturation.

Possible actions to reduce risk include:

- Drilling additional boreholes in the downstream shoulder and filter to investigate materials, particularly percentage fines, and to install additional piezometers to monitor saturation of the downstream shoulder,
- Characterising the source of drain flows and seepage on the downstream face through water sampling,
- Trenching to investigate seeps on downstream face,
- Review subsoil drain installation records
- Review the stability based on the above investigations

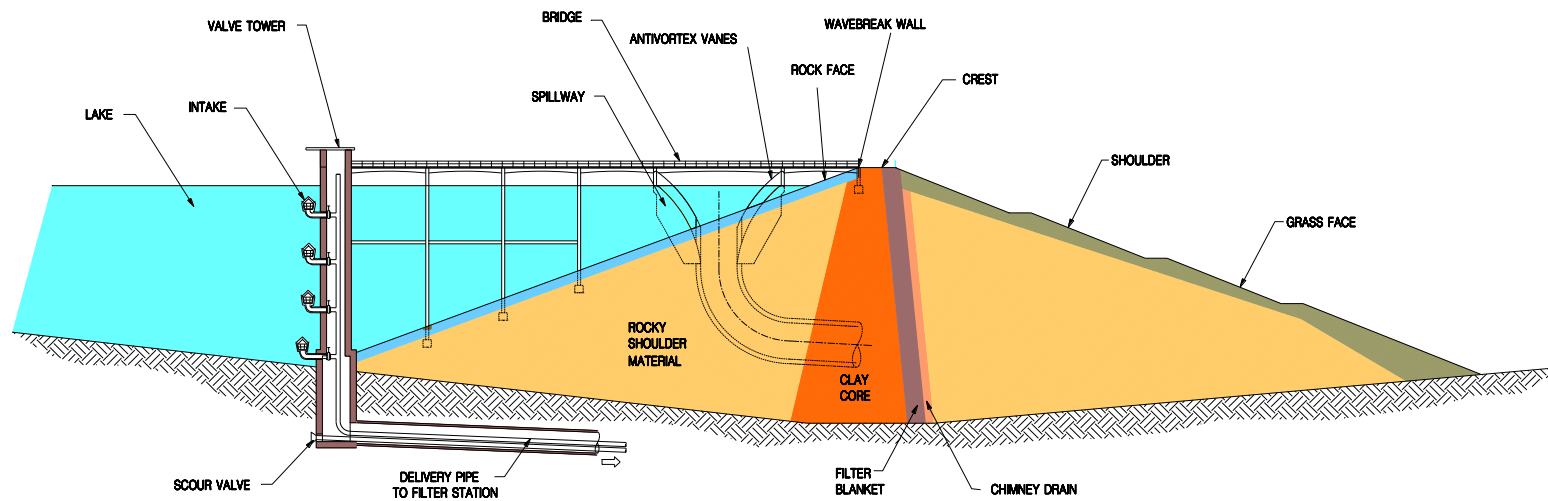
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Lower Huia Dam Route Map



DWG No. 2000435.083 B

Cross-section



Dambreak Path



Hydraulic Structure Data

Hydraulic Structure	Lower Huia Dam	
Structures		
Levels	Lands & Survey Datum 1946 (approximately)	
Top of clay core	Varies from 40.452 to 42.697	
Top of plastic concrete core	Varies from 43.816 to 44.487	
Dam crest	44.70 to 44.97	
PMF peak level	43.624	
Top of spillway crest	41.454	
Top of auxiliary spillway crest		
Intake No 1	35.300	
Intake No 2	30.000	
Intake No 3	21.900	
Intake No 4	13.786	
Intake No 5		
Intake No 6		
Scour intake	7.928	
Discharges		
Maximum Capacity of supply to treatment	54,500 m³/day (4 pumps)	
Maximum flow capacity spillway crest	461 m³/s	
Maximum capacity IFDV		
Maximum capacity EFDV	2.7 m³/s	
Catchment		
Water Source	Huia Stream	
Catchment area (ha)	1430	
Surface area of full lake (ha)	50.3	
Live Storage at full volume (m³)	6,422,000	
Storage between spillway and crest (m³)	1,525,000	
Hydrology		
Return Period	Q (m³/s)	
	Inflow	Outflow
Mean Annual Flow	45	27
5year	63	42
10 year	78	54
20 year	93	66
100 year	111	80
500 year		170
1000 year		185
PMF	528-315.4	461-287.71
Benchmark		
Valve Tower	44.510	

The spillway level above is based upon the lowest level of the spillway. The spillway rating is based on the nominal spillway level.

Hydraulic Structure	Lower Huia Dam
Background	
Location	Huia Valley, just above Huia Bay
Water source	Huia Stream
Purpose	Water supply
Date Built	1967 - 1971
Dam engineering	Auckland City Council
Dam construction	Green & McCahill
Official opening	September 30 1971
Dam Construction	
Structure	Earth/rockfill embankment with filter blanket
Height (m)	39.6
Crest length (m)	366
Crest width (m)	
Dam volume (m ³)	916,000
Valve tower	Free standing
Spillway type	Bellmouth
Auxilliary spillway	
Scour Valves	
Valve type located outside dam	FDV (Sleeve valve)
Size (mm diameter)	450
Installation date	1971
Condition	Good
Method of operation	Manual
Maximum scour flow rate (m ³ /sec)	2.4
Maximum scour rate off the intakes (m ³ /sec)	2.7
Valve type located within dam	
Size (mm diameter)	
Installation date	
Condition	
Method of operation	
Maximum scour flow rate (m ³ /sec)	
Maximum scour rate off the intakes (m ³ /sec)	
Notes	
The PMF flood will extend to 700 mm above the crest of the Lower Huia Dam	

Lower Huia Dam Spill Flow Rating

Pertinent details used in spillway flow rating

1) Type: circular bellmouth spillway with ungated overflow crest with an assumed ogee weir profile.

2) Spillway diameter (outside) 21.336m, with 4 piers (fins) 0.381 m thick.

3) Design head: 1.96 m inferred from drawings

4) Apparent discharge coefficient range: 1.63 to 2.16 $\text{m}^{0.5}/\text{s}$

5) Lowest spillway crest level: 41.445 m RL

6) Maximum spillway crest level: 41.459 m RL

7) RL of local gauge datum: 13.71 m RL

8) Nominal dam crest level: ~~43.76 m RL (WCS, 1993)~~ **44.70 (WGM 2018 is true figure. Wrong value was used in the original Works calculation)**

9) Low range rating data: 0 to 0.3 m head

10) Full range rating data: 0 to 2.30 m head

11) Probable maximum flood (PMF) estimate: 461 m^3/s (WCS, 1993)

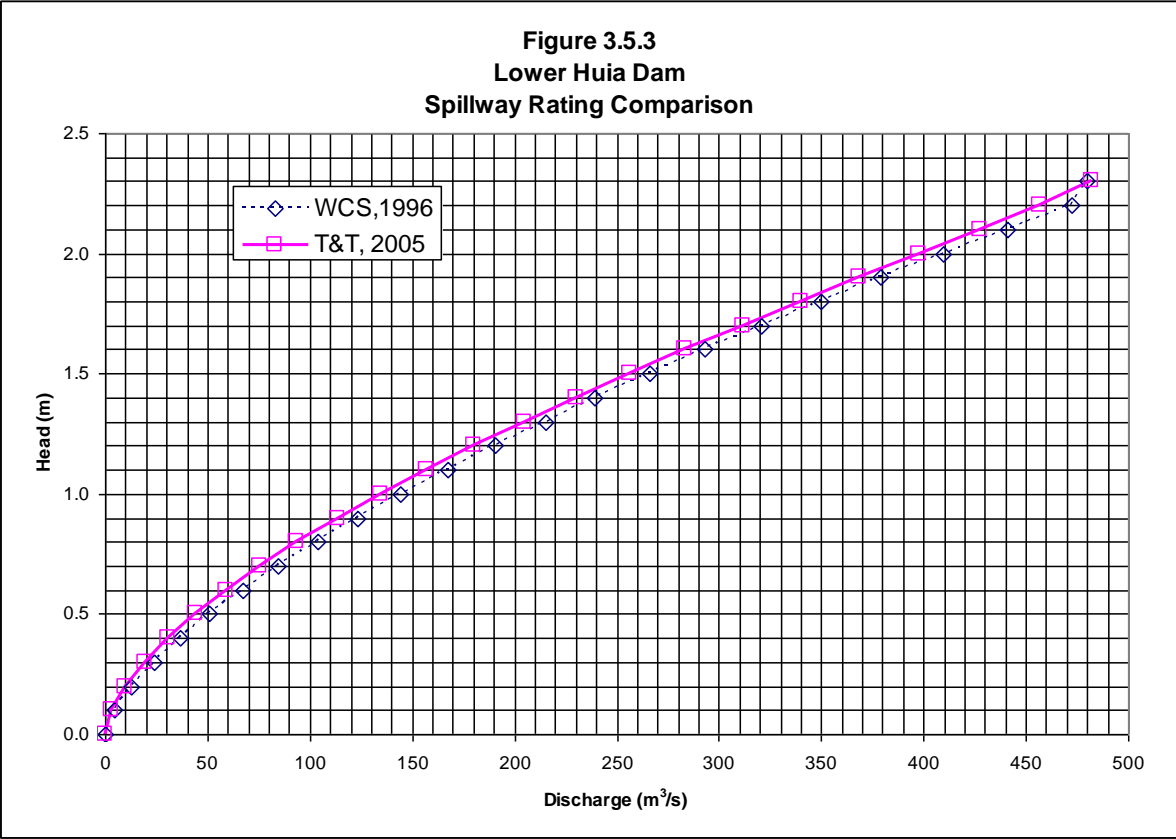
12) Dam overtopped in PMF: No (WCS, 1993)

Note that the highest rated flow in the data is 482 m^3/s at a head of 2.3 m, which is just below the dam crest. Note that the spillway barrel starts to choke the flow above a head of about 2.3 m.



Lower Huia Dam Full Range Spillway Rating (Tonkin & Taylor)

Water Level (m RL)	Gauge Level (m)	Head Over Spillway Crest (m)	Total Discharge (m ³ /s)
41.445	27.735	0.000	0.0000
41.470	27.760	0.025	0.2291
41.495	27.785	0.050	0.919
41.520	27.810	0.075	1.887
41.545	27.835	0.100	3.083
41.570	27.860	0.125	4.48
41.595	27.885	0.150	6.07
41.620	27.910	0.175	7.84
41.645	27.935	0.200	9.78
41.670	27.960	0.225	11.89
41.695	27.985	0.250	14.16
41.720	28.010	0.275	16.60
41.745	28.035	0.300	19.20
41.770	28.060	0.325	21.96
41.795	28.085	0.350	24.81
41.820	28.110	0.375	27.74
41.845	28.135	0.400	30.75
41.870	28.160	0.425	33.89
41.895	28.185	0.450	37.14
41.920	28.210	0.475	40.51
41.945	28.235	0.500	43.99
41.995	28.285	0.550	51.3
42.045	28.335	0.600	59.0
42.145	28.435	0.700	75.6
42.245	28.535	0.800	93.8
42.345	28.635	0.900	113.4
42.445	28.735	1.000	134.4
42.545	28.835	1.100	156.6
42.645	28.935	1.200	180.0
42.845	29.135	1.400	230.0
42.895	29.185	1.450	243.1
42.945	29.235	1.500	256.3
43.045	29.335	1.600	283.5
43.145	29.435	1.700	311.3
43.245	29.535	1.800	339.7
43.345	29.635	1.900	368.6
43.445	29.735	2.000	397.8
43.545	29.835	2.100	427.2
43.595	29.885	2.150	442.0
43.645	29.935	2.200	456.7
43.695	29.985	2.250	471.5
43.705	29.995	2.260	474.6
43.720	30.010	2.275	479.4
43.745	30.035	2.300	482.1



Lower Huia Dam Spillway Rating Curve

Estimation of Earthquake Spectra

Reference Report 4181 – Estimation of Earthquake Spectra For Watercare Dams by GNS Science Consultancy Report, May 2006
M. C. Gerstenberger & G. McVerry
GNS Science Consultancy Report 2006/048
May 2006

Report No 4128

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 (36.97°S, 174.57°E).

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 $\frac{2}{3}$ 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 SEE¹ Motions

Period (s)	Hunua			Waitakere		
	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 (% of 84-percentile motions for magnitude 6.5 earthquake at 20 km) govern rather than 500 year return period motions

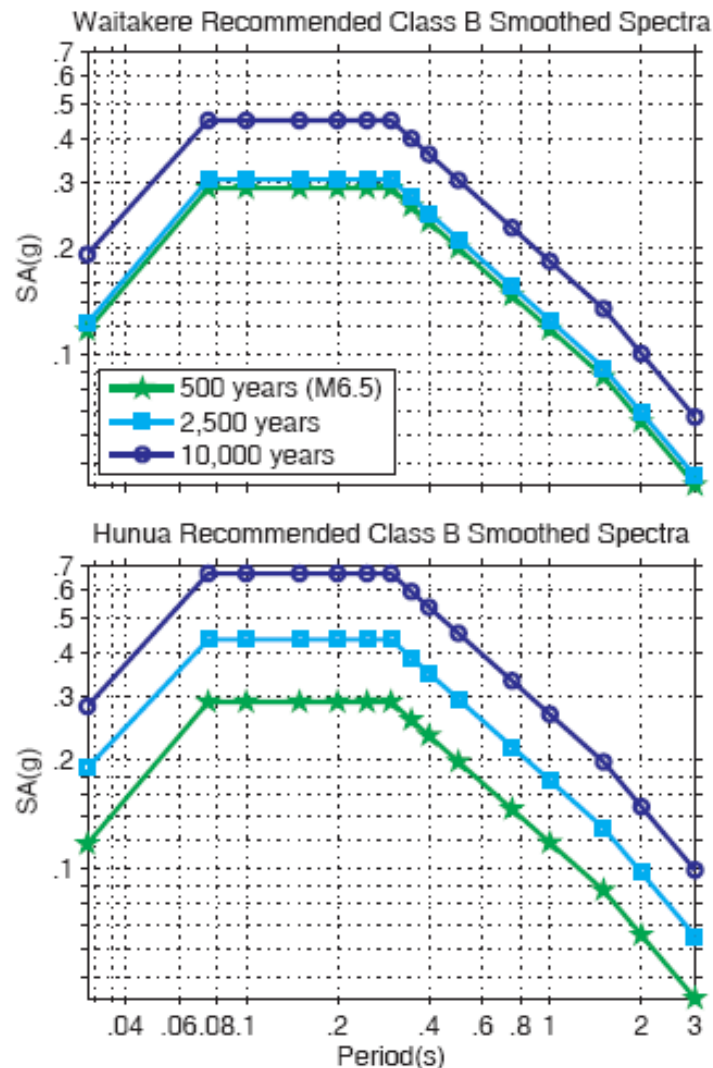


Figure 1 Recommended smoothed spectra for the Safety Evaluation Earthquake motions for the Waitakere and Hunua Ranges dams.

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 (36.97°S, 174.57°E). 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 $\frac{2}{3}$ 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 $\frac{2}{3}$ 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.

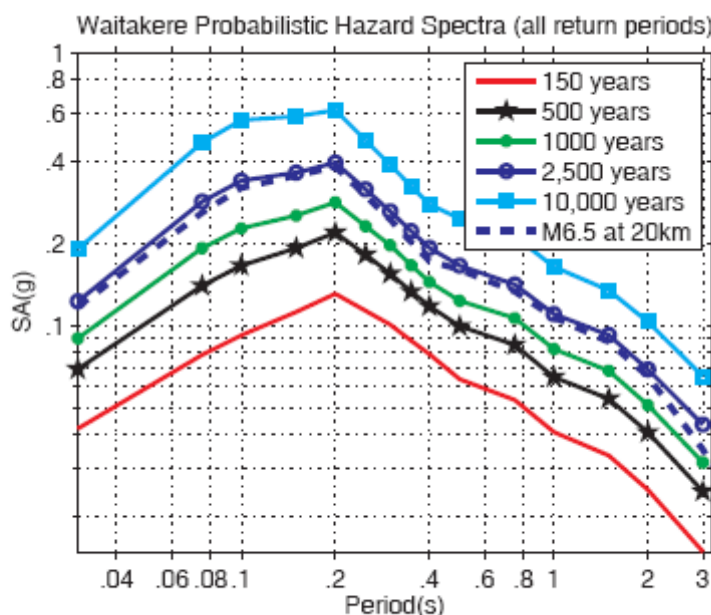


Figure 2 Raw spectra for all calculations done for the Waitakere Ranges site for a Class B rock site.

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 $\frac{2}{3}$ 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	84 th percentile for Controlling Maximum Earthquake (CME). Need not exceed 10,000-year motions	50 th – 84 th 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

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 T_1 , 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 $T^{-\gamma}$, 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 T_2 to 1.5s, with

T_2 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 T_1 and T_2 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)	$S_a(0)$ (g)	$S_a(T)$ for $T_1 < T < T_2$ (g)	$S_a(T)$ for $T_2 < T < 1.5s$	$S_a(T)$ 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 shown in Figure 3. The smoothed values for each period are listed in Table 1.

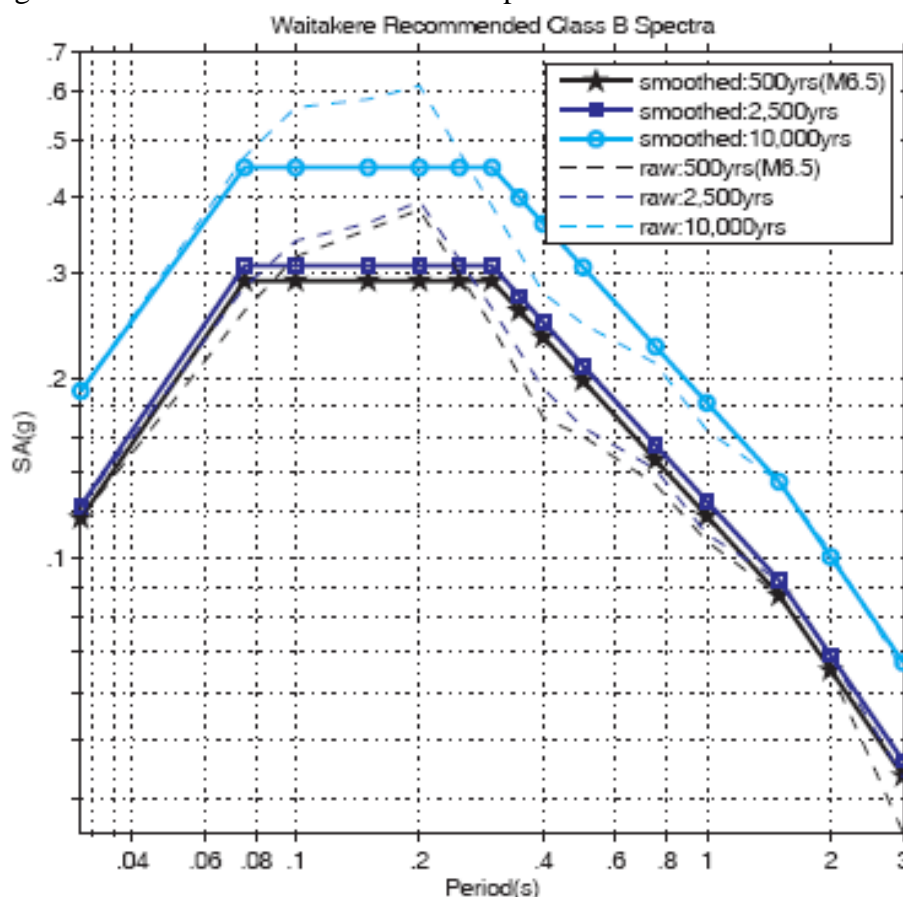


Figure 3 Recommended raw and smoothed spectra for the Waitakere Ranges for a Class B rock site.

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.

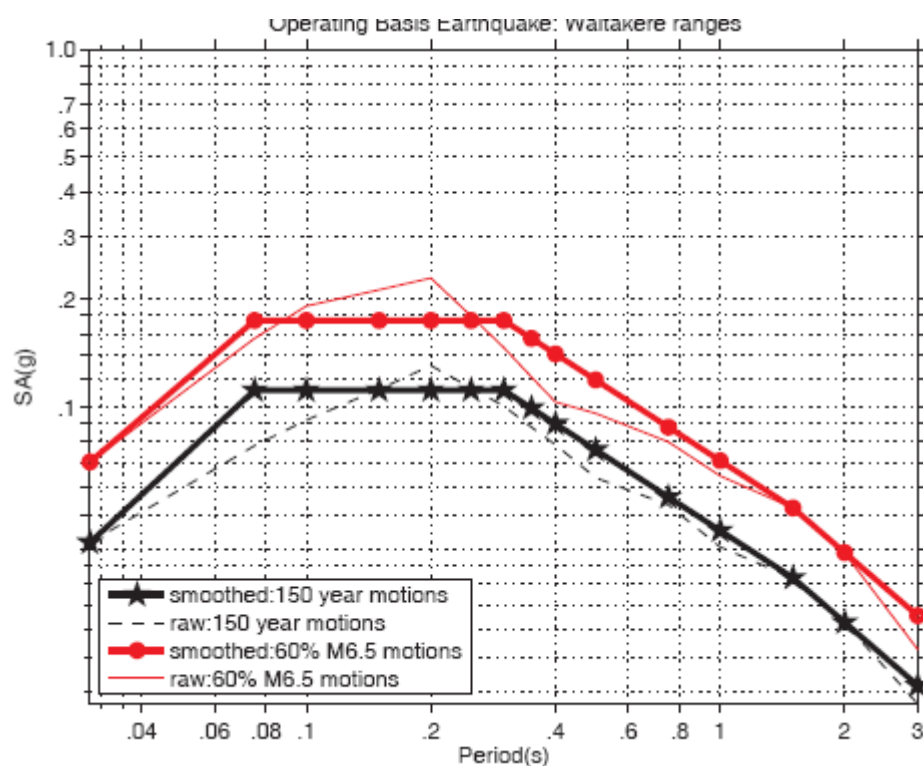


Figure 8 Candidate raw and smoothed OBE spectra for the Waitakere Ranges

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	0.158
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

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.

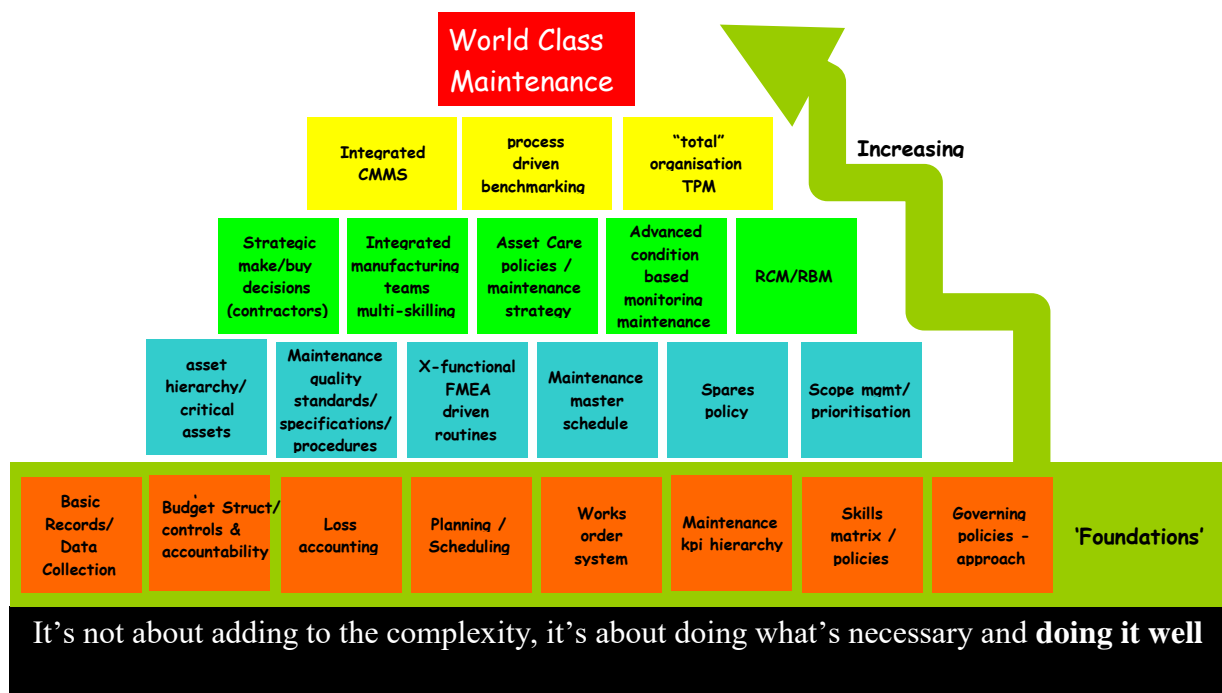
Watercare wanted a "blue print" based on best practice

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

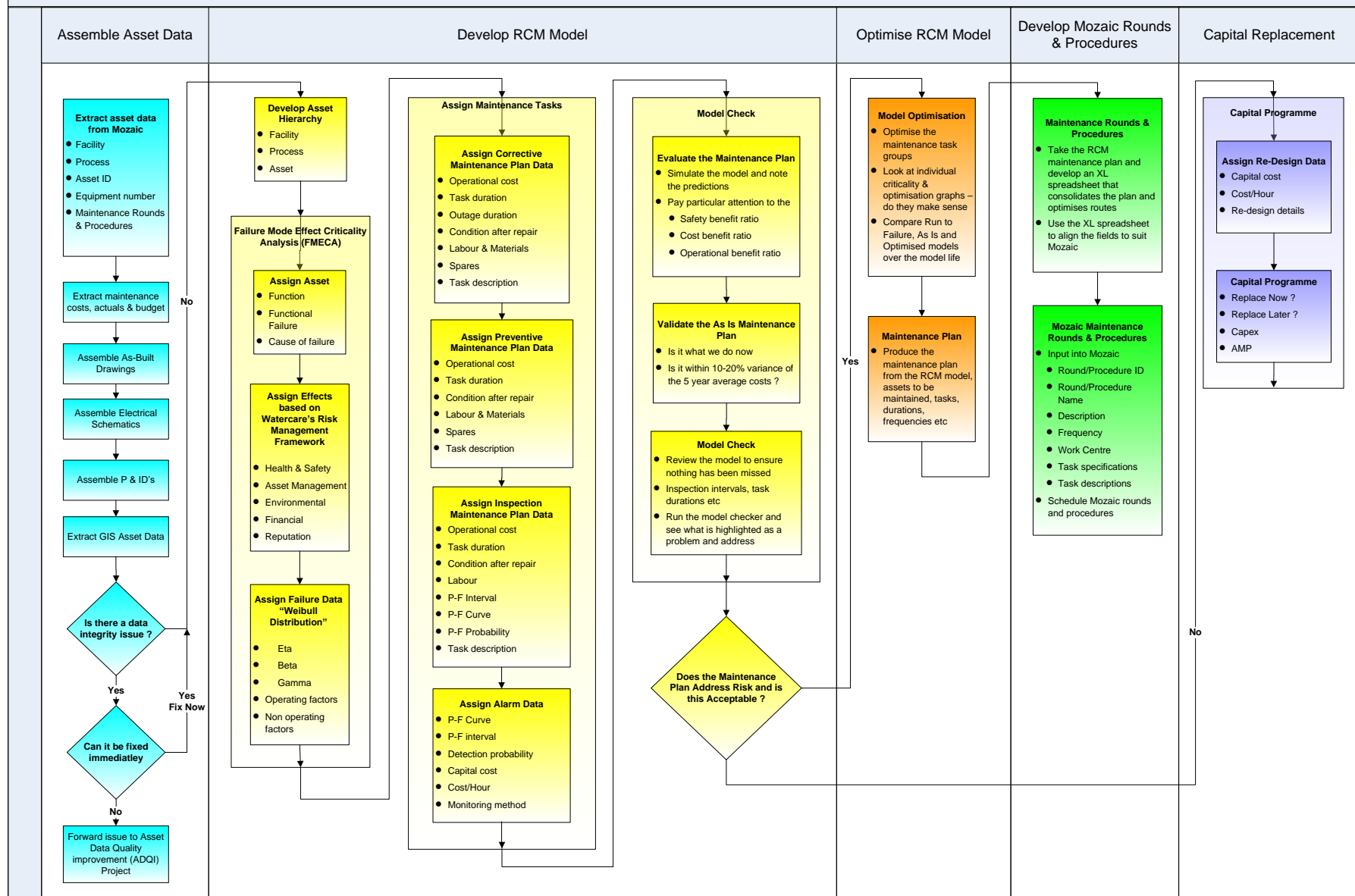
- Consistent maintenance strategy;
- Critical assets identified and ranked in priority;
- Maximised long term sustainability of assets;
- Optimised cost of maintenance vs. cost of failure;
- Alignment to an international standard;

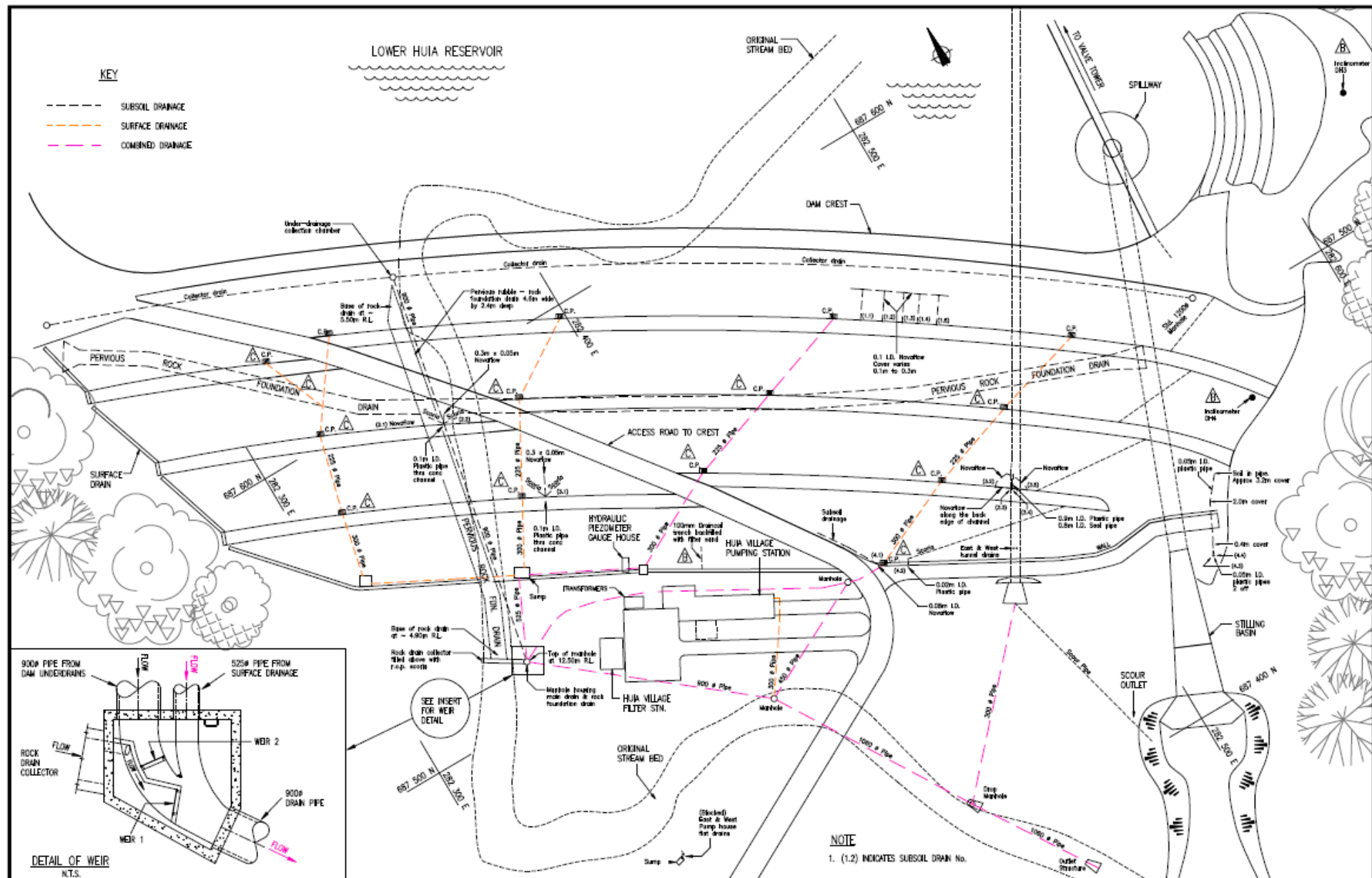
- Appropriate maintenance plans assigned to each asset (condition, performance, time based, run to failure); and
- Address Health & Safety, Environmental and Operational issues

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



Reliability Centred Maintenance (RCM) Implementation





				DESIGNED	J.L.	05-04	COPYRIGHT This drawing, the design and concept, remain the exclusive property of Watercare Services Limited and may not be used without approval. Copyright reserved.		LOWER HUIA DAM		DAD FILE 2003958.DWG		DATE 18-03-08				
				REV. CHECKED					ORIGINAL SCALE A1		CONTRACT No.		1:500		-		
				DRAWN	L.W. & L.A.C.	05-04			DRAWING No.		2003958		ISSUE		C		
									PROJECT LEADER		L.C.						
C	3-08	CESPITS AND DRAINS ADDED	L.W.	J.M.	DWG. CHECKED		ASSET MANAGEMENT			LOCATION OF DRAINAGE FEATURES							
B	2-08	INCLINOMETERS ADDED, DRAGONUL TRENCH & NOTE	L.W.	J.M.						AND SEEPAGE MONITORING INSTRUMENTATION							
A	10-07	FENCING EXTENDED, ROADS & BUILDINGS ADDED	L.C.	J.M.	DS APPROVED												
ISSUE		DATE	AMENDMENT		BY	APPROV.	BY			DATE							

Reports for Lower Huia Dam

Report No.	Reports Title	Corp Author	Publ. Date
503	Test Boring Report Sheet - Huia Lower Dam Titirangi Reservoir Huia Filter Station		21/08/1970 0:00
561	Lower Huia Valve Tower And Access Bridge Structural Evaluation Report - Seismic Capability Review (Kennard)	TASS	1/02/1993 0:00
1042	Huia Headworks Operations And Maintenance Manual	Lummus	1/06/1993 0:00
2053	Pneumatic And Hydraulic Piezometers Evaluation And Maintenance Waitakere Upper Huia Lower Huia Cosseys Wairoa Mangatangi Dam	Opus Central Laboratories	
2110	Lower Huia Dam Five Year Safety Review	OPUS	1/10/1999 0:00
2195	Huia And Nihotupu Catchments Assessment Of Environmental Effects	Water	30/03/2001 0:00
2210	Annual Dam Safety Review Yr2001 Lower Huia Dam	G Euinton	15/06/2001 0:00
2431	Annual Dam Safety Report Yr2002	G Euinton	20/05/2002 0:00
2660	Lower Huia Dam Annual Safety Review 2002/2003	G Euinton	10/06/2003 0:00
2866	Annual Dam Safety Review 2003/2004	M Laws	30/04/2004 0:00
3084	Sinkhole Report In Upstream Riprap Near Centre Of Dam	Damwatch Services Ltd	30/08/2004 0:00
3178	Copy Documentation Of Dam Scour Valve For Hunuas And Waitakere	Watercare Services Ltd	12/12/1997 0:00
3391	Deformation Survey Report Full Survey 26 May 2005 Lower Huia Dam	Energy Surveys Ltd	26/05/2005 0:00
3406	Dam Deformation Survey Report 26th May 2005 Full Survey	Energy Surveys Ltd	26/05/2005 0:00
3466	Lower Huia Dam Intermediate Dam Safety Review Inspection Date 23 March 2005	Damwatch Services Ltd	16/08/2005 0:00
3682	Aeration Compressor Operation Maintenance Manual	Watercare Services Ltd	
3780	Deformation Survey Report Lower Huia Dam Left Abutment 18 January 2006	Energy Surveys Limited	18/01/2006 0:00
3830	2005 Lower Huia Dam Inclinator Monitoring	Geotechnics Limited	7/04/2005 0:00
4049	Lower Huia Dam Post Earthquake Observations		20/12/1998 0:00
4096	Response Testing Of Piezometers Lower Huia Dam December / January 1998	Richard Body	20/02/1999 0:00
4123	Intermediate Dam Safety Review Inspection Date 22nd March 2006	Damwatch Services Ltd	28/07/2006 0:00

Report No.	Reports Title	Corp Author	Publ. Date
4229	Intermediate Dam Safety Review Period 1st August 2005 To 31st August 2006	Ergo Consultants & MTL	28/09/2006 0:00
4265	Lower Huia Dam Left Abutment Deformation Survey Report	Energy Surveys Ltd	18/01/2006 0:00
4276	Lower Huia Dam Full Survey 19 April 2006 Deformation Survey Report	Energy Surveys Ltd	19/04/2006 0:00
4533	Dam Data Book Volume 1 Correspondence	Watercare Services Ltd	
4534	Dam Data Book Volume 1a Correspondence	Watercare Services Ltd	
4535	Dam Data Book Volume 2 Structural Evaluation Study 1993 WSL	Watercare Services Ltd	
4536	Dam Data Book Volume 2a S E E D Report 1993 To 1994	Watercare Services Ltd	
4537	Dam Data Book Volume 2b S.E.E.D. Report 1999	Watercare Services Ltd	
4538	Dam Data Book Volume 2c Investigations	Watercare Services Ltd	
4539	Dam Data Book Volume 3 Photographic Record	Watercare Services Ltd	
4540	Dam Data Book Volume 3a Photographic Record	Watercare Services Ltd	
4541	Dam Data Book Volume 4 Drawings	Watercare Services Ltd	
4542	Dam Data Book Volume 4a Drawings	Watercare Services Ltd	
4543	Dam Data Book Volume 5 Electrical & Mechanical Data Operations	Watercare Services Ltd	
4544	Dam Data Book Volume 6 Annual Safety Review	Watercare Services Ltd	
4683	Cathodic Protection Installation For The Waitakere Headworks Water Pipelines At LHD	NGC Specialist Services	5/03/1999 0:00
4860	Lower Huia Dam Comprehensive Safety Review Five Yearly Independent Audit	MAUNSELL;AECOM	1/06/2005 0:00
4865	Intermediate Dam Safety Review Period 1 March 2006 To 28 February 2007	Damwatch Services Ltd	1/05/2007 0:00
5102	Lower Huia Dam Slumping At The Left Abutment April 2008	Damwatch Services Ltd	2/04/2008 0:00
5126	Lower Huia Dam CCTV Of The Downstream Face Stormwater Drains	Underground Vision Ltd	13/03/2008 0:00
5128	Lower Huia Dam Structural Check On Valve Tower Bridge	Babbage Consultants	30/01/2008 0:00
5143	Lower Huia Dam Inclinator Monitoring Results To 24april 2008	Geotechnics	24/04/2008 0:00
5161	Lower Huia Dam Intermediate Dam Safety Review 1 March 2007 To 29 February 2008	Damwatch Services Ltd	29/02/2008 0:00
5575	Intermediate Dam Safety Review Period 1st March 2008 To 28th February 2009	Damwatch Services Ltd	8/05/2009 0:00

Calculation Records

Calculation #	Subject	Originator
87	Huia Valley access bridge	Auckland Regional Authority
119	Lower Huia Dam - Scour valve jet disperser design assumption	Turner
120	Lower Huia Dam - Calculation type a and b	
121	Lower Huia Dam – Calculation	
127	Lower Huia Dam - Moisture determination concrete aggregate and grading curve	Harbridge
128	Lower Huia Dam - Spillway Tunnel	Wilson
129	Lower Huia Dam - Spillway bellmouth	
130	Lower Huia Dam - Survey data	
131	Lower Huia Dam - Spillway stilling basin	Bell
132	Lower Huia Dam - Spillway outlet work	Wilson
133	Lower Huia Dam - Spillway outlet	Turner
134	Lower Huia Dam - Stability calculation	
135	Lower Huia Dam - Stability analysis d/s shoulder	
136	Lower Huia Dam - Stability analysis rapid drawdown	
137	Lower Huia Dam - Stability analysis steady state u/s shoulder and investigation into various method	
138	Lower Huia Dam - Stability analysis rapid drawdown u/s shoulder	
139	Lower Huia Dam - Stability analysis computer using modified bishop	
140	Lower Huia Dam - Perforated pipe drain and vertical pipe	Wilson
141	Lower Huia Dam - Soil testing during construction	
142	Superseded Lower Huia Dam - Laboratory record	
143	Lower Huia Dam - Tunnel concrete analysis	
362	Lower Huia Dam - Scour	Wilson
370	Lower Huia Dam - House foundation	Wright
402	Lower Huia Dam - Investigation	Hoyle
406	Lower Huia Dam - Spillway outlet	Bell
408	Lower Huia Investigations	D.B. Hoyle
1008	Lower Huia Dam - House foundations	A H Wright
1407	Lower Huia Dam - Valve tower	Goldwater

Data Books Schedule of Contents - Lower Huia Dam

Volume	Pages	Description	Author
1	5 - 29	Geology of the Waitakere Ranges – Field Notes	Bruce Haywood
	29 - 117	Contract 410 – Lower Huia Dam	Auckland City Council
	118 -124	Silt Accretion at Huia Bay	ARWB (Now ARC)
	125 - 129	Surface Seepage Left Abutment	Don Wilson
	130 - 131	Glover Bridge	ARC
	132	Water Supply Main	ARC
	133	Request for Work Programme	ARC
	134 - 135	Insurance Details	
	136 - 147	The Huia Water Supply Extensions for the City of Auckland	A Mead
	148 - 157	Scour Valve Performance	
	157 - 170	Hydraulic Piezometers	
1A	4 - 157	Lower Huia Dam Calculation Relative to Type C Dam	
	158 - 209	Lower Huia Dam Stability Analysis – Steady State U/S Analysis (Report 137)	
	210 - 238	Lower Huia Dam Stability Analysis – Rapid Drawdown (Report 138)	
	239 - 247	Lower Huia Dam Tunnel Concrete Analysis (Report 143)	
	248 - 378	Lower Huia Dam Spillway Outlet.	
2	5 - 55	Lower Huia Valve Tower & Access Bridge Structural Evaluation	
	56 - 85	Auckland Regional Council Dams Seismic Hazard Study, May 1990	
	86 - 135	Appendix D Original Calculations	
	136 - 245	ARC Water Supply Dams Flood Capacity Study November 1991	
	246 - 278	ARC Bulk Water supply dams Seismic Hazard study, May 1990	
	279 - 296	Addendum to Probable Maximum Precipitation in New Zealand	
2A	5 - 51	Huia Dams – Report on Dam Break Analysis	ARC
	52 - 93	Probable Maximum Floods for 10 Water Supply Dam in the Auckland Region	WCS
	94 - 172	Lower Huia Dam Safety Evaluation Stage 1	Tonkin & Taylor
	173 - 267	Lower Huia Dam Safety Evaluation Stage 2	Tonkin & Taylor
	266 - 315	Lower Huia Dam Safety Evaluation Stage 2 Calculations	Tonkin & Taylor

Data Books Schedule of Contents - Lower Huia Dam (Continued)

Volume	Pages	Description	Author
2B	5 - 69	Lower Huia Dam Five Yearly Review	Opus
	70 - 118	Lower Huia Dam Piezometer Installation	
2C	5 - 72	Compliance and Issues Report	Design Services
	73 - 154	Pneumatic and Hydraulic Piezometers	Opus
3	7 - 8	Aerial Photographs	
	9 - 11	Weir Photographs	
	12 - 22	Dam Site & Reservoir Photographs	
	23 - 25	Diversion Tunnel Photographs	
	26 - 28	Valve Tower	
	29 - 31	Spillway Photographs	
	32 - 34	Quarry Photographs	
	35 - 37	Machinery Photographs	
3A	7 - 9	Miscellaneous Photographs	
	10 - 28	CCTV Inspection of the Underdrains	
4	5 - 8	General Details Drawings	
	11 - 17	Reservoir Site drawings	
	18 - 24	Diversion Tunnel drawings	
	25 - 54	Dam Site drawings	
4A	7 - 30	Spillway and Stilling Basin Drawings	
	31 - 47	Valve Tower drawings	
	48 - 52	Instrumentation drawings	
	53 - 61	Electrical, mechanical drawings	
6	5 - 58	2002/2003 Annual Safety Review of Lower Huia Dam	Design Services
	59 - 108	2001/2002 Annual Safety Review of Lower Huia Dam	Design Services
	109 - 163	2000/2001 Annual Safety Review of Lower Huia Dam	Design Services

Lower Huia Dam, continued

Benchmarks

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

Benchmark Location	Reduced Level (RL)
Valve tower floor by lake level recorder.	44.510

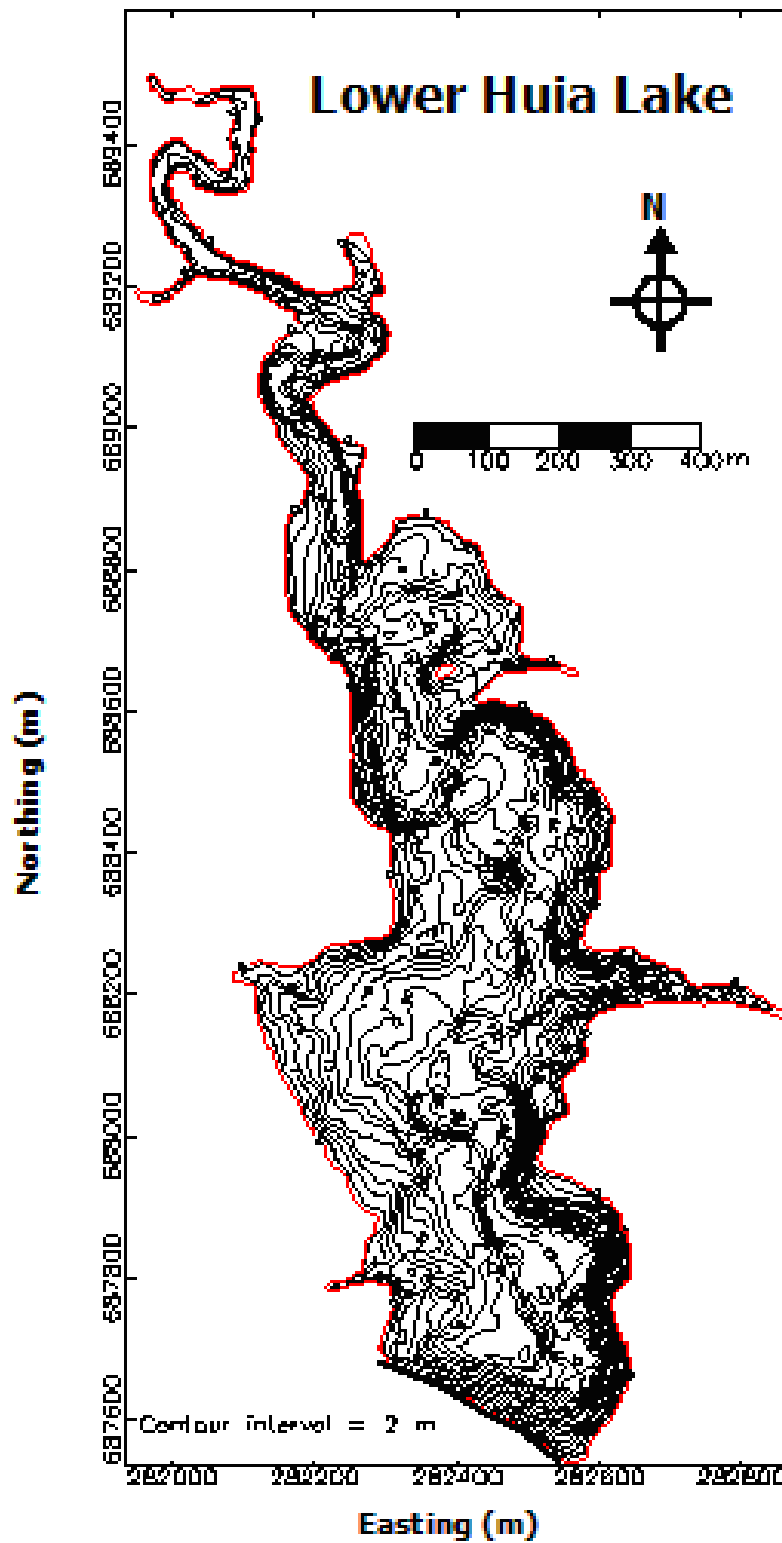


Continued

Lower Huia Dam, continued

Bathymetric Plan

The following bathymetric plan shows the area and contours of Lower Huia lake.



Lower Huia Dam Structural Details

- 1) Type: Circular bellmouth spillway, with ungated overflow crest with an assumed ogee weir profile.
- 2) Spillway diameter (outside): 21.336 m, with 4 piers (fins) 0.381 m thick
- 3) Design head: 1.96 m inferred from drawings
- 4) Apparent discharge coefficient range: 1.63 to 2.16 $\text{m}^{0.5}/\text{s}$
- 5) Lowest spillway crest level: 41.445 m RL
- 6) Maximum spillway crest level: 41.459 m RL
- 7) RL of local gauge datum: 13.71 m RL
- 8) Nominal dam crest level: 43.76 m RL (WCS, 1993)
- 9) Low range rating data: 0 to 0.30 m head
- 10) Full range rating data: 0 to 2.30 m head
- 11) Probable maximum flood (PMF) estimate: 461 m^3/s (WCS, 1993)
- 12) Dam overtopped in PMF: No (WCS, 1993)

Note that the highest rated flow in the data is 482 m^3/s at a head of 2.3 m, which is just below the dam crest. Note that the spillway barrel starts to choke the flow above a head of about 2.3 m.



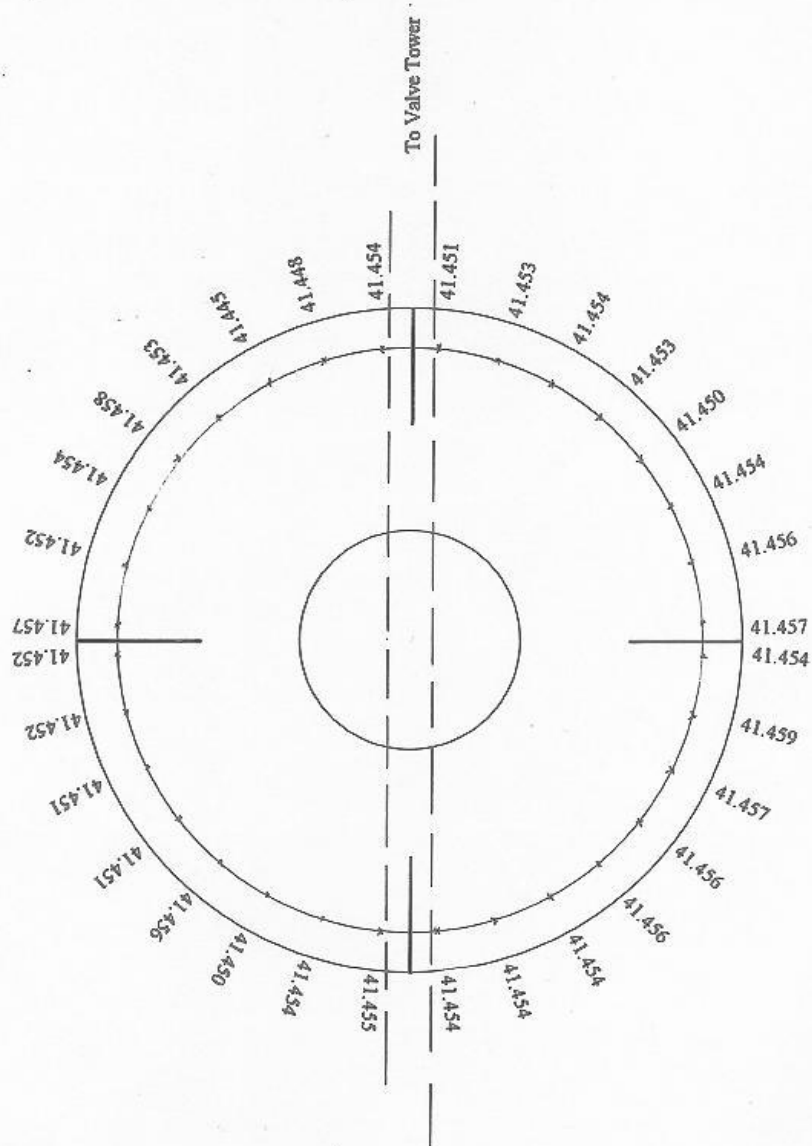
Lower Huia Dam Spillway Levels

LOWER HUIA DAM

Spillway Crest Levels

Sketch only : Not to Scale

Levels spaced at 2m intervals in each bay



Lower Huia Dam Confirmation of Benchmark



Watercare Services Limited
 273a Church Street Telephone
 Private Bag 92802 634 4827
 Penrose Facsimile
 Auckland 636 6204
 New Zealand

Fax Message

To Royd Cummings
 Company Auckland Regional Council Fax _____
 Subject Bench Mark For Lower Huia Dam No of Pages 1
 From Wallace McQuarrie Date 21 October 1998
 Group/Unit Water

There is an anomaly in the lake levels for the Lower Huia Dam.

The Automatic digital level recorder and the electric plumb bob are both reading 97mm too high at present. The data is as follows:

RL of cu plug on the Valve tower deck	RL 44.510
RL of the spillway	RL 41.454
RL of the Lowest intake	RL 13.786
RL of the tide gauge at 95.5 ft	RL 42.790
Assumed height of spillway above bottom intake	27.790m (91.12 ft) $91.12' = 27.775m$
Calculated datum based on tide gauge	13.664 (41.454 - 27.790)
Measured level for the datum	13.786
Correction required to the level recorder	0.124
RL of lake level at 10/10/96	RL 41.471
Lake level recording	27.782
Datum by calculation	13.689

$TWL = 41.471$ 27.782 $91.175' = 27.791m$ Measured

guess top
bottom intake = 13.689 13.680 13.786

Lower Huia Dam Correction to PMF Calculation of Heights

Lower Huia Dam PMF

Reference the Report by Works Consultancy dated 24 March 1993

Description	Works Assumptions & Analysis		Revised levels on structures	
	(Levels in gauge heights)(m)	Gauge heights	RLs	
Crest Height	30.05	31.019	44.700	
Height of PMF Flood	29.92	29.943	43.624	
Height of spillway crest	27.75	27.773	41.454	
Bottom intake		0.000	13.681	
Height of PMF above the spillway crest	2.17	2.17	2.17	
Freeboard available at PMF	0.13	1.076	1.076	

Lower Huia Dam Datum Checks



Memorandum

TO: Wallace McQuarrie
FROM: Phil Salmon
SUBJECT: **Checking the Level Recordings at the Lower Huia Dam**
DATE: 21 October 1996 FILE: 11/18/11

Wallace,

In reply to your memo dated 10 Oct 1996 I reply as follows,

- The spillway crest levels immediately adjacent to the valve tower bridge deck were checked to confirm agreement with my memo dated 12 May 1995 (copy attached). For practical purposes the average value of the spillway crest level is taken as RL41.454 (A Mcpike)
- The Cu Plug benchmark in the centre of the valve tower deck RL44.510 was established in 1986 from the level traverse shown on drawing 114145-02.
- As the tide gauge graduations are coarse, and as the setting out of the numbers was not to absolute millimetre accuracy it is not possible to give a precise gauge reading for the spillway overflow level.

The 95.5^{FT} graduation is at RL42.790.

By deduction the tide gauge reading for the spillway crest TWL is
 $\{95.5 - (RL42.790 - RL41.454) / 0.3048\} = 91.12^{FT}$

- At the time of survey the following values were recorded –

Lake level (by survey)	RL41.471
Automatic digital level recorder	27.782
Electronic plumb bob	27.783

- At the time of survey the height of water passing over the spillway was deduced –

Lake level - Spillway level $RL41.471 - RL41.454 = 0.017m$

27.783
17
27.766

Lowest Inlet 13.786 VT 021 44.510

- The best value we have at present for the bottom intake level is from an unchecked sounding. There is a discrepancy of 0.069m between this sounding value and the original contract plan value. The only practical method of checking the bottom intake level would be to carry out another independent sounding.

You ask for an assessment of the bottom intake level. While taking the original sounding, measurements were taken on both the left and right side flanges of the intake structure, a difference of 0.016m was noted, indicating that the intake may not be horizontal.

Phil Salmon.

Phil Salmon
Surveyor

Extract from:

This is a confidential PRELIMINARY DRAFT manuscript

for A HISTORY OF THE WATER SUPPLY OF METROPOLITAN AUCKLAND

by G.J. Murdoch, ARA Planning Department

It should be quoted or copied in any way without the permission of:

Dr G.H. Campbell, Planning Dept
Mr R.J. Turner, Bulk Water Dept
Mr G.J. Murdoch, Planning Dept

The construction of the Lower Huia pumping scheme 1966-1971

The contract for the construction of the Lower Huia Dam and ancillary works was let by the Auckland City Council as agents for the Auckland Regional Authority in December 1966. Investigatory and preparatory work had however begun many years prior to this date. The Lower Huia catchment had been gauged since 1916 at which time Mr A.D. Mead had carried out a rapid survey of the area and further survey work was undertaken by Mr Joe Clarke after the completion of the Upper Huia project in 1928. By the mid 1930's the ACC had decided that an impoundment would ultimately be constructed in the Lower Huia Valley, and the necessary land had been purchased prior to the outbreak of World War II in 1939. A pumping scheme in the Lower Huia Valley was included in A.D. Mead's investigation into 'the future augmentation of water supply' in 1943 with further survey work and the collection of hydrological data being authorised at that time. The lack of finance and the staff shortages experienced during the War meant however that little work was carried out. By 1945 Auckland was facing major water shortages and the Council was keen to establish the City's next bulk water source as soon as possible. In the same year Mr J.R. Lee the ACC Chief Surveyor undertook some detailed survey work in the Lower Huia area and a potential dam site was roughly determined. At the same time Jack Lawrence of the Waitakere Headworks staff dug a number of test shafts on the dam site to help determine the suitability of foundation rock in the area. All further investigatory work was however postponed indefinitely in May 1946 when the ACC accepted A.D. Mead's recommendation to proceed with the construction of a dam at Cosseys Creek, Hunua, in preference to the Lower Huia scheme.

It was not until 1963 that the Council, taking the advice of the ACC Director of Works and Chief Engineer Mr A.J. Dickson and the Chief Engineer Water Supply Mr C.W. Firth, decided to return to the Waitakere Ranges to complete Headworks development in that area. A pumping scheme was to be constructed at Lower Huia following the completion of the Upper Mangatawhiri Dam in 1965 while at the same time site investigation and preliminary work was to continue at Wairoa and Mangatangi. In 1964 Mr Fred Bryant an engineer with the ACC began investigatory work at Lower Huia carrying out further dam site survey work, site materials investigations and borrow pit investigations. By early 1965 most of the proposed dam site had been cleared by local Headworks staff under the control of Mr 'Took' Moore and a large number of investigatory boreholes and shafts had been sunk as part of the geological investigation undertaken by Mr Cyril Firth. This investigation revealed that the 'Manukau Breccia' and 'Waitemata Sandstone' country rock at the proposed dam site would provide a good foundation and that suitable materials were available to construct 'an earth-rock dam'.

Investigatory work continued in early 1965 under the control of Mr Dave Hoyle who had replaced Fred Bryant on site, with both men coming under the overall supervision of Mr Cyril Firth. Mr Hoyle who was from the ACC Works Design Office continued on with materials investigations and with the design of the Diversion Tunnel needed to divert the Huia Stream prior to the commencement of dam construction. In March 1965 as the Upper Mangatawhiri project neared completion the Deputy Resident Engineer Mr Don Wilson was transferred to the Lower Huia Dam site as Resident Engineer and District Engineer Waitakeres. While Dave Hoyle busied himself with the design of the Diversion Tunnel Don Wilson began to organise establishment work for the project. Working from a small site office just opposite the Caretaker's house in the lower valley area his first task was to organise accommodation and the installation of services. A new site office was trucked to the site by Keith Hay Ltd and established on the western side of the Huia Stream just below the dam site. This task

however was not achieved without some difficulty because of the poor condition of the narrow Lower Huia Valley access road. While being transported the site office grounded at the sharp corner by Huia Bridge blocking the access road for three days. A soils laboratory was transported to the site from Moumoukai and located near to the site office and a septic tank was installed. Electric power was connected to both buildings and a water supply was connected to the office block, the laboratory, and to Mr Moore's house via a 2 inch pipe from the existing run-off-stream pump station located above the dam site,. A telephone was also installed, although until 1967 it was only available on a restricted hour basis through the Huia Store P.O. By June 1965 a house had been erected for the Resident Engineer Mr Don Wilson overlooking Huia Bay and in September two further Reid-built houses were transported to the same location by James Davern Limited.

The final task in the establishment phase was to improve access both within and to the Lower Huia site. The old Huia landing used during the construction of the Upper Huia Dam was recommissioned and the access track to it regraded in case the Contractor wished to transport materials to the project by barge. The Huia Valley access road was also widened and retarred, and a crib wall was constructed to stabilise a difficult section just above Huia Bridge. The most important job was the upgrading and sealing of the main road between Parau and the Huia Dam site. This was a joint exercise financed by the both the ARC (ultimately to the extent of \$277,000) and Waitemata County, with the contract being carried out by Green of McCahill Limited between 1966 and 1968. By late 1965 the establishment phase of the project was largely complete and the staffing of the project had been organised for the first stage of construction. Dave Hoyle was to continue working as a Senior Engineer on the site concentrating on the task of supervising work on the Diversion Tunnel and Jack Clapperton was appointed as Deputy Resident Engineer. John Stoll was Technical Assistant and was to assist Mr Wilson in a wide variety of administrative tasks as he had done since March 1965

The first major construction job on the Lower Huia project involved the tunnelling and lining of the Diversion Tunnel. Investigatory work and conceptual design for the tunnel was carried out by Dave Hoyle with detailed structural design being done by Mr C. Henson of the ACC Works Design Office. Design work was completed by July 1965 at which time contract documents for the job were drawn up. The Diversion Tunnel was of standard horse shoe cross section of 10 feet (3.04 m) equivalent diameter, and 607 feet (185 m) in length. In late July the decision was made by Messrs Dickson, Firth and Wilson to construct the Diversion Tunnel under a joint labour contract as had been used successfully for the Hunua tunnelling work. The Council was to provide a lunchroom and ablutions building as well as all tunnelling equipment and materials, while the tunnellers were to work at a set rate per foot. The joint contract was let to a gang of eight tunnellers with Carl Mann as principal contractor and Johnny Johnson as his 2 i/c. Work on the tunnel began in August 1965 and it was holed through eight months later in April 1966 without any major problems. The tunnel was lined by the same gang using form work fabricated in the Rockfield Road Drainage Department carpentry workshop, and ready mix concrete trucked to the job. It had been found after a brief trial period that this was cheaper than concrete mixed on site. Concrete was placed using a pressure concrete pump hired by the ACC from M.O.W. Turangi and all grouting work was carried out by Milne Construction Ltd. The Lower Huia Diversion Tunnel was completed by early December 1966 at which time the Huia Stream was diverted through it, thus dewatering the dam site and allowing preparatory work for the construction of the dam embankment to commence.

While the Diversion Tunnel was under construction Don Wilson organised further preliminary work. He also coordinated design work for the dam and associated structures, as well as the preparation of contract documents for the project. During 1966 access roads were formed

throughout the reservoir site and to the main borrow areas by Bunny Lawrence and Jack Rewa who contracted to the Council as owner-operators of their bulldozers. A small crusher from the ACC's Mt Eden quarry was also installed to produce concrete aggregate and graded metal for the Chimney Drain. However, because ready mix concrete proved to be cheaper, and there was insufficient suitable local rock for foundation drainage, it was never used. At this time a contract was let to B.J. Curtis and Co to clear the reservoir area of all remaining vegetation at a price of 167 pounds per acre. Prior to this all millable timber in the reservoir area had been surveyed and cutting rights sold to Henderson and Pollard Ltd. They extracted some rimu, miro and kahikatea, as well as 34 486 H.D. ft of kauri over the next twelve months, with most trees being used for peeler logs and poles. Mr Wilson's main task in late 1965 and 1966 was to coordinate design work for the dam and associated structures. Almost all of this work was carried out at the Lower Huia Site office, (except for the design of the Valve Tower), with on-site engineering staff doing all of their own design calculations and producing their own finished drawings.

The design work for the Lower Huia Dam itself was in the main undertaken by the R.E. Lower Huia Don Wilson with occasional advice from Mr Cyril Firth then newly appointed as Director of Works for the ARA. A civil engineering consultant Mr Peter Taylor was used during the investigation of borrow materials and the Deputy R.E. Jack Clapperton undertook some of the detailed design work for the dam abutments, foundation drainage, and reinforcement to the spillway tunnel.

By this time ACC and ARA engineering staff had gained a great deal of experience in the design and construction of controlled rolled-fill embankment dams during the Lower Nihotupu, Cossey's Creek and Upper Mangatawhiri projects. Mr Wilson was able to draw on this expertise and his own experience gained in the design of the Upper Mangatawhiri Dam. The location of the dam had been decided as a result of survey work carried out earlier, however Mr Wilson's first task was to site the dam exactly and to determine its shape. This was done taking into consideration the following factors: foundation conditions, the contour of the valley, the appearance of the dam in an area of high scenic beauty, and the best use of the main quarry located high above the western abutment.

Work on the structural design of the dam was the next priority. Earlier investigatory work had determined that there was sufficient local material available to construct a rolled fill embankment dam. This material was however considerably different to that experienced at Cosseys Creek and Upper Mangatawhiri in the Hunua Ranges. Therefore a number of different possibilities had to be investigated when deciding on an appropriate dam cross section. At this time, insufficient clay-silt material had been found to construct an embankment with an impervious rolled core as used by the ACC in its other rolled fill dams. Because of this a rockfill embankment was designed as a supporting structure for an impervious upstream face of reinforced concrete slabs, with joints sealed using annealed copper water stops and hot-poured rubberized bitumen. This type of embankment had not previously been constructed in New Zealand, although several had been developed successfully in the USA and sufficient literature was available indicating that this form of dam design was feasible at Lower Huia. An asphalt upstream face was also considered briefly, however this idea was rejected because of the possibility of phenols leaching into the water and tainting the supply.

As further materials investigation took place during the design phase it was revealed that sufficient local material was available with which to construct an impervious core of 'clay-silts' or 'clay-silts stabilised with Portland cement'. This allowed an embankment of conventional cross sections as used at Upper Mangatawhiri to be designed by Mr Wilson. Once the cross-

sections of both types of dam had been determined structural design could begin. This included design tests for foundations and structural stability against normal gravity forces, earthquake forces, and forces likely to be encountered by rapid drawdown of the lake reservoir. The design techniques used in the design of Lower Huia Dam were the same as those used in designing Upper Mangatawhiri Dam. At Lower Huia however a more accurate measurement of embankment pore pressure was available, thus allowing a better assessment of the strength of the embankment under load to be calculated. This was done using a Leonard-Farnell triaxial testing machine which had been purchased by the ACC for use in the design of Cosseys Creek Dam. It had proved to be difficult to use and had been abandoned, however Don Wilson employed his knowledge of electronics to modify it, thus enabling its full capabilities to be used giving more accuracy in design. In this era the main improvements being introduced in the design of rolled-fill embankment dams were in the field of internal dam drainage and in methods of measuring internal pressures in dams. A chimney drain, as used at Upper Mangatawhiri was incorporated in the design of Lower Huia Dam although a new innovation was also introduced. This was the use of a 'collector drain' which was to be laid at the base of the chimney drain to pick up any seepage through the core section of the embankment into the Chimney Drain. It consisted of a perforated concrete pipe which would take seepage out under the downstream shoulder to a measuring chamber at the toe of the dam.

By late 1966 design work for the Lower Huia Dam embankment was complete as was most of the design for the associated concrete structures. Tenders for the construction of a 'compacted fill dam and ancillary works' were called by the ACC acting as agents for the ARA in September 1966. The contract was to include the construction of either a compacted rockfill dam with reinforced concrete upstream face slab (Type A) or a compacted rockfill dam incorporating upstream and downstream shoulder zones as well as an impervious central core (Type B). It also included the construction of a reinforced concrete valve tower, Spillway and valve tower bridge. Four tenders for a 'Type A' dam were received by the ACC, ranging in price from \$2.5 million to \$3.3 million while three tenders for a 'Type B' dam were received. These tenders ranged from \$2.4 million to \$3.2 million with the Engineer's estimate for the favoured 'Type B' dam being \$2.9 million. On December 8 1966 the contract for the construction of Lower Huia Dam and associated structures was let to Green & McCahill Ltd for a price of \$2,491,822 which was nearly half a million dollars below the Engineer's estimate. After discussion between Messrs Firth, Smithson, Wilson and the contractor it was accepted however, as Green & McCahill were a reputable firm who had experience in the unknown factor of quarrying the andesitic conglomerate material to be used at Lower Huia.

Mr D.M. Graham Project Manager for Green & McCahill Ltd began establishment work immediately to enable the contractors to take advantage of the summer earthmoving season. At the same time further staff were added to the ACC team by R.E. Don Wilson. Jack Clapperton remained as Deputy Resident Engineer, and like Senior Engineer Dave Hoyle he was resident at Lower Huia. John Stoll continued as Technical Assistant doing clerical work, simple design calculations and a small amount of survey work. He also acted as 'mailman' for the project travelling to work each day via the City office. Graeme Barnard was placed in charge of the Soils Laboratory and was to be 'Embankment Engineer'. To assist him Pieter Wieringa was recruited as 'Soils Laboratory Technician' from the M.O.W where he had been employed on the Matahina Dam project. Survey work was to be the responsibility of ACC Surveyor Phil Salmon. In early 1967 Green & McCahill established their work camp on site. A Site Office was located adjacent to the ARA Site Office and a large workshop was erected near the Diversion Tunnel outlet. By late 1967 a singlemen's camp consisting of approximately 40 huts had been established near the Office Blocks on the site of a former Church Camp.

In 1967 Green & McCahill concentrated their efforts on stripping the dam site of overburden, opening the main rockfill quarry site, as well as on the excavation of the horizontal section of the Spillway Tunnel. Most access roads to work areas had been developed by the ARA prior to the letting of the contract, however the Contractor had to form the steep access road to the quarry located high above the Western abutment. Green & McCahill slowly formed the road up the slope by dozer. They then worked in a bench at the highest level using an Atlas Copco ROC 300 crawler drill powered by an air compressor of the same make. By November 1967 the quarry had been stripped and was ready for use. Major progress had also been made in stripping both the dam foundation area and the abutments. The project was however to receive a major setback when a major storm hit the area in mid November 1967. A severe rainstorm in February 1966 had held up work on access roads and another in June 1967 also held up preparatory work when the coffer dam above the Diversion Tunnel was briefly breached. Between November 13 and 16 1967 thirteen inches (330 m.m) of rain fell in the Huia catchment causing widespread damage in the district. As a result the upgrading work being carried out on the Parau-Huia Road was seriously disrupted and the foreshore road between Huia and Little Huia was completely cut by slips and erosion. The flood proved to be too much for the Diversion Tunnel coffer dam and flood waters poured across the dam site disrupting excavation work in the foundation area. The flood also disrupted some of the huts in the Contractor's singlemens camp, lifted the underground fuel storage tank, and damaged some earthmoving machines.

By this time work had also begun on the excavation of the vertical section of the bellmouth spillway and the storm caused formwork and scaffolding erected for this purpose to collapse, thus holding this important job up for several months. The spillway tunnel and bellmouth section had been designed by Jack Clapperton in 1966 and consisted of a 20 feet (6 m) diameter tunnel 380 feet (114 m) in length, a vertical shaft 130 feet (39 m) in height, and a bellmouth inlet structure 70 feet (21 m) in diameter. By early 1968 the excavation of both the spillway tunnel section and vertical shaft had been completed by Green & McCahill and tunnel lining had begun using formwork fabricated at the Hopetoun Street Workshops. A rough concrete slab was first poured on the tunnel floor and then the cylindrical steel lining formwork was run into the tunnel on rails. Concrete for the lining of the Spillway Tunnel was mixed at a batching plant located near the tunnel outlet. It was then fed by conveyor belt to a concrete pump from which it was pumped to the area of the tunnel where lining was taking place.

Initial difficulties were faced in pumping the basalt aggregate being used, however after a switch was made to McCallums aggregate lining of the tunnel section proceeded rapidly and had been completed by mid 1968. Lining then began on the elbow section of the spillway which was poured using wooden formwork and on the vertical shaft using metal shutters. By November 1968 the spillway had been completed to ground level.

Throughout 1967 and 1968 Green & McCahill worked steadily on stripping the dam foundation area back to the sandstone base under the core zone, and to suitable firm ground under the shoulder zones. All topsoil, root remains and unsuitable fill material was removed, and then transported to two large strippings dumps located just downstream of the dam on the western side of the Huia Valley Road. Stripping the abutment areas had proved to be relatively straight forward, however unforeseen problems were encountered in stripping the old stream bed. Here a dragline was used to excavate approximately 100,000 cu yds. of swampy material not previously detected. This created a huge hole which then greatly increased the amount of fill material needed and added \$680,000 in cost to the project. By late 1968 the entire dam foundation area had been cleared to foundation levels. Shattered or seeping areas in the core zone were then carefully cleaned using hand tools and grouted or sealed with dental concrete. By early January 1969 preparatory work in the embankment

foundation area was complete so that the placing of fill could begin during the coming summer season.

During this period, stripping of topsoil from the clay-silt borrow areas had revealed that sufficient clay materials could be extracted to permit a conventional compacted clay core to be used, rather than the cement - stabilised type tendered on. In addition to this, tests on materials extracted from the quarry using the heavy machinery now on site, revealed that conventional spreading and compaction techniques could be employed rather than the tipping and sluicing method tendered on. Acceptable unit rates for these changes were negotiated with the Contractor, and construction proceeded accordingly on a 'Type C' conventional rolled-fill dam incorporating a clay core.

While the contractors had been busy on the main Lower Huia Dam contract the ARA had called tenders for another important part of the project, the construction of four Huia Valley Access Bridges. They were to be constructed to carry the rising main and permanent access road over the Huia Stream and its major tributaries on the western side of the valley. Bridges were to be built over Stoney Creek, Clay Creek and Huia Stream, both to carry the new rising main between the Huia Pumping Station and the Upper Huia pipeline, as well as the new access road to the Upper Huia Dam. A bridge was also to be constructed across Georges Creek to replace the old rail bridge that would ultimately be submerged under the waters of the new reservoir. These four bridges totalling 460 feet (140 m) in length were designed by Ray Turner of the ARA Water Design Office using the latest design techniques. All four bridges followed similar design having reinforced concrete piers and abutments, pre-cast and pre-stressed beams, and pre-cast running planks bolted to the beams. The earthworks for both the new road and the rising main were formed up the Huia Valley by Jack Rewa using a TD 9 dozer in early 1968 with help from Vuksich & Borich Contractors Ltd. This included the construction of a major diversion over 'Tunnel Point' where the old road would be lost under the lake. The contract for the construction of the Huia Valley access bridges was let to Milne Construction Co Ltd and they began setting out work in February 1968. The piers and abutments had been poured by September 1968 and the beams pre-cast by Canal Construction placed in December 1968. By mid 1969 all of the bridges had been completed, the rising main installed as far as 'Smith's Track', and the road metalled.

The construction of the Lower Huia Dam embankment proceeded relatively smoothly over two placing seasons during which time 1,200,000 cubic yards (917,500 cu. m) of fill was placed and compacted. After the core zone foundations had been cleaned meticulously a twelve foot thick clay bonding and levelling layer was placed directly on the foundation rock to give an impervious seal between the core and the sandstone base. A shallow core trench was then excavated across this material and the perforated concrete pipe drain laid along this at the base of the vertical chimney drain. Several transverse pervious rock strip-drains were also laid across the downstream foundation in order to pick up any foundation seepage that might occur downstream of the core. These strip drains fed into an open pervious rock drain that ran alongside the reinforced concrete drain taking seepage from the Chimney Drain to the Measuring Chamber at the downstream toe of the dam. The Chimney Drain itself was placed as the dam rose. It was two metres in width and consisted of a narrow filter zone of intermediate graded scoria and a wider drainage zone of open graded scoria.

During the first placing season which extended from January 22 1969 until May 1969 150,000 cu yds of material was placed and 75,000 cu yds of rockfill was stockpiled for placement in the dam shoulder zones during the next season. The core zone was constructed using river silts and clays taken from river terraces located in the valley floor just upstream of the dam. Material was taken from this area by CAT 631 carryalls push-loaded by

dozer, transported to the embankment, and spread in layers. Core material was then worked by grader to the correct level which was determined by the embankment engineer. Once in place the clay-silt material was then compacted using a sheep's foot roller towed by a dozer.

The construction of the shoulder zones of the embankment was a far more time consuming process. Material for the dam shoulders consisted of compacted conglomerate rockfill extracted from the main quarry which was benched into the rock face high above the western abutment. At the quarry rock was drilled, shot, then ripped by CAT D9 dozers which often worked into the night to ready material for placement the following day. This operation ran smoothly during the first placing season, although progress was not as fast as expected because of the constant problem of the single-tine rippers used behind the CAT D9 dozers being bent by conglomerate 'goolies' encountered in the quarry. Rockfill was picked up 25 cu yds at a time by 35 ton CAT 631 carryalls that had been specially imported for the project by Green & McCahill. They were push-loaded at the quarry by the CAT D9 dozers and then descended the winding road to the embankment where they spread their load. They then returned up the steep 3.8 in 1 slope to the quarry, with the whole operation taking approximately ten minutes. After initial spreading by the CAT 631's the rockfill was then spread more accurately by either a Michigan loader, a Euclid C100 or a CAT D7 depending on the location on the embankment and the availability of machinery. Once in place the coarser shoulder material was compacted using a PAC-ACE vibrating roller which proved to be a great improvement on compacting machinery used previously in the construction of rolled fill embankments by the ACC. When compacting placed material in areas adjoining the abutments or around concrete structures, a rubber tyred loader, and even a hand operated pneumatic compactor were used. When the moisture content in the fill material was too low it was watered using a Euclid water tanker and when it was too high the material was disced.

This cycle of placement ran according to plan during the first season with the only major problems being the difficulties encountered in extracting the conglomerate rock from the quarry and in negotiating the steep quarry access road after wet weather. The latter exercise was not only difficult but also dangerous as was illustrated by a tragic accident that occurred during the first placing season. On the Sunday afternoon of March 16 1969 the project was marred by the death of one of Green & McCahill's operators. Mr Jack Wikaire aged 24, a married man with three young children, was killed when the brakes failed on the Michigan loader he was driving while descending the quarry road and his machine fell to the valley floor below. Only seconds later a work mate David Te Wake suffered a serious leg injury when the truck he was driving to summon help skidded and overturned on the partially completed embankment.

Setting out for placement on the embankment was carried out by Green & McCahill staff who also directed the placement of fill. Graeme Barnard the ARA embankment engineer generally oversaw the placement of fill material while ARA Surveyor Phil Salmon checked the Contractor's setting out. Mr Salmon also surveyed a monthly cross section of the embankment to calculate the quantities of fill placed thus allowing Jack Clapperton to calculate progress payments. All of the on-site ARA engineering staff assisted the embankment engineer by taking their turn in overseeing placement during the busy summer season and by doing duty every fourth weekend. This included Deputy R.E. Jack Clapperton and R.E. Don Wilson who at one stage did a stint of seventeen consecutive days supervision on the embankment during the summer of 1969-70 so that other staff could take their annual leave. The engineering staff supervising work on the embankment had to ensure that loads were correctly graded and spread, and that they were compacted in the appropriate manner depending on the type of material. They also had to ensure that the embankment cross section was correctly shaped so that in the event of rain it could be quickly sealed off to allow the rain to be shed, especially on the clay core zone.

As in the construction of all controlled rolled-fill dams the work of the Soils Laboratory was under the overall supervision of Graeme Barnard. Its daily operation was however in the hands of the 'Soils Technician' Pieter Wieringa who also played an important role in embankment supervision having been in charge of placement quality control on M.O.W. Matahina Dam. Several other staff were also employed permanently in the Soils Lab and additional staff were taken on to assist at the height of the placing season. Most of the actual laboratory work was carried out by 'Soils Assistant' Jim Harbridge and later by a local resident Barry Miller. Gary Dilly a junior ARA engineer also carried out soil testing for a short time, both in the laboratory and on the embankment as did another local headworks staff member John Walsh.

Standard materials tests were carried out as at Upper Mangatawhiri to establish correct grading, permeability, shear values and moisture content. Liquid and plastic limits were also established and Standard Proctor tests were undertaken to ascertain 'Proctor optimum moisture content' for core material both in borrow areas and on the embankment. This procedure was however modified at Lower Huia because the coarser shoulder material encountered was too large for the standard 4 inch Proctor cylinder. As a result the U.S. Bureau of Reclamation relative density test was adopted using moulds and a vibrating table specially fabricated for the project at the Hopetoun Street Workshops. Quick tests of shear strength were also made on the embankment during placement using an unconfined shear machine.

As work on the embankment proceeded throughout 1969 a great deal of progress was also achieved in the construction of the associated concrete structures. The bellmouth section of the spillway was completed, the downstream concrete collector outlet drain was installed, the Valve Tower structure was constructed, and design work for the Spillway outlet structure was carried out. By February 1969 the vertical section of the bellmouth spillway was completed to crest level using a tower crane to lift concrete and other materials. Over the next few months the curved bellmouth itself was constructed using wooden shutters, the guide vanes were poured, and supports for the access bridge to the Valve Tower were formed on the top of the bellmouth. The 36 inch (910 m.m) collector drain encased in reinforced concrete had also been completed as far as the Measuring Chamber by February 1969 and several layers of piezometers had been placed into position in the embankment.

The main development in the construction of concrete structures at Lower Huia during 1969 was the completion of the Valve Tower. This structure, which was 110 feet (34 m) in height and 14 feet (4 m) in diameter, was designed by Mr R. Jones of the ACC Works Design Office in 1966, with detailed design of standpipe details and inlet screens being undertaken on site by Jack Clapperton. The Lower Huia Valve Tower was designed along conventional lines (as in all previous ACC dams) being a dry well, vertical tower structure. An improvement on the Upper Mangatawhiri Valve Tower design was however made in the design of the intakes where the stainless steel screens were enlarged and shaped so that they would better fit the stainless steel guide horns. The reinforced concrete base for the Valve Tower was completed by Carl Mann and his tunnelling party (who had previously built the Diversion Tunnel) during February 1969 under the supervision of Dave Hoyle.

The tower structure itself was constructed between March 27 and 29 1969 using the 'slipforming' process that had been a major success in the construction of the Upper Mangatawhiri Valve Tower. The 'slipforming' of the tower took place continuously over 24 hours at a rate of approximately 1 ft (300 mm) per hour. It was carried out by subcontractors Wilkins & Davies Ltd with Northern Steel Ltd placing the reinforcing steel. The job was supervised by engineering staff from both Green and McCahill Ltd and the ARA who ensured that reinforcing was correctly placed, that concrete was correctly compacted and of specified quality, that blockouts were correctly placed, and that the structure was truly aligned. Concrete used in the slipforming process was trucked to the site and then placed directly from an Acrow Coneflow Concrete bucket lifted by a NCK 50 mobile crane. The concrete was placed in 6 inch layers in the moving form which was attached to jacking rods secured to the completed section of the tower, with timber blockouts being carefully placed where

needed for the inlets. The pipework for the tower had been cast by Swanson Foundry Ltd New Plymouth and was installed by Green & McCahill who also fitted the interior ladders and landings that had been fabricated in the Water Department's Hopetoun Street Workshops. They also fitted the stainless steel screens and guide horns that had been fabricated in the ACC Water Department Hopetoun Street Workshop. The crest of the Valve Tower was constructed in mid 1970 and by late 1970 all lighting and fittings had been installed and the scour works had been completed by ARA staff.

While work proceeded smoothly on the construction of the concrete structures during the first placing season, problems had arisen with the reservoir clearing contract which had been let to B.J. Curtis & Co in 1966. The contract had proceeded slowly throughout 1967 and in mid 1968 a formal complaint was made by the ARA to the Contractor. A new gang was started on the job and some of the ARA headworks staff were employed to clear the difficult sections in the upper catchment area. Problems continued throughout 1968 and into early 1969 when the Contractor went into voluntary liquidation. At this time the contract was wound up and ARA staff took over the task of clearing the remaining vegetation from the reservoir area.

By mid 1969 excellent progress had been made on another important facet of the Lower Huia project, the installation of the rising main between the pumping station below the dam and the Huia Aqueduct. This main, which consisted of 1.93 miles (3.1 km) of 32 inch (812 m.m) and 0.45 miles (0.72 km) of 36 inch (910 mm) of concrete lined, tar and enamel coated, all welded steel pipe, had been installed as far as the tunnel between Huia Valley and Nihotupu Valley by June 1969. Work then began on the replacement of the 40 year old 27 inch (685 m.m) steel locking-bar main across the Nihotupu Valley with a 39 inch (990 mm) main. Both mains were laid by an ARA Hopetoun Street pipegang, although culverts for the pipe route and access roads in the Nihotupu Valley were constructed by Bill Beveridge, Steve Schischka, and Mike Malone, with occasional help from other men from the Parau Depot. Main laying was supervised by Foreman Graham Boag with Peter Rameka acting as ganger. Hepi Rudolph operated the Austin-Western mobile crane used to position the pipes and shared the operation of the 22 RB digger with Andy Noa. Other members of the gang included: Oue Oa'ariki, Tom Metuariki, Ken Jacobs, Jack Smith, and Cyril Miru who also acted as a driver transporting the gang to and from the job each day. Barney Poka trucked in all of the pipes from Humes Ltd, often working seven days a week, and all pipewelding was carried out by Abe Hunia and Bill Kem.

The pipegang often worked in relatively inaccessible localities and were unable to truck in a hut to use as a shelter and cookhouse. Proving that the pioneering spirit was still alive, a nikau cookhouse was constructed in the traditional woven manner by the four Cook Island members of the gang, one of whom Nga Keu also acted as cook. This structure was much appreciated by the gang, not only as an eating place but also as a shelter during what was often a wet winter job. In winter conditions Peter Rameka's gang made remarkable progress with the Huia Valley section of the rising main having been completed by June 1969 and the whole main by late 1970. In April 1970 the gang created a new ACC-ARA pipelaying record while working in the Nihotupu Valley. The ARA Journal announced, "beating the previous record, created in March 1969 of 4129 ft of 32" pipe, in the last four weeks this gang has laid 5435 ft of 39" interior diameter concrete lined steel pipe". **(2)**

Prior to the commencement of the second placing season Green & McCahill Ltd applied for an extension of the project completion date from April 30 1970 to April 30 1971. This request was granted by the ARA because most of the delay was due to circumstances that were beyond the Contractor's control, and it was obvious that the contract could not realistically be completed by the original date. At this time Green & McCahill also informed the ARA that they were growing increasingly concerned about the economic viability of the project. They had found that quarry production was far more expensive than they had estimated because of difficulties experienced in extracting the conglomerate rock. It was also noted that machinery costs had greatly increased as a result of the November 1967 currency

devaluation, thus increasing the cost of the CAT 631's and other machinery specially imported for the project.

Because of the problems they were facing the Contractors lodged a claim for an increase of 23.3 cents per cubic yard of quarried rock for the second placing season. The ARA responded by stating that the Contractors "were expected to first complete the contract, after which the questions of costs and losses could be discussed." **(3)** The contract continued along this basis throughout the second season, however in January 1971 Green & McCahill made a formal request to be released from the contract stating that they faced a projected loss of \$380,000 which would put them out of business. C.W. Firth, the ARA Director of Works informed the Authority that it was essential that the Lower Huia project be completed by April 30 1971 in order to meet forecast demand. He suggested that he be authorised to negotiate an agreement with the Contractors to enable them to complete the contract. A payment of \$50,000 was made for all material quarried and an offer was made of five cents per cubic yard additional payment for all rockfill quarried after January 1971. The Contractors accepted this agreement and reorganised their quarry operations. Barney McCahill one of the Company principals joined the project and took control of the quarry often operating a CAT D9 in the quarry himself. This revitalised the project and greatly improved the economic viability of the project for the Contractors.

The second placing season had proceeded satisfactorily with the embankment being 62% complete and sufficient fill material having been stockpiled to ensure the completion of the job in the coming season. Finishing work was now proceeding on the Valve Tower and Valve Tower access bridge and work had begun on the spillway outlet. Major progress had been made on the expansion of facilities to cope with the treatment and distribution of the additional water that would be made available when the Lower Huia Dam was commissioned. The most important development in the construction of the associated concrete structures in 1970 was the work carried out on the spillway outlet stilling basin. Prior to the letting of the main Lower Huia contract design work had been completed for all parts of the spillway except for the outlet structure. Early design work for the dam spillway structure carried out by Jack Clapperton and Don Wilson had shown that a standard type of 'standing wave' stilling basin would have been far too large for the site. After a great deal of consideration Don Wilson came up with the idea from a German text on hydraulic structures of using an 'energy dissipator' at the Spillway Outlet. This idea was adapted to produce an outlet structure much shorter than either the type of outlet used at Upper Mangatawhiri or a conventional 'standing-wave' basin.

By mid 1968 a model of the spillway outlet structure had been constructed by the Rockfield Road carpenters to Clapperton and Wilson's design. It consisted of a 'Schoklitsch Bucket' stilling basin incorporating 'dragon's teeth' or baffle blocks designed to produce turbulence and energy dissipation. The model was tested on site by Jack Clapperton and Graham Barnard with advice from Professor Raudkivi the Senior Lecturer in Hydraulics at the Auckland University School of Engineering. By late 1968 the final design had been adopted and the structure designed by Jack Clapperton. Detailed structural design was then carried out by members of the ARA Works Design office now under the control of Mr Warwick Clay. Staff involved in this detailed design work included: Colin Bell, Ray Turner and Ngoyan Tan Do. The excavation of the site had been completed by late 1968 and work on the structure had begun in October 1969, with the then quite innovative structure being completed by December 1970.

While work was proceeding on the spillway stilling basin, construction of the Lower Huia Pump Station was also taking place. The pump station building sited immediately downstream of the dam was designed by Dave Hoyle in early 1969 and constructed by Van Driel and Blanken Ltd between February and October 1970 for \$28,643. Dave Hoyle also designed the pumping system needed to lift water up the 9000 ft (2745 m) rising main between the pumping station and the Huia gravity aqueduct. The pumping station incorporated six electrically driven pumping units. They included five Thompsons Ltd 2 stage electrically driven pumps of 4 m.g.d capacity, each being powered by Newmans 380/420 V 300 h.p motors. There was also one 4 stage centrifugal H. Wernert pump of 3.2 m.g.d

capacity powered by a Brook Motors 400 V 300 h.p motor. It is of interest to note that this pumping unit had been used from 1963 to pump from an auxiliary weir in the Lower Huia Valley via a 12 inch pipe to the Upper Huia gravity main. As it was compatible with the Lower Huia pumping scheme it was installed in the pumphouse to save the cost of a new unit priced at nearly \$4000. Electrical installation work was undertaken by Crusade Electrical Ltd while all other installation work in the pumphouse was carried out by Jim Orr and his ARA gang based at the Hopetoun Street Depot.

Another key part of the Lower Huia scheme that had begun to be developed in conjunction with the construction of the Dam was the upgrading and expansion of the Huia Filter Station at Titirangi. The Station, originally constructed in 1929, had been expanded from 8 m.g.d to 14 m.g.d capacity in conjunction with the development of the Lower Headworks in 1948. It now needed to be further expanded to take the additional 12 m.g.d peak flow that would be made available with the commissioning of the Lower Huia scheme. Mr C.W. Firth the ARA Director of Works noted that, "the enlarged Huia Station will deal with 67% of the total yield from the Waitakere catchments. This predominance and its consequent importance in controlling quality and supply of Waitakere water means that protective features appropriate to this role must be provided, in addition to increased filter capacity as such." (4) He then outlined a proposal to achieve both of these aims at an estimated cost of \$1.78 million. His basic proposal was accepted by the Authority and the contract for the necessary layout design work and equipment supply was let to PCI (NZ) Ltd in mid 1968. The upgrading of existing facilities began in 1968 under Mr Wilson's overall direction and a pilot treatment plant was established. Construction proper began in late 1970 and continued for the next six years. The details of this upgrading and expansion programme are covered later in the section dealing with developments in Water Treatment 1966-77.

In order to cope with the increased volume of treated water from the Lower Huia scheme it had also been necessary to consider the whole question of increasing service reservoir storage capacity at Titirangi. In the 1968 loan proposal assembled for the Lower Huia scheme development, provision had been made to extend the water storage associated with the Huia Filter Station which was then limited to the 1 million gallon (4630 cu. m) capacity Titirangi No 1 Reservoir constructed in 1947. As C.W. Firth informed the ARA Works Committee the existing clearwater storage was, "inadequate for effective control. It frequently restricts station operation, instead of being the means for maintaining steady through put and maximum efficiency irrespective of variation in demand on the transmission system." (5) It was obvious that a major expansion in clearwater storage was urgently needed and it was proposed to construct a 8 million gallon (37040 cu. m) capacity service reservoir immediately adjacent to the Huia Filter Station. Site investigatory work revealed however that the site was an old slip face with no solid foundations for a depth of 90 feet (27.4 m) quite unsuitable for a large reservoir project.

Because of the poor foundation conditions beside the Filter Station and the lack of suitable flat land in the surrounding area a decision was made to construct a smaller reservoir beside the existing Titirangi No 1 Reservoir in Konini Road. The use of this site also meant that the ARA could avoid the construction of a second supply tunnel under the Titirangi ridge that would have been necessary if the original site had been developed. The reservoir site was inspected by the RE Lower Huia Mr Wilson and surveyed by Phil Salmon, with Colin Bell carrying out foundation investigatory work. The site was found to be suitable for a reservoir of up to 4 million gallon (18520 cu. m) capacity, although it was discovered that a neighbour's house, septic tank and swimming pool were partly located on the ARA owned site. A swap at land was then arranged so that the owner retained his house and pool but had to construct a new access road. Because of the close proximity of the adjoining house it was also noted that extensive stabilisation work would have to be carried out in conjunction with excavation for the reservoir.]

Preparatory work for the construction of the 'Titirangi No 2' Reservoir began in early 1970 under the site supervision of Garry Dilly, a junior engineer working on the Lower Huia project. The site was cleared by ARA Water Department headworks staff. The underlying sandstone was then excavated to a depth of six metres and replaced with rock fill to provide a solid

foundation for the reservoir. By this time design work for the 3.5 million gallon capacity service reservoir had been completed by Mr Ray Turner of the ARA Water Design office. It was a reinforced concrete, partly buried, structure with conventional walls, floor and roof columns. The reservoir did however include a new 'shrinkage compensated' roof design which was incorporated to overcome the perennial problem of cracking in concrete reservoir roofs. The roof was to be prestressed by post tensioning after being built free from the walls, until shortening due to prestressing, and shrinkage due to concrete drying, had taken place. The roof and walls were then to be locked together by a concrete joining strip continuously poured when temperatures were relatively low. This innovative design devised by Mr Turner proved to be relatively successful in overcoming past problems encountered in service reservoir roofs and was subsequently incorporated into many other service reservoir designs over the next decade. Design work had been completed by July 1970, however the tenders that were received were well above the Engineer's estimate, and it was some time until the tender of Fletcher Construction for \$417,158 was accepted. Work on the reservoir began in early 1971 and it was completed without major problems in mid 1972 under the site supervision of ARA Engineer Roy Headland.

The construction of the Lower Huia pumping scheme not only necessitated an expansion in treatment capacity but also the installation of a new trunk main between Titirangi and Khyber Pass Reservoir. This was necessary in order to carry the new peakload capacity of 12 m.g.d that would be available from 1971, and also to replace the old 27 (685 m.m) inch trunk main along Great North Road as well as the old 33 inch (838 m.m) main that ran along the railway line beyond New Lynn. The new trunk main consisted of 8.9 miles of 51 inch (1300 m.m) all welded, concrete lined, steel pipe. It was also to include a direct feed into the North Shore feeder from a branch crossing Newton Gully to Ponsonby. The installation of this main was a major job and was carried out between 1970 and 1975 by both ARA pipelaying gangs and outside contractors under the supervision of ARA engineering staff. It is described in greater detail in the section covering advances in bulk water distribution 1966-77.

While work was proceeding satisfactorily on the various contracts associated with the main Lower Huia Dam contract, the construction of the embankment was also on schedule. The final placing season had begun in September 1970, and although progress was initially slowed, placing continued without hitch throughout the 1970-71 summer and by mid March 1971 the core had been completed. On March 26 1971 the last load of shoulder material was placed and compacted and the 1.2 million cu yd (917500 cu. m) embankment had been completed twenty six months after placing had begun. During this last phase of construction the ancilliary structures were also completed. This included the final fitting out of the Valve Tower, the completion of the Valve Tower Access Bridge and Spillway Stilling Basin, and the installation of the reservoir scour pipework and outlet. The scour pipework was conventional, consisting of a 780 ft long 24 inch (600 m.m) CLS pipe running from the base of the Valve Tower via the diversion - delivery tunnel to a downstream outlet. The scour outlet included what was a new device for Auckland waterworks - a 'Jet Disperser Unit'. This sleeved free discharge valve, supplied by the English Electric Co Ltd of NZ, had a moveable sleeve which allowed water to discharge around a central 90 degree cone forming a hollow conical spray of water. This valve could operate against a high head, and was a major improvement on the conventional gate valves which had been used previously and which so often proved almost impossible to use. This was a precedent which was followed in modified form at Wairoa and Mangatangi.

Soon after placement had finished on the dam embankment the R.E Lower Huia Don Wilson was transferred back to Regional House to his new position of Assistant Chief Engineer Water, and Jack Clapperton was placed in charge of finishing work at Lower Huia. In May 1971 the Diversion Tunnel was plugged allowing the dam to begin filling while rip rap was being placed on the upstream shoulder. In July the Soils Laboratory was moved to Wairoa Dam site and the Contractor's Camp and Camp Kitchen demolished. At this time Mr Clapperton moved back to Regional House along with the other ARA engineering staff who had been supervising the main contract. John Stoll remained on site to handle clerical and administrative duties, and Harvey Stewart was appointed ARA Site Representative to

oversee the completion of the siteworks and cleaning up. The three months prior to the scheduled opening in September was a hectic time. The placing of rip rap on the upstream dam face was completed, berm drains and sumps were installed, the wave break was placed on the crest. All bare areas were hydroseeded and extensive planting was carried out by Headworks staff with advice from the ARA Parks Department. Roding and fencing work was finished on the dam and roding was carried out at fever pitch right until opening day. The Lower Huia Dam was officially opened on September 30 1971 in a simple ceremony attended by approximately 100 people including representatives of a number of Auckland local bodies. Speakers included Mr L.I. Murdoch Chairman of the ARA Works Committee, Sir D.M. Robinson Mayor of Auckland, Mr J. Colvin Chairman of the Waitemata County Council, and Mr T. Pearce Chairman of the ARA who officially opened the dam and unveiled the commemorative plaque. Detailed finishing work continued until November when the project was completed at a final cost of \$2.92 million. The completion of the Lower Huia Dam added an average design yield of 8 m.g.d (36,000 cu. m.d) and an estimated maximum yield of 12 m.g.d (54,500 cu. m.d) to the ARA bulk water supply system. Its commissioning completed the Headworks development programme that had begun in the Waitakere Ranges seven decades earlier and attention now returned to the completion of the Hunua Headworks system.