Wilsons Dam: Design expectations compared with measured performance

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Wilsons Dam is an 18 m high, zoned earthfill water-supply dam, constructed for the Whangarei District Council to supplement the Bream Bay supply. The dam is located on the Waiwarawara Stream close to State Highway One at Ruakaka. The stream catchment area is relatively small (3.9 km^2) and a viable water supply in the Northland climate required seepage losses to be minimised. The valley floor comprises a 15 m deep sequence of soft alluvial silts and clays interbedded with permeable sand and gravel lenses.

The foundation conditions raised a number of issues for the design of the dam, including seepage; construction stability; settlement; seismic stability; and liquefaction. These were addressed by a range of design provisions that included: wick drains, cement-bentonite slurry cut-off wall, wide berms on the upstream and downstream shoulders, highly plastic core, staged construction, and instrumented monitoring of foundation performance.

Construction took place between October 2000 and July 2002 and was hampered by continual wet weather throughout 2001. First filling of the reservoir is expected to be complete by October 2003. This paper compares some design expectations with dam performance to date and presents key conclusions that: settlements were predicted adequately by conventional consolidation theory, and; monitoring and review of stability during construction allowed construction to proceed safely whilst maximising reservoir storage volume.

Keywords: *soft foundation, instrumentation, slurry cut-off wall, seepage, construction stability, settlement, wick drains.*

Introduction

Wilsons Dam is an 18 m high, zoned earthfill embankment constructed on the Waiwarawara stream in Ruakaka, approximately 25 km south along SH1 from Whangarei (Figure 1). The reservoir forms part of the Whangarei District water supply network and stores 2.4 Mm^3 for the Bream Bay area. Key dam statistics are shown in Table 1.

Bream Bay is the second largest water supply area within the Whangarei District. The area is predominantly coastal, and is subject to seasonal fluctuations in water demand, between 6000 and 11 000 m^3/day. The area is fully metered with 2328 water connections, including the district's largest consumer, the New Zealand Refining Company at Marsden Point, which uses between 4000 and 6500 m^3/day. The area is experiencing high growth and has significant zonings for both industrial and coastal residential areas. A new port at Marsden Point has recently complemented the industrial zoned areas and there has been significant industrial growth associated with the timber industry. The unaccounted-for water in the Bream Bay Water Supply Area is 9%.

The geological setting includes abutment ridges of highly weathered greywacke (Waipapa group) forming a V-shaped valley, infilled with sediments of Pleistocene age. The main borrow was in Tertiary siltstones, which have largely weathered to plastic clays.

The presence of deep soft soils, with permeable lenses, in the dam foundation meant that special consideration had to be given to several aspects of the design. The dam profile and construction programme were dictated by the need to avoid overstressing the foundation. A cement-bentonite slurry cut-off wall was used

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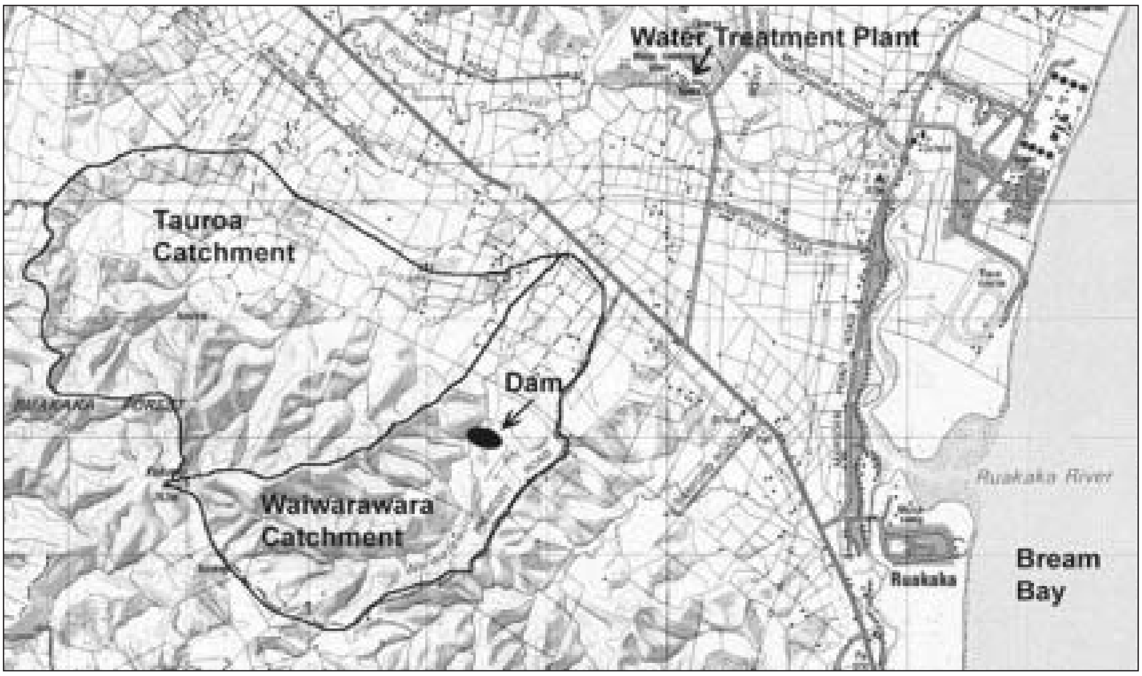

Figure 1. Location of Wilsons Dam, Ruakaka, Whangarei District.

Table 1. Summary of dam features, Wilsons Dam, Ruakaka.

Dam height & crest length	18 m × 200 m
Reservoir capacity	2.6 Mm^3 (2.4 Mm^3 live storage)
Earthfill volume	210 000 m^3
Service spillway	Uncontrolled 3.9 m diameter bellmouth
Diversion	2.55 m dia concrete pipe, combined with spillway on left abutment
Auxiliary spillway	60 m wide grassed channel on right abutment
Outlet Pipework	375 mm dia multi level offtake and 3.5 km delivery line to Ruakaka WTP
Owner	Whangarei District Council
Designer	MWH Ltd
Constructor	Roadstone Construction Ltd

to restrict seepage in the near-surface permeable layers. Settlements in the order of one metre were expected and dam components have to be sufficiently plastic to accommodate the movement without cracking.

Instrumentation was installed to record settlement, lateral displacement and pore pressures. Monitoring of the instruments provided the ability to maintain construction stability by controlling rates of fill placement.

Site selection

The Wilsons Dam site was first investigated in 1978 by the former Whangarei County Council in a report covering five potential supply schemes for the Bream Bay Water Supply Area. The report recommended that the Wilsons Dam catchment together with one other catchment should be investigated in more detail. This was eventually undertaken in 1987 when the Council commissioned an Engineering and Environmental impact report. The report concluded that Wilsons Dam was the preferred option, based on economic and environmental issues, together with its close proximity to the Ruakaka Area.

In 1988 the Council obtained a water right to dam and take water from the Waiwarawara Stream (Wilsons Dam Catchment), and in 1996 the Whangarei District Council (WDC) received resource consent to dam and abstract water. Between 1988 and 1995, WDC collected rainfall and stream flow data and commenced purchasing land within the proposed lake area.

WDC undertook a strategic review of its public water supply in 1998. The primary objective of this study was to rationalise the six independent Water Supply Areas within the Council's jurisdiction and to improve water security up to a 1-in-50-year Return Period Drought supply capability.

The Strategic Plan for Water Services covered a number of issues, and identified the Bream Bay Water Supply Area as the most at risk in terms of supply security. Prior to the inclusion of Wilsons Dam, the area was supplied from three run-of-river sources; the Ruakaka River; Ahuroa River; and the Pohuenui River. These sources are relatively small and only provide a 1-in-4-year Return Period Drought supply security when based on current resource consents and only an average year's security when evaluated against the Revised Proposed Water and Soil Plan. A number of options were considered, including connecting the Bream Bay Water Supply Area to the Whangarei City Water Supply Area (which has a 1-in-30-year Return Period Drought capability). However, the City Water Supply Area is also in need of an additional water source, and the pipeline costs were estimated to be in excess of the Wilsons Dam option. Other dam locations and groundwater options were considered, but discounted in favour of the Wilsons Dam project.

One of the many recommendations from the Strategic Plan was to proceed with a feasibility study for the construction of Wilsons Dam, and this was undertaken in 1999. The feasibility study confirmed the conjunctive supply capability for the scheme, the design requirements, and cost estimate.

In 1999 a four-stage professional services contract was awarded to MWH for 'Engineering Services for the Design, Construction Contract Documentation, Contract Procurement, Construction Supervision and Commissioning for the Wilsons Dam'. Peer review contracts were also awarded to both Tonkin & Taylor and Meritec.

The first stage confirmed that Wilsons Dam could meet WDC's objective of providing a 1-in-50-year Return Period Drought supply capability at the end of a 50-year planning period. Differing dam heights and cost estimates were presented and a decision was made to go for a larger dam (should ground conditions permit), providing an additional 29% live storage for little additional cost.

To provide additional security, the pump station used to pump raw water to the Ruakaka Water Treatment Plant was located adjacent to the Tauroa River (on route to the treatment plant). This provided the ability to pump up to 750 000 m^3 of water from an adjacent catchment, providing a 'belt and braces' approach for little extra cost to the overall scheme.

Council approved the project and remaining land purchases were made. Construction of the dam started in October 2000 and reservoir impoundment commenced on 15 July 2002.

Dam foundation characterisation

The foundation soils at the dam site were investigated between 1987 and 2000 during a series of feasibility studies and the final design. In all 15 trial pits, 10 boreholes and 19 CPTs were undertaken, with laboratory and field testing for plasticity, consolidation properties, pore pressure dissipation, triaxial shear strength, vane shear strength and permeability.

Significant uncertainties associated with foundation permeability, consolidation characteristics and liquefaction risk were targeted in the design phase investigations.

The foundation model included a 3 m thick crust of 'floodplain alluvium' (silts), overlying 'upper gravels' 2m thick. 'Estuarine silts' 10 m thick and 'lower gravels' 5m thick then overlie basement greywacke at approximately 20m depth. The investigation recognised that the 'upper gravels' appeared to be laid down in channels and did not form a continuous sequence. Several layers of varying thickness and depth, within the 'estuarine silts', could be found at any particular location. Groundwater level was identified between 1 m and 3 m below the ground surface, with some tendency for artesian pressures from deeper aquifers.

During construction further geological information was obtained from: drillholes for extensometers and piezometers; seven CPTs used to confirm the cut-off wall depth; and eight CPTs used to confirm the depth of sediment along the abutment contacts. In addition, the rate of penetration for wick installation was monitored on a 10 m grid, yielding rudimentary information at relatively close spacing, particularly related to the depth of the 'lower gravels'.

In general terms, the design foundation model was confirmed and the complexity of the 'upper gravels' sequence defined in more detail. For example, the right hand side of the valley on the dam centreline has thick sequences of fine 'upper gravels', whereas the left side has minimal gravels and in one hole only silts were found. In another location the 'upper gravels' appeared only as a 300 mm sand layer, whereas 5 m away there was a 3 m thick gravel layer. The ‘estuarine silts’ were seldom found in the full 10 m expected layer thickness, with lenses of sands and or gravels appearing as breaks in the sequence.

Design considerations

The dam foundation of deep soft soils, with permeable lenses, has driven many aspects of the dam design. Special consideration needed to be given to construction stability, settlement, foundation seepage, and liquefaction under seismic loading.

Design features specifically incorporated to deal with these issues include:

- Wick drains, to accelerate drainage and consolidation of the soft foundation soils
- A cut-off wall formed from cement-bentonite slurry, minimising seepage through the ‘upper gravels’
- 30 m wide berms on the upstream and downstream shoulders, to consolidate and strengthen a wide area of the foundation
- A plastic core zone, to accommodate substantial settlements
- Staged construction, allowing dissipation of excess pore pressure between earthworks seasons
- Instrumentation along the crest and on two instrumented cross-sections, comprising 54 vibrating wire (VW) piezometers in the foundation, 16 VW piezometers in the fill and 15 combined inclinometer/extensometer tubes extending from bedrock up to the dam surface.

An indicative cross-section through the dam and its foundation is shown in Figure 2.

Design solutions and recorded performance

The following sections discuss how three key issues were addressed and makes comparisons of design predictions with monitored performance.

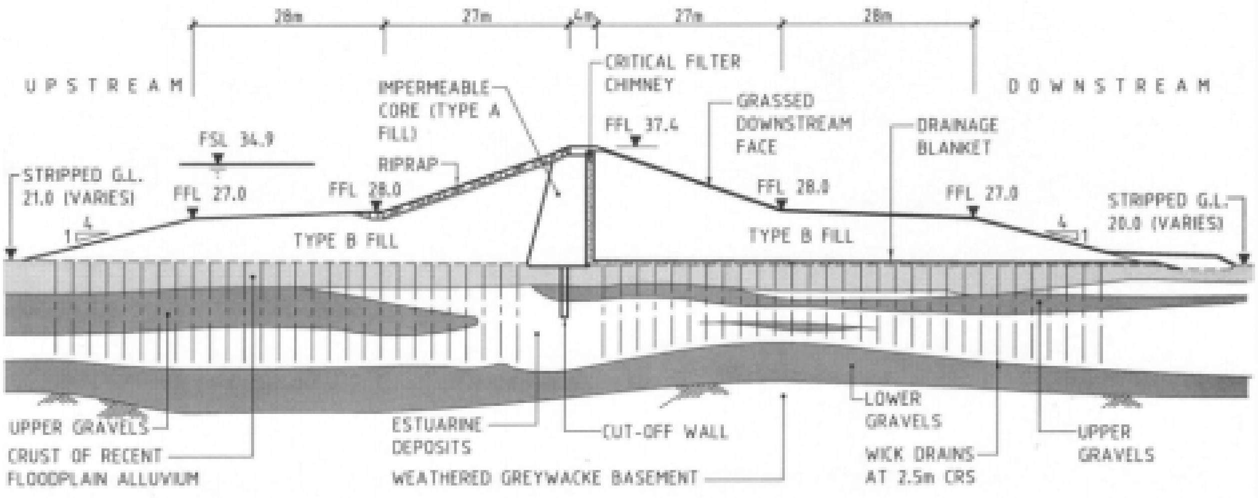

Figure 2. Cross-section of Wilsons Dam.

Construction stability

The 1995 failure of a Northland irrigation dam, during construction on a similar foundation (Freer 1997) illustrates the need for extreme caution in the design and construction of such dams. The key to embankment construction on soft ground is to avoid overstressing and yield of the foundation soils. At Wilsons Dam this was achieved by the adoption of staged construction; the provision of wick drains to speed consolidation; close monitoring of foundation piezometric pressures to control fill rates; and stability reviews at critical points based on measured pore pressures and foundation shear strengths.

Wick drains were used to speed consolidation and in particular to allow rapid drainage from the middle of thick layers of silts, thereby eliminating any laterally continuous weak zones. Wick design was based on

Bru's Chart 1981 (from Leroueil et al. 1990) and a square grid spacing of 2.5 m was adopted. One rig took four months to install 25 km of wick to an average 11 m depth (14 m max.). The lateral extent of installation was limited to where fill depth exceeded 2.5 m, wicks were also curtailed where the valley sides reduced the thickness of soft foundation soils. A section of the foundation 12.5 m wide, beneath the core, was left without wicks. The upstream wicks were collected into a piped drainage system, discharging to the diversion tunnel; flows of up to 4.5 litres per minute were observed during construction. The downstream wicks are terminated in the drainage blanket, which discharged freely to a temporary open channel throughout construction; flows were too small to be measured.

An undrained shear strength profile, based on the work of Leroueil et al. (1990), was developed for the design. Gains in shear strength resulting from consolidation were predicted using the increase in effective stress resulting from dam fill. During construction, strength gains in the foundation were measured using 'Geonor' shear vane tests. The predicted 'design' strength profiles are shown in Figure 3 and compared with test data and interpreted 'reasonable lower bound' strength profiles used for stability review. The comparison demonstrates that, while there was significant scatter in the test results, trends in actual strength gain were reasonably approximated by the design profiles.

Stability analyses were conducted using the predicted undrained strength parameters for the embankment and foundation. Slope/W software (Slope/W Version 4.2.1, Geo-Slope International) was used with the Bishop Method for circular failure surfaces or the Janbu method for non-circular failure surfaces. The design soil profile for stability modelling is shown in Table 2. This model excludes the 'upper gravel' layers because of their lack of continuity.

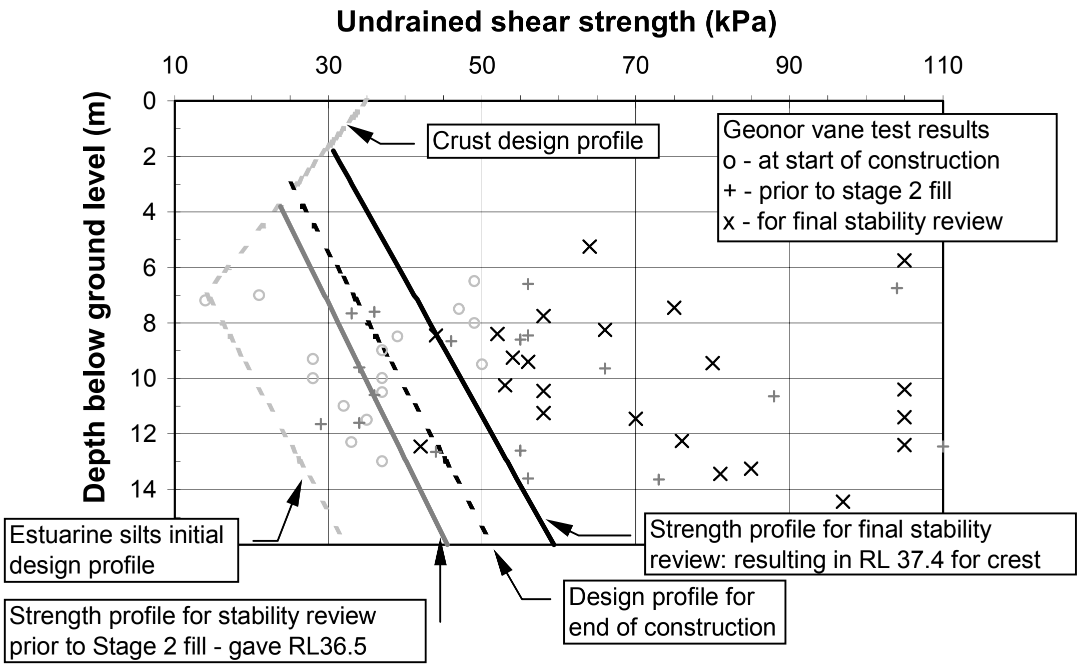

Figure 3. Shear strength profiles.

Table 2. Soil profile for stability modelling.

Soil layer	Depth below ground level	C(kPa)	*f*
Embankment fill		80	0
Recent floodplain alluvium and upper part of estuarine silts, which show a variable degree of overconsolidation.	0–7 m	Ref Fig. 3	0
Lower part of the estuarine silts, which are normally consolidated silts and clays.	7–15 m	Ref Fig. 3	0
Lower alluvial gravels	15–20 m	0	30°
Weathered bedrock – Waipapa group greywacke and argillite	> 20 m	20	40°

Stability reviews during construction were based on the same soil profile but with revised foundation strengths based on Geonor and piezometer results. The fill density was also adjusted to match field measurements. Factors of safety in excess of 1.3 were calculated for all temporary load cases at the design stage and these were confirmed during construction.

An additional goal of the stability reviews was to consider the possibility of raising the dam crest height and improving reservoir storage. A poor foundation response would mean the crest height was limited to the design target of RL36.3 whereas a good response would allow construction to proceed to a maximum of RL37.5 giving a 29% increase in live storage volume.

The very wet conditions during all of 2001 severely restricted the contractor's ability to place fill. Towards the end of the first construction season, it was recognised that there was insufficient fill in the downstream platform to achieve the required strength gain in the foundation soils. Accordingly, eight days in late May 2001 were spent pre-loading this area with uncontrolled fill, which was removed at the commencement of the next construction season. Berm level was not reached until early February 2002 and a stability review at that time, using the indicated strength profile in Figure 3, suggested a maximum final crest level of RL36.5. A final stability review was conducted as the fill neared that level and it was found that the interim strength gain allowed the final crest height to be raised to RL37.4. This strength gain is attributed to the increased time taken to place the second stage fill and hence the greater consolidation that occurred in the foundation.

When normally consolidated soils are loaded in undrained conditions, pore pressures increase proportionally with load. However, as loading continues the soil will eventually begin to yield, at which point the excess pore pressure rises at a rate greater than the applied load. Routine monitoring of fill rates was achieved by plotting pore pressure rise in the foundation piezometers vs estimated fill load, as shown by sample traces from two instruments in Figure 4. Both the piezometers plotted are located approximately 8 m upstream of the dam centreline and at the mid-layer depth in the 'estuarine silts'. The contract conditions included provisions to suspend filling if pore pressures were judged to be excessive. However, in the reality of the 2001 earthworks seasons, this option was not exercised and fill rates were effectively constrained by weather.

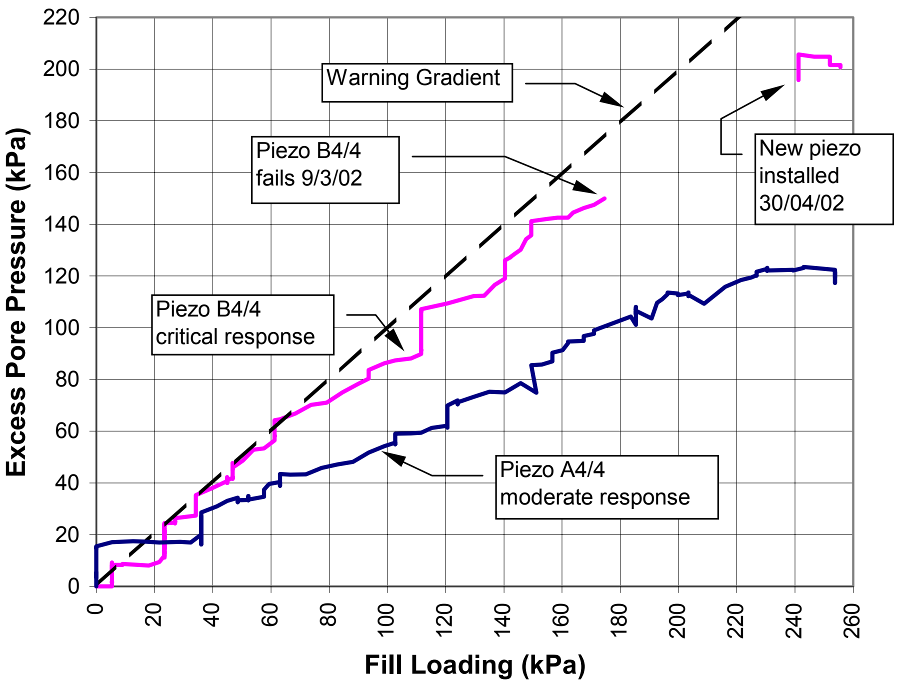

Figure 4. Foundation pore-pressure response monitoring.

Settlement

Foundation settlements under the dam considered components of elastic settlement, primary consolidation and secondary compression, predicted from conventional consolidation theory (Terzaghi & Peck 1967). Settlement predictions are summarised in Table 3.

Table 3. Calculated foundation settlements (mm).

Component	Lower bound	Upper bound	Best estimate
Elastic settlement	20	75	35
Primary consolidation	575	1400	1075
Secondary compression	25	275	60
Total	**620**	**1750**	**1170**

Primary consolidation settlement was estimated from laboratory oedometer testing. The observed range of results for the coefficient of compressibility (m_v) was 0.15 to 0.37 m^2/MN. The best estimate is based on m_v = 0.27 m^2/MN.

Due to a lack of site specific data, secondary compression was estimated from the natural moisture content of the foundation soils. Mesri's Chart (Mesri & Choi 1975) indicated that the secondary compression ratio would probably lie between 0.07% and 0.9% with the most likely value being 0.21%.

Settlement during construction depends on the response of the foundation soils under load and the efficiency of the wick drainage. Expectations were that the degree of consolidation would be at least 50% and possibly 75% by the end of construction. Settlement predictions during construction were thus in a range of 570–845 mm. By the end of construction, settlement had reached 900 mm as measured by magnetic extensometer probe under the crest. Assessments of actual consolidation by the end of construction were higher than expected with a degree of consolidation at 80–90%, as a result of the extended duration for fill placement.

Results of settlement measurement at the crest and other key locations are shown in Figure 5 and indicate a high degree of conformity with the original predictions.

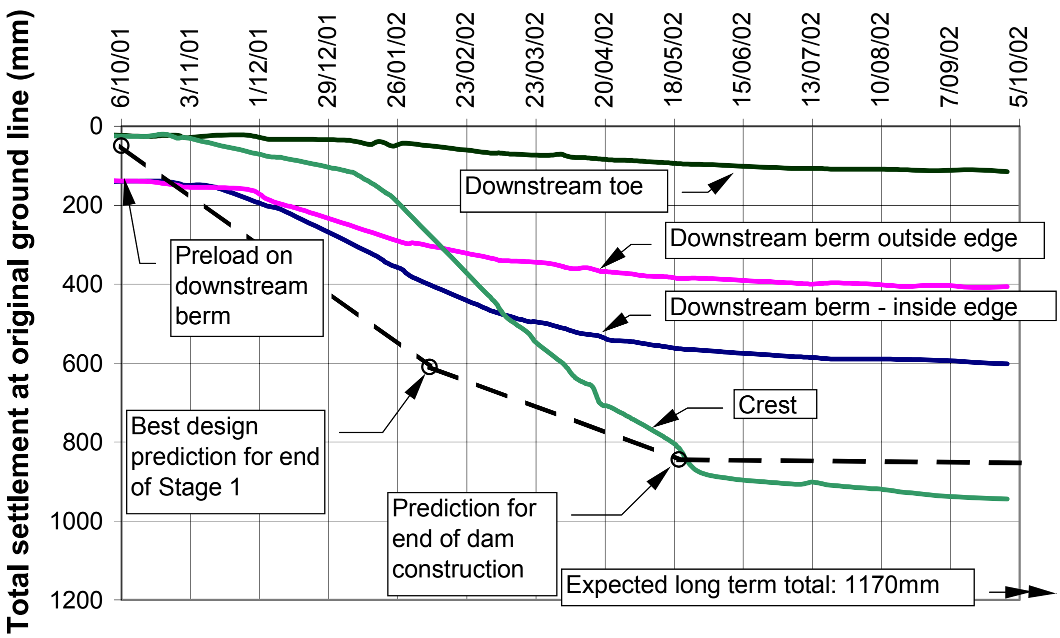

Figure 5. Measured foundation settlements, mid-valley.

The extensometer tubing has experienced a high failure rate under the large settlements that have occurred. The first tube to block was investigated by camera and could be seen to have buckled where the settlement gradient was high in the 'estuarine silts'. Other tubes appear to have buckled at the location of thin walled joints near the foundation/fill interface and may be the result of in-built misalignments at these points. Improved durability may be achievable by the use of heavy duty telescoping joints through the fill as well as in the foundation.

Surface monuments have been installed on the dam to allow future monitoring of settlements and horizontal displacements by conventional survey. The extensometers have served their intended purpose of construction monitoring, however, there are still nine tubes able to be read to full depth and this is sufficient to enable differentiation between foundation and fill settlements during ongoing monitoring.

Seepage

Reservoir losses through the dam, its foundation and the abutments were modelled using Seep/W software (Seep/W Version 4.20, Geo-Slope International). The seepage model differs from the stability model to include appropriately conservative assessments of permeable layer thickness, allowing for the interlayering of permeable lenses within the 'estuarine silts'. The model also extends hundreds of metres upstream and downstream of the dam to account for losses through the 'lower gravels'. The seepage parameters incorporated in the model are summarised in Table 4.

Table 4. Permeability model parameters.

Layer	Top RL (m)	Base RL (m)	Permeability range (m/s)	Most likely permeability (used for both horizontal and vertical, m/s)
Embankment fill	38	20	$10^{-9} - 10^{-11}$	10^{-10}
Recent alluvium	20	18	$10^{-8} - 10^{-10}$	10^{-9}
Upper gravels	18	14	$10^{-4} - 10^{-6}$	10^{-5}
Estuarine silts	14	6	$10^{-8} - 10^{-10}$	10^{-9}
Wick drained silts	20	7		10^{-6}
Lower gravels	6	0	$10^{-4} - 10^{-6}$	10^{-5}
Bedrock	0	–40	$10^{-6} - 10^{-8}$	10^{-7}

Specific steps taken to minimise foundation seepage included construction of a cement-bentonite slurry cut-off wall through the 'upper gravels' and minimising the interconnection of wick drains with the 'lower gravel' layers.

The cut-off wall was specified as a very weak cement-bentonite slurry with requirements on plasticity, permeability and a 28 day unconfined compressive strength of 150 kPa. The mix design and installation techniques were developed by Australian consultants, M.P.A.Williams and Associates. Roadstone planned and managed the construction with assistance from Richardson Drilling for slurry mixing. Installation was achieved with local resources. MWH set trench target depths based on a series of CPT's along the dam centreline and construction was monitored closely to ensure that the wall extended into the 'estuarine silts'. Some problems of trench collapse occurred, where gravel deposits were thicker and close to the surface. However, initial concerns that these collapses would bridge the trench have been allayed by comparison of piezometer records (see Figure 6) between Line B where trench collapse occurred and Line A where the trench stayed intact.

The final installed depth for wick drains was determined by observation of penetration resistance for a pilot series of wicks on a 10m grid. Increased resistance in the expected depth range, established from investigation drilling, was used to identify 'lower gravels'. A 'lower gravels' surface was then interpolated from this grid and intermediate wicks were installed with a nominal metre clearance from this surface. By this method, an interconnection rate of 1-in-15 should be achieved, however, a minimum 1-in-10 interconnection rate was used for design, allowing for some additional accidental connection.

Best design estimates of seepage, with the cut-off wall installed, ranged from 0.4 to 1.2 l/s depending on the degree of interconnection between wicks and the 'lower gravels'. Upper bound seepage is an order of magnitude greater at 4 to 12 l/s. From conjunctive use water modelling it can be demonstrated that seepage losses

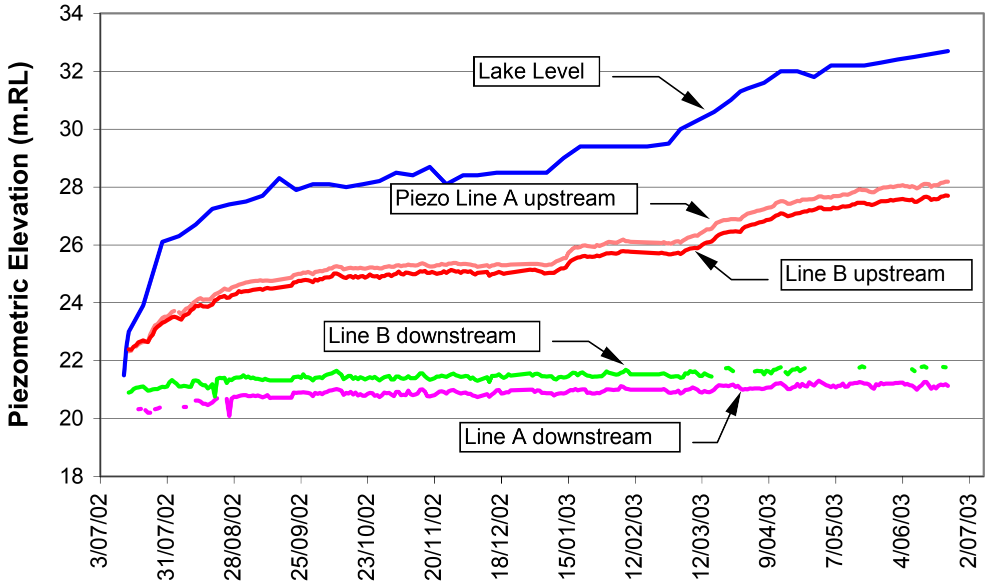

Figure 6. Head loss across slurry cut-off wall.

up to 12 l/s will not affect reservoir viability. Figure 7 shows measured seepage in relation to reservoir level during commissioning, a target maximum of 1 l/s has been set and expectations are that this will not be exceeded.

Seepage is collected in a drainage system leading to a series of manholes across the downstream toe of the dam. The downstream drainage blanket is separated into 6 drainage zones in order to differentiate between flow sources. Another drain collects seepage along the toe of the right abutment. It is assumed that the wicks will provide a preferential path for deep seepage to enter the collection system via the downstream drainage blanket. However, it is possible that some seepage will exit the site in either the 'upper' or 'lower gravels' without being measured.

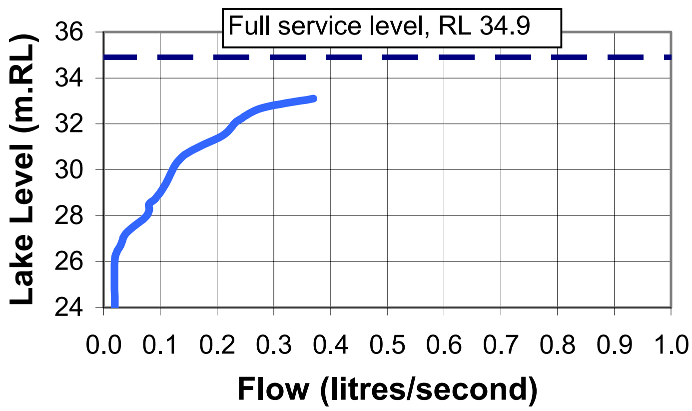

Figure 7. Seepage with change in lake level.

Summary and Conclusions

Conclusions from the design and construction of Wilsons Dam are:

1. The soft foundation conditions presented significant challenges for dam design and construction. Monitoring to date has confirmed the success of the design.
2. An undrained shear strength model, based on the work of Leroueil et al, adequately predicted the

strength gain in the foundation soils arising from construction of the embankment.

3. Traditional consolidation theory adequately predicted the range of settlements experienced to date.
4. Continual monitoring and updating of stability predictions recalibrated with actual field data, allowed the dam height and reservoir volume to be maximised during construction.
5. Seepage losses have been minimised through careful installation of wick drains and by provision of a cement-bentonite slurry cut-off wall through the permeable ‘upper gravels’ and into ‘estuarine silts’.
6. Large settlements have resulted in the loss of a significant number of instruments. However, the instruments have served their main function of ensuring safe construction and a sufficient number of instruments remain for the ongoing monitoring of performance.
7. Ongoing monitoring will be in accordance with a surveillance manual and emergency action plan prepared in accordance with NZSOLD guidelines.

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The upgrade of Watercare's Cosseys Dam: Project implementation and consenting

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Two events, in 1987 and 1994, where embankment pressures rose above historic maximums raised concerns about the stability and seepage patterns within the earth embankment of Cosseys Dam near Auckland. Risk reduction measures were instigated after the 1994 event. Investigation indicated that the underdrain was not filter compatible with the overlying core, transition and shoulder materials plus there was strong evidence of contamination of the underdrain, which was apparently blocked beneath the transition zone. The meandering shape of the underdrain and the three dimensional shape of the abutments provided challenging design issues. The water catchment parkland and the steep topography at the dam site contributed to the design and environmental issues that were resolved. A two-phase remedial works programme is underway to excavate the underdrain and replace it with a compatible filter and drainage system. The drawing down of the lake to 3% full capacity opened up environmental issues that did not have management precedent elsewhere.

Keywords: *internal erosion, resource consents, consultation, risk mitigation, management plans.*

Introduction

Cosseys Dam, Auckland's third largest dam, is located 17 km east of Papakura (Figure 1). The dam is currently undergoing the second and final stage of major upgrade works, scheduled for completion in January 2004. The upgrade works will bring the earth dam up to modern design standards and improve the seismic performance of the valve tower.

The dam, commissioned in 1955, is 40 m high and impounds 14 million m^3 of water at full storage level, 14% of the storage capacity for reticulated supply to municipal Auckland. The original design shows a zoned earthfill embankment with shoulder, transition and core comprised of unweathered to completely weathered greywacke rock won from borrow areas within the reservoir. Spoil from the excavation of the diversion tunnel was laid in the old creek bed to form an underdrain that was filtered from the core with a 50 mm layer of clean sand.

The land associated with Cosseys Dam and reservoir is designated in favour of Watercare Services Ltd (Watercare) for Water Supply Purposes, within the Franklin and Manukau District Plans. The land is part of the Hunua Regional Park, and is classified in the Local Government Act as Water Catchment Parkland.

Watercare holds resource consents for a duration of 35 years to dam, to take, and to discharge water at Cosseys Dam. These

Figure 1. Cosseys Dam before works began.

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consents have a number of conditions pertaining to the ongoing operations of water supply, and do not envisage major upgrade works. The discharges captured under the operating consent are for residual flow and spill flow purposes, and occasional flushing flows to maintain the health of the downstream aquatic ecosystem. There is a requirement for involvement of an independent audit group should any significant alteration be made to the existing structure, as was the case at this time. The role of the Audit group is to provide expert technical advice in the areas of dam safety, water resources, and ecology, plus confidence to the Auckland Regional Council that Watercare is operating in a responsible manner.

The dam safety studies in the 1980s had confirmed that the valve tower did not meet current seismic guidelines. In 1987 changes in the internal seepage patterns were detected, and following refilling of the dam after the 1994 water shortage, further changes were detected. This prompted Watercare to initiate a series of safety reviews and investigations by dam experts. Initial risk reduction measures included installing temporary well pumps to relieve pressures upstream of an apparent blockage in the underdrain, and lowering the operating level of the reservoir by 5 m to 60% full capacity.

By August 2000, investigations revealed that the underdrain system was not filter-compatible with the embankment materials. Based on drilling results, the sand filter layer above the underdrain was thinner than indicated on the design drawings and was not present in some locations. Strong evidence of fines contamination of the underdrain, in conjunction with the growing body of seepage monitoring data, supported the conclusion that the underdrain had become blocked near a natural constriction of the abutments. Additional risk reduction measures of further lowering the reservoir level together with increased surveillance monitoring while a range of upgrade options were assessed.

In February 2001, the Watercare Board approved a recommendation to proceed as quickly as possible with implementation measures to secure the safety of Cosseys Dam. The drought management plan for the supply of treated water to urban Auckland was updated in liaison with the Local Network Operators and externally reviewed by overseas expertise to manage the risk of a water shortage during the rehabilitation.

A downstream shoulder reconstruction was selected as the best solution considering a broad range of objectives and constraints. These included the impact of lowering the reservoir, social and environmental effects from the construction activities, the safety of the dam during construction and the risk of residual defects remaining within the dam after completion of the remedial works.

Civil works

URS NZ Ltd were retained to carry out the detailed design in February 2001. The key features of the upgrade design (Figure 2) were:

- Lowering the reservoir to a safe level;
- Removing the incompatible underdrain materials from below the dam with a large downstream excavation;

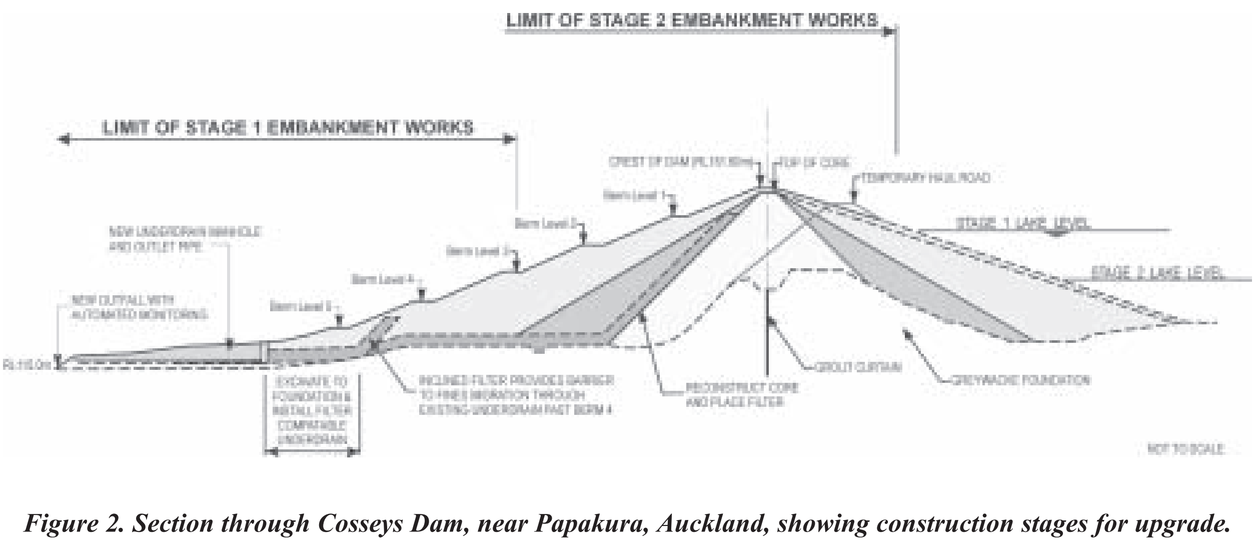

Figure 2. Section through Cosseys Dam, near Papakura, Auckland, showing construction stages for upgrade.

- Replacing the underdrain beneath the core of the dam with compacted core material where accessible and grout to seal the remaining underdrain that was not accessible;
- Reconstructing the underdrain with filtered drain material below the shoulder of the dam, downstream of the reconstructed core;
- Placing an inclined chimney drain/filter across the reconstructed core and abutments to provide filter protection and control ongoing seepage flows into the underdrain;
- Reconstructing the downstream shoulder of the dam and crest roadway; and
- Strengthening the valve tower to resist the appropriate seismic loads by anchoring into the rock abutments.

The project was divided into two distinct construction stages (Figure 2), each to be undertaken in consecutive construction seasons. A single construction contract was awarded for both stages. The contract was awarded in October 2001 and the work commenced on the replacement of the old single carriageway bridge over the Wairoa River on the only viable access road to the site in November. A replacement bridge and upgrading of the access through the private right of way and internal roads was undertaken at the start of Stage 1 to provide access for the plant and vehicles required for construction.

The Stage 1 works comprised the construction of a new underdrain outlet control structure and the underdrain interceptor and collection system. This was a key element of the overall rehabilitation of the dam and provided a significant reduction to the risk of an uncontrolled piping failure. A portion of the toe of the dam was removed and the underdrain outlet installed, to contain any internal erosion that may occur during the following winter.

Stage 2 encompassed the removal of the downstream shoulder of the dam and part of the core, removal of the old underdrain, abutment and rock treatment and construction of a new drainage system protected by a compatible layer of filter material. Reconstruction of the dam embankment (Figure 3) included the construction of an inclined chimney filter between the downstream face of the core and the shoulder, extension of the under drainage, a new access road across the dam crest and reinstatement of the disturbed areas. The strengthening of the valve tower was included in the Stage 2 works.

The utilisation of the experienced Peer reviewers and Project Manager throughout the project has been conducive to the rapport with the stakeholders and the progress that has been achieved to date on this site, despite Auckland's weather the unforeseen additional foundation treatment works that were identified as the excavation progressed.

Environmental measures and consents

Sinclair Knight Merz were responsible for assessing the environmental effects of the project, and developing measures that would remedy, alleviate or mitigate the hazards. A close liaison was maintained between the civil and environmental consultants. DamWatch was appointed as an independent peer reviewer to review

Figure 3. Cosseys Dam with the face removed to allow the installation of the underdrain and filter.

the entire project, to ensure that any fatal flaws would be identified in a timely manner, and also to ensure that the safety of the dam met international practice.

As the design progressed, additional investigations were undertaken. These resulted in a major shift to the development of the remedial works enabling a significant decrease in the environmental effects of the project. The following environmental issues were taken into consideration during the preparation of the consents to enable the project to proceed.

Timing of consents

The consent process was critical to the timing of the project, and Watercare was keen to see the Auckland Regional Council set a Hearing date, thereby providing a programme for forward planning purposes. Obviously the issue of appeals could still have a bearing on this, as it might on how critical the works were becoming.

In terms of public notification, without the approval of critical affected parties, one being the local landowner, there would be no chance of meeting the *de minimis* or negligible test set out in s94 of the Resource Management Act, because it would be difficult to argue that the potential adverse effects would be minor. Their approval was critical to achieving a start to construction within the shortest time period.

Identification of consents required

Watercare initially applied for consent to draw down the water level of the Reservoir to a minimum capacity of 25% by discharging water into Cosseys Creek. This consent was issued in July 2001. However, following further investigation into the stability of the dam, it was deemed necessary to drop the water level to approximately 3% capacity and split the project into 2 stages.

Early in the process the ability to utilise emergency powers to undertake the upgrade works was considered, but dropped, for the works were not entirely 'unforeseen', and nor could it be demonstrated at that time that the remedial works couldn't wait for due consent processes.

The project team quickly identified which works must be commenced at the start of the contract (i.e. road works) and considered consent requirements for those, e.g. are they all within the designated area? This ensured vital elements were prioritised to prevent delays down the track and allow consent conditions to be known before critically necessary. It was established that consents were required from both the Auckland Regional Council and Franklin District Council.

Aside from numerous consents to upgrade the access road, strengthen the valve tower, remove vegetation etc, the consent packages were split into two distinct groups:

- Upgrade works – this comprised the physical earthworks, stockpiling and construction; and
- Drawdown – this covered the ongoing and intermittent releases of water downstream.

The Assessment of Environmental Effects (AEE) was reviewed by legal counsel before lodgement, and a stand-alone executive summary was produced for interested parties.

Discharges prior to receiving consents

Heavy rainfall events in April and May of 2001 required Watercare (for dam safety assurance) to release water before the discharge consent processing had been completed. This was to prevent the water level rising beyond acceptable criteria set by the designers. The lake was then being kept at the 60% full level. The Regional Council was notified at the time of the discharge. A retrospective consent was sought and subsequently granted.

Consultation

Fronting up to the regulatory authorities was essential. Numerous issues were arising and the importance of taking the regulatory authorities along the journey with the project team was identified. A joint programme of education was required to understand all the technical and operating constraints involved in managing a dam of this nature.

During pre-application discussions with the Auckland Regional Council a list of interested parties with whom to consult was scoped. Wide and thorough consultation was considered necessary to ease the consent process and avoid any repercussions at a later date.

Public relations was also considered to be very important at this stage. The quiet rural community perceived that we were to cause significant disruption, and questions of ‘Why now?’ and ‘How critical is it?’ were expected to be raised by locals and politicians alike. The message of dam safety and benefits to the region needed to be stressed without creating hysteria.

A key affected party was the local landowner, whose property provides the right of way for accessing the dam, and on whose property the stream below the dam meets the Wairoa River. Nearby is a Presbyterian Camp, known for offering peace and tranquillity, amongst other aspects. Rural landowners further down the Wairoa Valley also had a high interest, and not only from a dam safety standpoint. For years they have hotly debated the cause of sedimentation of the riverbed and coastal marine area, which has been linked to flooding in the area. Up to six groups affiliated with iwi in the region raised their hands in interest early in the process. Regional Parks, as landowner of the surrounding parkland, was also recognised as needing individual and attentive consultation.

These interested parties were kept informed of the project details through letters, individual meetings and informal telephone follow-ups, site visits, public meetings, Cosseys newsletters, web site information, and a 0800 telephone line.

Submissions

The applications were publicly notified, and against all early predictions, only a handful of submissions were received, with support from Manukau City Council, the owner of the private right of way, the Presbyterian Camp, Federated Farmers, DOC, Iwi, and Fish and Game. One outstanding objection was revoked following further consultation and the end result was that a Hearing was not required.

Wet weather contingency

While it was intended that the Stage 2 works would be complete by the end of the 2002/03 construction season, it was conceivable that poor weather or unforeseen matters would require further works the following construction season. On that basis, Watercare built in contingency and applied for both construction and discharge consents up to 30 June 2004.

Environmental effects and mitigation measures

Cosseys access road and bridge

Before works could begin, it was necessary to replace the bridge that once crossed the Wairoa River on the private access road to the dam. The old bridge could not support the increased size and volume of heavy traffic involved in the project. Consents to replace the bridge were immediately sought from the Auckland Regional Council and the Franklin District Council. In the interim it was an all-day exercise to haul site investigation machinery in to the site via alternative forestry roads that were of poor construction and unreliable in wet weather.

The bridge was upgraded to take the expected heavy traffic, and also provide a walkway for the safety of future pedestrians.

No public access has been permitted during the construction works. During the period of high truck movements as filter material was transported to the site, the access road became one-way and was split into four radio-controlled zones with strict procedures on movement.

Truck movements on public roads

Trucks were needed to import the filter and drainage material required on site from a quarry 15 km away. Up to 180 truck movements per day were estimated to travel through the rural community during critical periods. Children normally walked home from school along these relatively narrow local roads, and the health and safety of other road users was identified as a potential issue. The introduction of fully loaded trucks along narrow roads and tight corners could be a safety hazard in the local community.

Watercare offered to operate the truck journeys on a one-way system, turning a selection of local roads into a one-way route, so residents would always know from which direction these vehicles would be travelling. The District Council felt it preferable to minimise the range of roads utilised in this operation and declined this option. They did accept Watercare's suggestion that no trailer units would be employed. A before and after study of local road conditions was undertaken with the agreement that Watercare would leave the roads in a state no worse than their original condition.

Stockpiling of shoulder and core materials

With the reservoir initially proposed to be drawn down no lower than 25% capacity, there was nowhere on site to stockpile material removed from the face of the dam. Preliminary design drawings and early consultation was undertaken on the basis that 200 000 m^3 of earth from the dam would be stockpiled on pastoral land near the entrance to the park. This land belongs to a local farmer who regularly hosts equestrian events on the subject site utilising both the flat land and the natural water feature that flows through his property below the dam.

The stockpile location was subject to periodic flooding and required extensive protection to prevent erosion taking place. Concerns were being raised from a range of groups over protection of the stockpile, noise of trucks and machinery working the site, and visual imposition as it was estimated to require an area of around 20 ha.

The landowner objected to the loss of pasture, amenity values, noise and dust disruption from bridge construction, truck movements during construction, and machinery working on the stockpile area. This proposed stockpile site was still approximately 3 km from the dam face, and while not ideal, provided the best of only three options at the time.

When later investigations showed it would be necessary to draw the reservoir down to just 3% capacity, the opportunity arose to stockpile material in part of the empty lakebed formed by Quinns Stream. Originally it was not considered this option would 'fly', but it soon became a key measure in mitigating and avoiding the issues and concerns raised by stakeholders in relation to stockpiling on nearby farmland. It also meant the stockpile, while having its own sediment control measures, would be located within the larger sediment retention pond, being the reservoir itself. The impact of this change had a major effect from a consenting perspective. It meant significantly less noise, dust, and vehicle movements to the local community. It removed the public visual impact altogether, the bulk of the works could now be undertaken well away from the nearest neighbours, and the proximity of the stockpile to the dam face now meant the programme of construction could be shortened. This amendment consequently removed the objection of at least two critical parties and had a large bearing on the consent process.

The Quinns Stream stockpile site was covered in deposits of silt laid over the last fifty years that had to be treated to prevent contamination of the stockpile. Other works included the laying of a diversion pipe beneath the stockpile, cutting haul roads and building a working platform in a steep sided valley, and preventing the erosion of the stockpile into the remnants of the lake. Development of the stockpile close to the dam embankment ahead of the option over farmland at the base of the Cosseys Access Road has provided significant savings to the project, reduced the risk associated with hauling material along the narrow access road, and significantly altered the impact on the local community.

Recreational use of Regional Park land

A survey during the previous year of the Hunua Regional Park by the Regional Council indicated some 55 000 visitors go to the Wairoa and Cosseys Creek localities every year. Releasing water into Cosseys Creek downstream of the dam was seen to be potentially hazardous for visitors who might be in or near the stream at the time releases are made. Therefore closure of the Cosseys loop track crossing the creek was necessary to avoid any adverse effect on track users. Regional Parks considered the opportunity cost to these potential visitors was significant. It was rather unfortunate that the *NZ Herald* featured an article 'Discovering Auckland' in Nov 2001 identifying the track past the dam as the best loop walk in the region – without the knowledge it had been temporarily closed.

It was agreed that Watercare and Regional Parks would share the costs of constructing an alternative track at an approximate total cost of $40,000. High visibility signs providing warning of sudden increases in water levels in the creek plus maps indicating track closure were erected at several locations, bearing both the Watercare and Regional Parks logo. These are checked at regular intervals.

Lake level management plan

A management plan was developed to link the control of the lake levels with rainfall predictions and dam safety assurance. While this was not a tool envisaged at the outset, it became apparent that it would be necessary to manage the discharge operations. Different stages of construction held different risks, and coupled with nature's elements, the alarm triggers for operating the two discharge valves needed to be flexible and simple to amend as the project progressed. The benefits of this meant the consent could be issued with the outstanding specifics addressed in the management plan with input from the design consultant. The management plan had to be submitted and approved by Regional Council before the consent was exercised. Amendments to the Management plan could be made by the Regional Council's Officers instead of going through another consent review process.

A predictive hydrological forecasting technique was developed as a management tool to assist in decision making regarding discharges. The model took into account the catchment response, the rainfall, lakebed geometry, and the current rate of discharge form the dam. This meant if heavy rainfall was predicted and other information suggested that dam safety was at risk, a pre-emptive discharge could be made

Water quality

With an amended dam design requiring the lake level to reduce to around 3% full, some 108 hectares of exposed bank within the reservoir would become evident. The Regional Council held concern regarding the potential instability of this exposed sediment and the opportunity for it to be mobilised and move downstream. It was very difficult to predict the levels of turbidity and the suspended sediment loads from erosion of the exposed bands and the reservoir bed with inflow stream water cutting through soft sediment and picking up material. It would be greatly influenced by rainfall events.

With the lake at 3% for the Stage 2 works, access to the streams feeding the reservoir was not readily achievable and no additional measures were undertaken to reduce sediment entering the reservoir.

Oversowing of the lakebed by helicopter was undertaken, with patchy results due to seasonal conditions, heavy rainfall and steep-sided slopes. This was not a consent condition, but was undertaken to show Watercare's willingness to mitigate where feasible.

A natural sediment pond at the southern end of the reservoir was identified and a purpose-made silt screen was installed across the outlet of this zone, some 60 m wide. While the Regional Council were sceptical as to the likely effectiveness of the silt curtain, Watercare made the decision to proceed. The results were much better than expected and the silt curtain has played a significant role in controlling the turbidity level of the water being discharged to Cosseys Stream.

Management of discharges

Initially, as much water as was possible was drawn off from the lake for water supply, but this was limited to 0.7 m^3/s by the capacity of the raw water delivery system. Increased discharges to the creek were used to prevent any sudden increase in lake level during rainfall events, and to complete the lowering of the lake in time for construction. These could be up to 13 m^3/s flows, depending on the rate of rise of the reservoir. There was concern that juvenile fish that breed during spring would not survive a passage through the discharge valves. A target of 1 September was set for completing the lake drawdown. The major discharges were completed before the juveniles began to develop in the lake.

In order to minimise potential downstream effects, it was proposed to discharge at as low a flow rate as possible, over the longest period of time available. The estimated rate was around 1–2 m^3/s. The duration of this discharge was dependent on how early the consents were issued, and rainfall received. It was estimated to be around ten weeks. The Regional Council agreed this method would also provide the maximum time for the exposed reservoir banks to revegetate. It was proposed to take water for discharge to the stream via the valves in the standpipe tower, thereby taking 'cleaner' water from the higher levels of the water column. At the same time, it was understood by the regulatory authority that flow rates could not be safely capped at 2 m^3/sec.

On several occasions obstructions ranging from logs, bars from the intake screen, and rocks all caused operational difficulties. On more than a couple of instances debris was lodged in the system during discharges, making either manual or automated shutoff unresponsive. In effect these were uncontrolled discharges. At this time the importance of having the Regional Council conversant with our operating system and aware of these risks was essential. It left no doubt that the many hours spent alongside the regulators in advance of these occurrences had been worthwhile.

The receiving environment

The Cosseys reservoir discharges into Cosseys Creek. Cosseys Creek is approximately 2 km in length and is accustomed to low flow conditions. Its water comes from the small catchment area below the dam, from spilling, and occasional flushing and maintenance flows released from the scour intake on at the base of the dam's valve tower. The downstream environment is surrounded by native vegetation until it reaches the alluvial floodplain of the Wairoa River below. At this point the creek is bordered by pastoral farmland. Generally baseflow widths vary from around 1–2 m. The impact of drawing down the dam on the receiving waterways was identified as a key issue for environmental groups. Close to the dam, the streambed is steep and comprises boulders and bedrock that would resist erosion. Although NIWA had predicted a prolonged flow of 2 m^3/sec would cause little disturbance, flows of up to 13 m^3/sec were being applied for to meet discharges required for dam safety assurance. There was some question whether this might have an affect on the lower reaches of the stream before meeting the Wairoa River. The discharges were small in comparison with the Possible Maximum Flood outflow of 366 m^3/sec.

To understand and assess the impact on both Cosseys Creek and the Wairoa River, it was agreed that water sampling be undertaken whenever water was discharged from the dam. This approach meant the impact on the receiving water downstream could be established. If background water in the receiving environment was sediment loaded due to heavy rainfall further up the Wairoa catchment, it figured that the discharge from Cosseys would have lesser effect than in dry conditions. Where practicable, discharges would deliberately coincide with rainfall events, except where required for the gradual draw down of the lake.

The outcome was consent conditions that required Watercare to notify the Council whenever the discharge turbidity exceeded 50 Nephelometric Turbidity Units (NTU) upon discharge from the dam. There was also provision for consultation into how this could be mitigated, such as deferring the discharge to allow turbidity levels to decrease without compromising dam safety. At times turbidity reached in excess of 200 NTU, although these were for relatively short intervals coinciding with high rainfall under storm events. At these times, levels in the Wairoa River were also significantly elevated of their own accord.

Exotic fish

Exotic fish such as rudd, perch, and carp were illegally released into the water reservoir over 10 years ago, and there was a risk that during drawdown these would be released into the downstream environment. However, Watercare already had consent to discharge down Cosseys Creek four times per year under normal operating conditions, during which times exotic fish may escape.

The degree to which this would become an issue was possibly underestimated. To further understand the threat of exotic fish release, the Regional Council required Watercare to undertake a detailed investigation on the matter, and NIWA was consequently engaged to carry out this work. Evidence was gathered that the downstream environment already contained these species. It was still an issue for various parties, including local Iwi and the Council. The Regional Council felt it would be a good opportunity to remove or eradicate exotic fish from the reservoir altogether. Watercare held a firm position that no chemicals should be used in the water supply lake. With ponding likely to occur in the lake, tributaries still harbouring juveniles, and inaccessible areas in the lakebed, NIWA concluded that it would be extremely difficult if not impossible to achieve eradication. No example of the success of such a project could be found in the literature.

To alleviate concerns over further release of exotic fish, Watercare installed a fish net at the base of the stilling basin to capture and record the species that escaped from the reservoir. Approximately 2500 exotics were collected, all of which were dead on arrival.

Iwi

Iwi interest was greater in the Stage I works, when details of the project were less understood and up to six individual groups were consulted with at that time. Concerns regarding the historical and cultural aspects of the damming of the waterway and the impact on the downstream environment were raised. Any archaeological finds would be unlikely, as the area had previously been significantly modified. They voiced an apparent reduction in the eel fishery since the construction of the dam. It was generally felt that the steep-sided reservoir meant there was no longer an optimum habitat for eels. A further issue of concern was the release of sediment-loaded water into the downstream environment that could potentially affect the food-gathering areas near the Clevedon coast and accentuate the flood events that occasionally occur in the lower reaches of the Wairoa River. Ngai Tai showed a keen interest in being involved to monitor the impact on the downstream environment utilising cultural methods of setting indicators.

As a means to recognise and provide for the relationship Iwi have with their ancestral waters, Watercare provided for a representative of Ngai Tai to undertake regular Tangata Whenua characterisation studies on the effects of the project on the receiving waters pursuant to cultural values.

Local Iwi were offered access to trap any eel that may be living within the lakebed. Only around 40 eel were found to have exited the lake through the downstream discharge valve to date.

Presbyterian Camp

An additional neighbouring party is a camp on land owned and run by the Presbyterian Church. We were told camp visitors walk the local track, utilise the river, and occasionally have choir groups rehearsing and recording in these tranquil surroundings. The originally proposed stockpile site that would have been constantly in use some 400 m away in clear view of the camp was not considered a compatible activity. However, with the stockpile in the reservoir this was no longer an issue.

Complaints register

A comprehensive system for receiving and responding to complaints was set up. This included distributing frequently asked questions (FAQ) sheets and fax complaint forms to Regional and District Council staff, Watercare’s Communications Department, and Watercare’s 24-hour control room.

Reporting of environmental effects

Watercare was required to submit a comprehensive monthly report to the Regional Council. This includes all monitoring and recording, details of any mitigation measures undertaken or proposed, a comment on the effectiveness of the fish net, flow hydrographs for flows in the Wairoa river whenever water had been released from the dam, notification to a range of downstream landowners and affected parties whenever discharges were made, notification of turbidity levels and records, a macro-invertebrate assessment including an interpretation of any changes, and a record of any complaints received. During the consent process there were a number of uncertainties in the potential effects on water quality downstream. The stringent monitoring programme and reporting conditions gave the Regional Council confidence that any effects would be detected early, reported, and where possible, mitigated.

The monitoring requirements can be summarised as follows:

- Daily measurement of rainfall and lake level;
- Daily real-time measurements of reservoir discharge volume and rate;
- Daily measurements of discharge water quality at the outfall and at selected points downstream;
- Monthly visual and photographic characterisation studies undertaken in the creek downstream of the dam to monitor and record changes over time;
- Monthly studies of the reservoir banks to assess stability and erosion and to document revegetation. If this proved to be a problem, the situation would be reviewed in consultation with the Regional Council; and
- Studies of the impact on downstream ecology and invertebrates are also required at various stages of the project.

Conclusions

The upgrade works at Cosseys Dam are being successfully carried out after an extensive consultation and careful implementation of the resource consent conditions.

It has been extremely important to establish good working relationships and be ‘up-front’ with the regulating authorities. Most of the difficulties experienced in this regard all related to a lack of understanding.

Maintaining a strong and proactive communication network with ALL interested parties, including the Territorial Authorities and local community boards has been critical to the success of the consenting process.

The team formed from Watercare’s Environmental Planner and the independent environmental consultants were able to maintain a sound working relationship with the dam designers and provided robustness to the environmental process. The consequences of any proposal were thoroughly reviewed and options investigated without significant delays to the design time.

Grouting high-pressure seepage at Arapuni Dam, New Zealand

Peter D. Amos[1], Tom G. Newson[2], Murray D. Gillon[3], and John N. Stewart[4]

In September 2000, pressures being monitored in a geological fracture beneath Arapuni Dam were found to be rising significantly, indicating that a deteriorating condition was developing in the foundation. Two boreholes drilled in 1995 had intersected high water pressures within the fracture in an area close to the downstream face of the dam, posing a risk of major leakage developing from where the fracture day-lighted downstream of the dam. Investigations since September 2000 confirmed the extent of high pressure in the dam foundation and the nature of the deterioration, providing a baseline to assess the feasibility and performance of treatment options. The foundation fracture bearing the high water pressure was successfully grouted in December 2001 without lowering the reservoir or damaging the dam's porous concrete underdrain network. The key to the success of the operation involved three important elements in the approach to grouting: controlling foundation pressures within safe limits during grouting; designing a grout mix suitable for application in high-flowing seepage water; and implementing measures to prevent grout entering and blocking the dam underdrain network.

Keywords: *grouting, high-pressure leak, eroding foundation, Arapuni Dam.*

The dam

Arapuni Dam, located on the Waikato River 55 km upstream from Hamilton in New Zealand, is a 64 m high curved concrete gravity dam completed in 1927. The dam across the Waikato River bed forms the reservoir for a 186 MW hydroelectric power station, sited 1 km downstream at the end of a headrace channel that follows the left abutment. Penstock intake and spillway structures are on the headrace channel. A concrete lined diversion tunnel runs through the right abutment around the dam, with separate gate and bulkhead shafts. Arapuni dam and power station is owned and operated by Mighty River Power Ltd. The dam is shown on Figures 1 and 2.

The 64 m high concrete dam has a crest length of 94 m and a base/height ratio of approximately 0.73. Handman (1929) discusses the dam's construction. Original features of the dam include concrete cut-off walls and a network of porous (no-fines) concrete drains at the dam/foundation interface (the 'underdrain'). The cut-off walls extend beneath the dam and extend 20 m and 33 m into the left and right abutments, respectively, for the full height of the dam as shown on Figure 3. There was no grout curtain constructed during original construction.

The 'no-fines concrete' porous drain network (Figure 2) is the main uplift control at the dam/foundation interface. The underdrain includes a continuous drain, known as the circumferential drain, sited parallel to, and immediately downstream of, the upstream cut-off wall. The 600 mm high × 600 mm wide circumferential drain has radial porous drains discharging seepage water to the downstream toe, where seepage is measured at V-notch weirs.

Figure 1. Arapuni Dam, New Zealand, looking west.

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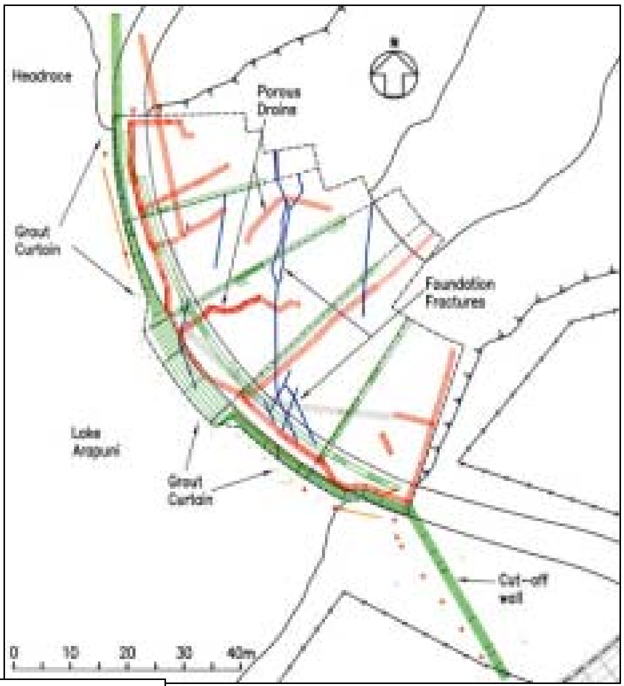
Figure 2. Plan view of Arapuni Dam. The positions of the foundation fractures noted during construction are shown. The diversion tunnel curves to the south of the right abutment cut-off wall.



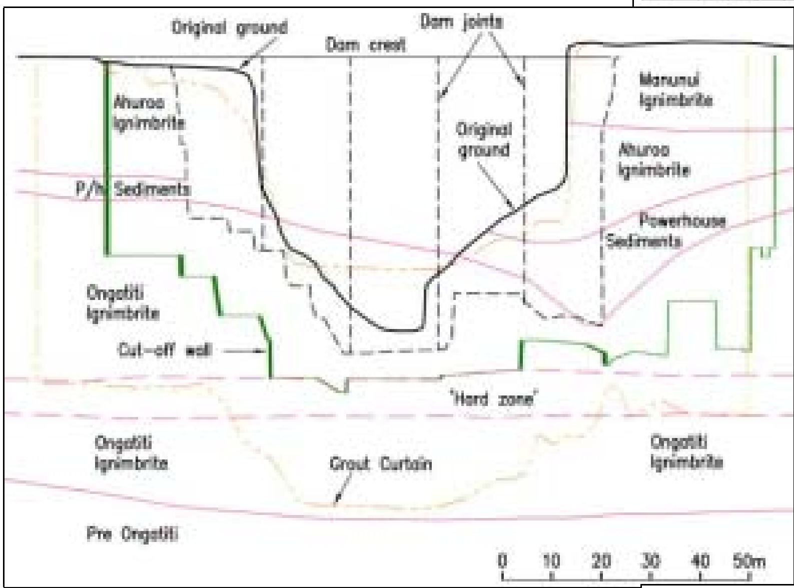


Figure 3. Elevation of Arapuni Dam, looking downstream.

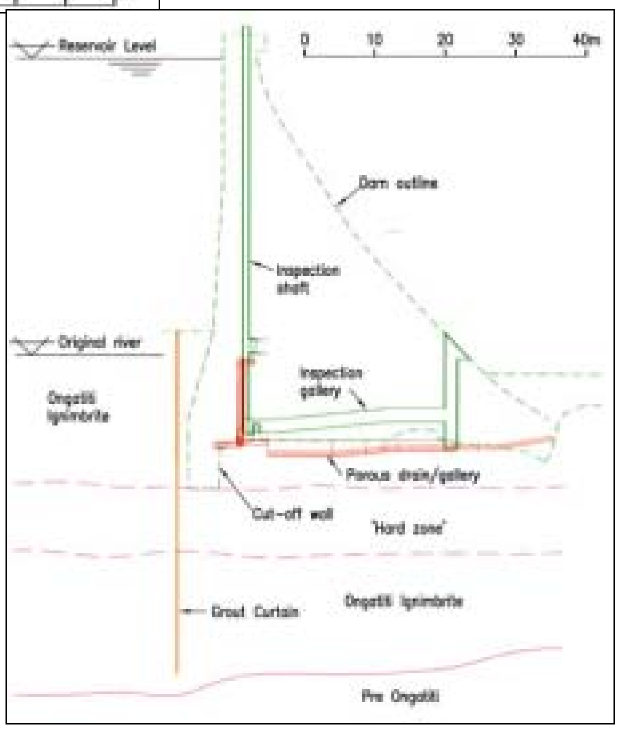


Figure 4. Cross-section of Arapuni Dam. (Note the spatial separation of the grout curtain from the dam or its concrete cut-off.)

In June 1930 the reservoir was dewatered following the development of a large crack in the headrace channel near the powerhouse. During the following two year period, while the headrace channel was being repaired, a grout curtain was constructed along the upstream heel of the dam and along the front of both abutment cut-off walls (Furkert 1934). The grout curtain was a single row cement curtain with mostly vertical grout holes at 3 m centres. It was constructed just upstream of the dam and cut-off walls, as shown on Figure 4, but does not appear to have been physically connected to the dam. At the steep gorge walls a bitumen plug was constructed between the dam and the gorge wall, and the grout curtain extended radially from the crest abutments. Figure 3 shows the extent of the grout curtain and cut-off walls.

The dam foundation

The dam site is in an area of multiple ignimbrite flows from volcanic eruptions over the last 2 million years. The ignimbrite deposits at the dam site are shown on Figure 3.

Two ignimbrite units form the gorge walls. The younger Mananui Ignimbrite is present as the upper unit on the right abutment only, while Ahuroa Ignimbrite is present on both abutments. Both ignimbrites are columnar jointed weak to moderately strong point-welded tuff.

The main dam footprint is founded on a 40 m thick sheet of Ongatiti Ignmibrite, a point-welded tuff. The upper part of the unit is very weak, with unconfined compressive strength of between 2 and 6 MPa, while below the dam cut-off wall Ongatiti is considerably stronger (up to 28 MPa) and identified as the ‘hard zone’ (Figure 3). A feature of this ignimbrite sheet is the lack of regular orthogonal vertical jointing often seen in ignimbrites. Three major subvertical cracks or fractures were identified during dam construction crossing diagonally across the dam footprint in an east–west orientation. These fracture extend for the full depth of Ongatiti and vary in aperture from closed up to 80 mm. The fractures relate to cooling of the ignimbrite after emplacement and are not tectonic in origin. Joint infill is generally present where the fracture opened around the time of emplacement.

Beneath Ongatiti, about 40 m below the base of the concrete dam, are older ignimbrite deposits, identified as Pre-ongatiti for this project.

At interfaces between ignimbrite sheets there tends to be unwelded material, either airfall tephras or unwelded ignimbrite. The most extensive interface deposit is between the Ahuroa and Ongatiti ignimbrite units, known as Powerhouse Sediments, with a thickness of 4–8 m.

Seepage history of the dam

The dam has experienced several seepage incidents since first filling. In the first two years of operation from 1928 to 1929, seepage from the reservoir reached 4200 litres/minute passing under the dam and issuing from the abutment rock faces immediately downstream of the dam. Several leakage connections were identified between the lake and the dam toe, via both the left and right abutments. These seepage paths appeared to be quite long and complex.

After remedial works, including construction of the dam grout curtain, the reservoir was refilled in April 1932. Total seepage flow was measured at 420 litres/min; an order of magnitude lower than before grouting. However from 1932 to at least 1945 there were several instances of sudden seepage increases of up to 120 litres/min. During this period, a number of deep drillholes in the right abutment between the diversion tunnel and dam were injected with hot bitumen grout. Overall there was no long-term effect on seepage flows that could be attributed to the bitumen grouting operation. Total seepage flow peaked at about 750 litres/min in 1943 before gradually declining to around 75 litres/min in 1960, and remained less than 75 litres/min until 1995.

In 1995, eight investigation holes were drilled from the downstream toe, angled upstream into the foundation under the dam to better understand foundation uplift pressures. These holes were percussion drilled to become drainholes and/or uplift measurement instruments. Two of these holes, known as OP05 and OP06, intersected high water pressures and each flowed at several hundred litres per minute after drilling. Closing the drains to measure pressure indicated leak pressure around 13 m below lake level with no relief operating. The high-pressure flow was encountered at depths down the holes that coincided with one of the sub-vertical fractures in the dam foundation recorded during dam construction. The drillers observed dirty water discharge from at least one of the holes for some time, indicating that some foundation material may have been eroded. At the time of drilling, the porous concrete underdrain system had very little seepage discharging (<15 litres/minute). Between 9 and 12 days after the drilling, the underdrain started to flow up to 200 litres/minute.

From 1995 to 2000 there was a clear increasing trend in pressure and flow. Hole OP06 was being used as a relief drain throughout this period, but pressures in the fracture had risen by at least 10 m. Fracture pressure (with relief drainage operating) was within 20 m of lake level pressure and this condition existed only 10 m horizontally from the downstream toe. By September 2000, solid material (clay and bitumen particles up to 20mm diameter) was observed to be exiting the drainage drillhole OP06. If erosion migrated along the line of the fracture and downstream of OP05 and OP06, then it was possible that an erosion pipe could connect to the downstream toe of the dam. The high pressure could have potentially blown out at the dam toe, and the resulting jet of water then eroded Powerhouse Sediments on the left abutment, destabilising the abutment rock face above.

In 1999 the diversion tunnel bulkheads were installed and the 35 m length of tunnel between bulkheads and gate dewatered for gate maintenance. During this period while the length of tunnel in the right abutment was dewatered, uplift pressures under the dam (i.e. in the fracture) were observed to increase significantly to within 7 m of lake level perssure. This was an unusual and unexpected observation.

Seepage investigations

While an immediate action involving grouting OP05 and OP06 could have been carried out in September 2000, this was not considered the best solution. Clearly an eroded path through the dam foundation had developed, but little was known about the seepage path, including the size of the void, nature of eroding material, the source of seepage water or the path of the leak. Furthermore during the dewatering of the diversion tunnel the dam foundation experienced pressures beyond those typically observed with the diversion tunnel full. Grouting OP05 and OP06 at this stage would have given an unknown result as to the effectiveness of the grouting. Excessive grouting pressures were also considered to have the potential to initiate blowout of the remaining fracture infill between OP06 and the downstream face. Furthermore, the leak was entering the dam's underdrain in at least one location, indicating that grouting OP05 and OP06 could lead to grout entering the underdrain and blocking it.

Therefore it was considered more prudent to investigate the leak, determine its path, and treat the leak when it was better understood. The investigations would be undertaken with the full reservoir in place, but in the knowledge that a deteriorating seepage condition existed in the dam foundation. The option to grout OP05 and OP06 was retained during investigations as a contingency should the dam's condition become unacceptable and action be necessary to stem the leakage flow.

The range of investigations undertaken are reported in Gillon & Bruce (2002). Investigation findings included:

- Four investigation drillholes intersected the leakage path. However, several other holes drilled into the same sub-vertical fracture did not intersect high pressures.
- High pressures were not present in other fractures or elsewhere in the central dam foundation.
- Leakage appeared to have sources in the lake bed and in the diversion tunnel. Lake biota including snails and fish (up to 5 mm) and algae was present in seepage flow exiting drain OP06.
- The leakage path was through the fracture infill and at least 80 mm wide in places, while being more constricted in other places.
- The fracture infill was identified as nontronite, an iron-rich smectite clay with a very high moisture content. This very weak clay is potentially easily erodible under pressure.

In October 2001 it was decided that sufficient investigation information had been gathered to identify the leakage path and have confidence that filling the void could be successful. Grouting of the open void in the main fracture in a high-quality, well engineered operation was programmed for early December 2001. It was recognised at the time of this decision that the prime objective of the fracture grouting operation was to fill the existing void to remove the high-pressure leakage with a secondary objective to preserve the existing foundation under-drain network. It was acknowledged that the highly erodable nontronite was likely to still remain intact in areas of fissure. While intact erodable material would not necessarily be treated in this operation, it could be treated once the high-pressure leak had been removed.

Organisation of grouting

Mighty River Power had overall responsibility for the grouting operation. Key individuals such as the Project Director and Project Manager were drawn from MRP's engineering staff. The Project Director reported to a Project Board consisting of company senior managers with responsibility for Arapuni Dam.

Mighty River Power had separately engaged an independent reviewer for technical review of the project and also an independent reviewer to audit project management systems and processes.

The grouting operation was carried out by a project team sourced from several organisations, with key team members appointed on a 'best-for-project' basis. This collaborative project style proved to be a practical and effective way to maximise use of the specialised expertise of the team members involved, and produced a cohesive team focused on the objective.

Mighty River Power engaged two experienced grouting supervisors from Austress Freyssinet to act as shift managers of the grouting teams. Grouting equipment was supplied by a local contractor, based on an equipment list specified by the Grouting Shift Managers. Mighty River Power purchased some specialist equipment such as grout monitoring and control instruments. The grout mixing and injection was undertaken by the local contractor, under the direction of the Grouting Shift Managers.

Drilling was carried out by two drilling contractors, each with experience drilling in different areas of the dam. One contractor drilled from the toe of the dam and the other from within the dam galleries.

Engineering design and detailed specification for grouting the feature was carried out by DamWatch Services, together with their specialist consultants Geosystems LP (USA). DamWatch were also responsible for on-site management of dam safety throughout the grouting preparations and during the grouting operation, including the responsibility for operating the relief wells to control pressures under the dam.

Preparations for grouting

The grouting objective was to control the high-pressure leak under the dam by filling the void with high quality grout while the reservoir remained full, i.e. place grout against high-pressure flowing water. Dam safety during grouting was therefore an important concern. Poorly managed grouting pressures had the potential to make the foundation condition worse. Care would be required not to over-pressurise the foundation and cause the high-pressure leak to either break out to the downstream toe or open up a leakage path in an adjacent fissure.

The highest foundation pressure recorded in the past was selected as the maximum limit for foundation pressure during grouting on the grounds that there had been no blowout of the fracture infill when this precedent maximum pressure occurred. Grouting pressures would not be allowed to push the overall pressure in the dam foundation above this limit. In order to meet this safety requirement and allow grout pumps to apply sufficient pressure to the grout, the dam foundation pressures during grouting would be deliberately lowered by relief drainage.

The design for fracture grouting had three key elements: depressurising the foundation using relief drainage, designing a grout mix suitable for application in high-flowing seepage water, and preserving the dam's porous concrete underdrain network.

Before grouting commenced, the seepage properties of the foundation were confirmed by testing so that a baseline could be determined, suitable for control of pressures during grouting and to determine if grouting had led to improvement in the seepage condition. Detailed procedures were developed for all pre-grouting tests of the foundation, underdrain and grouting equipment and for the actual grouting operation.

Pressure relief holes

Depressurising the foundation was done by using holes drilled into the high pressure fracture zone as relief drains. As well as the four holes drilled from the dam galleries (PZ08, PZ12, PZ13 and PZ15), a further five relief drains (PR01 to PR05) were drilled from the downsteam toe of the dam. Four of the new holes drilled from the dam toe made good connections to the high-pressure zone, while the fifth (PR04) was not as good a connection with the leak flow path. All of the holes into the high pressure leak were drilled with HQ size (96mm) core barrel, with steel casing from the collar down to within 2 m of the high pressure zone and a ball valve at the hole collar. Two of the holes drilled from within the dam had packers fixed down the hole with a 32mm riser pipe for the relief drain. Every hole into the high pressure leak was to be grouted.

Three further holes were also drilled from the downstream toe into adjacent fractures. These holes were to be monitored during grouting to determine if the leak had migrated into an adjacent fracture system. If flow was transferred to the adjacent fissure, the holes would be used for grouting the fracture receiving transferred high pressure seepage.

Grout equipment and mix design

Grout equipment was generally sourced in New Zealand. Important items of equipment included:

- High shear (or colloidal) grout mixer
- Large paddle mixer to hydrate bentonite
- A Mono (helical screw type) grout pump
- Piston grout pump for sand/grout mixes
- Paddle mixers to act as holding tanks at the toe of the dam.
- An electronic flowmeter grout control panel

The high shear mixer was considered essential for high quality grout. Paddle-type mixers were not acceptable. The helical screw type pump was preferred because the grout pressure could be more easily controlled. However, the pump was not suitable for sand/grout mixes, so the piston pump was available on standby. Equipment essential for the continuous grouting operation was duplicated to reduce the risk of a breakdown halting grouting.

Each drillhole had been fitted with branched pipework that allowed for grout injection line, pressure gauge and bleed pipe to test for grout arrival at the hole.

The setup of grouting equipment is shown on Figure 5, with the grout mixing station at dam crest level on the left abutment and the grout injection station at the toe of the dam.

The grout mixes to be used were cement-based grouts, with varying water/cement ratios, designed to be placed in either static or flowing water conditions. The mixes incorporated bentonite to reduce bleed, an anti washout agent and a superplasticiser. The proposed mix designs developed by the designers were trialed and tested in a mobile laboratory set up next to the mixing station. The mixes were modified as indicated by the tests to ensure the mixes would be stable, durable and possessed the appropriate rheological and hydration properties.

Seven mixes were finalised for possible use in the grouting. Four grout mixes had water/cement ratios in the range of 0.8:1 to 1:1 by weight and the other three mixes were sand/grout combinations to be used if a runaway condition developed during grouting. During grouting only two water/cement mixes were used, and no sand mixes were required.

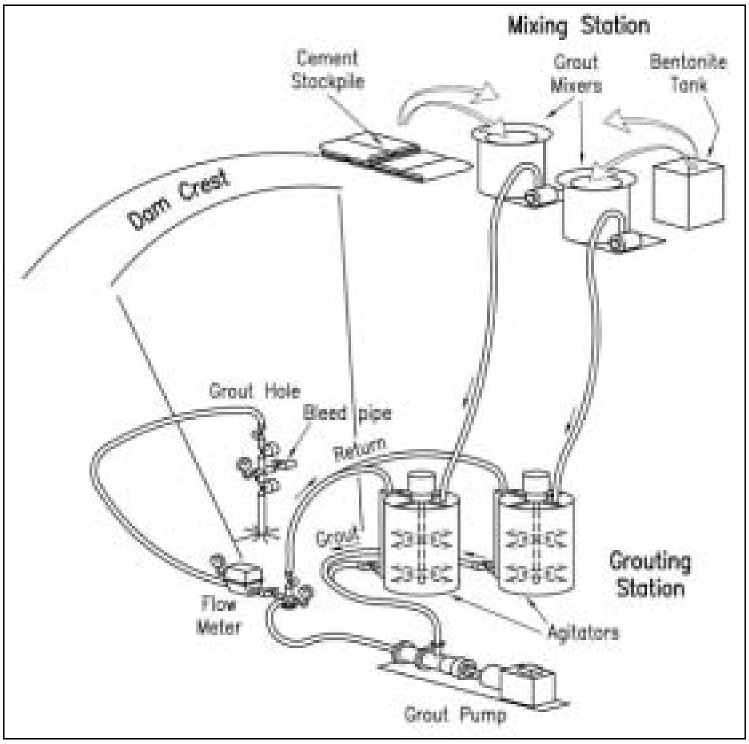


Figure 5. Grout equipment layout at Arapuni Dam.

Dam foundation underdrain preservation

The preservation of the porous drains forming the underdrain had two components:

(a) flushing the drain with large volumes of water to keep any grout mobilised, and

(b) preventing grout intrusion into the underdrain by sealing the connection between the leak and the porous drain with wood chips.

The flushing system which is shown in Figure 6 involved the installation of supply lines from a header tank high on the face of the dam to injection points at three locations at the toe of the dam. The two main injection points were at the right and left extremities of the underdrain (Gallery 1 and Drain 5 respectively), with a third injection point (drillhole PR09) close to the point of leak entry into the underdrain at Drain 2a. The third injection point (PR09) would only be used if grout was identified entering the underdrain. The header tank would be maintained full during grouting by using a pump from the reservoir.

Two additional holes were drilled into the underdrain (PZ18, and PZ09) to be used for, underdrain pressure relief and pressure monitoring respectively. These holes would also be used for monitoring to identify if grout entered into the underdrain.

The final preparation task before grouting commenced was the insertion of 0.1 m^3 of wood chips into the fracture to form a seal across the connection to the porous drain. Insertion was through drillhole PZ15 that intersected the high pressure fissure within 1 m of the underdrain. The wood chips, from a local timber mill, were wood shavings typically 30 mm × 15 mm × 2 mm thick, i.e. large enough to seal across the voids in the porous drain. A concrete pump sited on the crest of the dam was used to pump a water and wood mix to the drillhole. The wood chips were to be held in place across the connection to the porous drain by the pressure difference across the connection. To ensure the wood chips were drawn to the drain contact, relief drains in the high pressure zone were closed so that the underdrain became the only outlet for the high pressure.

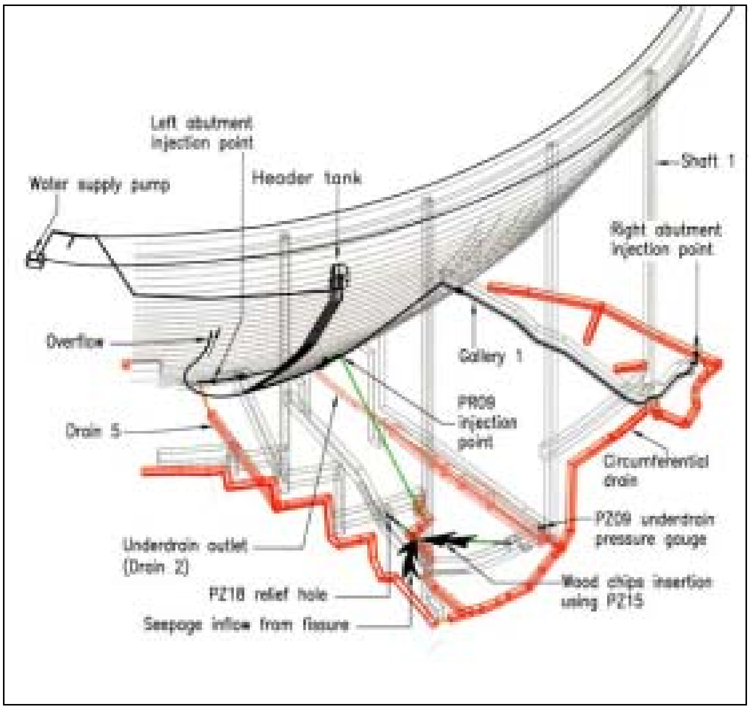


Figure 6. Method used for underdrain preservation at Arapuni Dam. (View looking obliquely downstream. Circumferential drain follows dam/foundation contact.)

The effectiveness of the wood chip seal was immediately apparent. Flow from the underdrain dropped by more than a half and underdrain pressure in the vicinity of the leakage entry point dropped by 15 m. Pressure in the fissure rose by 6m overnight following the wood chip injection because the pressure relief provided by the underdrain had been reduced.

Grouting

Grouting was planned as a 24 hour operation and commenced on 12 December 2001. The grout and QA teams worked on a shift basis. Grout teams were dedicated to the mixing station at the dam crest, the grout injection station at the toe of the dam or the flushing water supply station.

Three hours before commencement of grouting, the most upstream pressure relief well (PR05) was opened to depressurise the foundation by 20 m head. Two hours before commencement of grouting, the underdrain flushing operation was started. Water was injected from the left and right abutment extremities of the underdrain and flowed out from Porous Drain 2, the lowest level drain outlet. Flushing increased drain outflow to approximately 260 l/min, which represented the underdrain's maximum capacity.

Grout injection commenced from the most downstream hole (OP06) and was intended to work upstream towards the upstream pressure relief holes PR05 and PZ13. Grout flowed upstream at a fast rate. The order of grout appearance at other holes indicated that it first filled lower flow paths and then built up to appear at higher elevation holes. To minimise grout being washed out of the relief holes, PR05 was eventually closed after 6 hours and water was allowed to bleed out of the higher elevation PZ13. PZ13 was the final hole grouted.

The refusal criterion for each hole, except the final hole, was for zero grout take at the hole with the grout pressure at 2.5 to 3 bar. These limits were to ensure that the grout operation did not push dam foundation uplift pressure over the precedent pressures recorded in the dam foundation prior to grouting. The final (most upstream) grout holes PR05 and PZ13 were grouted to a pressure limit of 7.5 to 8 bar at refusal.

Grouting took 12.5 hours. A total of 11.5 m^3 of grout was placed. Of this total volume, 4.4 m^3 were placed in the upstream holes PR05 and PZ13, leading to the conclusion that grout placement had extended upstream of the dam.

Apart from isolated pH spikes, pH monitoring of the underdrain flow throughout grouting showed neutral results, indicating that there was no significant ingress of grout into the underdrain.

Quality assurance and quality control

The grout injection was controlled at the toe of the dam, close to the injection point, by managing grout pressure and flow rate. The grout station accurately monitored grout pressures, flows and volumes and a technician recorded the hole sequence and time history of grouting parameters.

Quality control at the mixing station was based on repeated laboratory testing by the on-site laboratory. Tests included Baroid Mud Balance, Marsh Cone, Bleed, Set Time and Strength. Testing frequencies were every batch, every fourth batch, or every hour, depending on the test.

Portable pH indicators were used to detect grout in the dam underdrain. If grout was detected, procedures were to be initiated to intercept the grout and flush it out of the drain close to the point of entry.

Dam safety during grouting

Grouting was a continuous round-the-clock operation once it began. To ensure continuity throughout the grouting operation, dam safety teams worked in shifts, with shift changes timed to avoid shift changes of grouting team.

Dam safety teams monitored key indicator instruments. A datalogger recorded readings from transducer instruments every 5 minutes. Recorded maximum precedent foundation pressures formed the upper limit for grouting pressures.

The dam safety team leader had the authority to suspend grouting at any stage if the grouting operation compromised dam safety.

Following completion of the gouting, foundation pressures have been closely monitored to validate the continued performance of the grouting work and to ensure the former high-pressure seepage does not develop an alternate path.

Verification of grouting effectiveness

As a direct result of the grouting, seepage flows from the dam foundation decreased from their former 800 l/min to 20 l/min. Foundation uplift pressures dropped by up to 17 m immediately beneath the dam. There were no indications of transfer of pressures to adjacent parallel fissure systems following grouting.

Four boreholes (PR10-13) were drilled from the dam toe into the grouted fissure immediately after grouting. Verification drilling did not detect significant flow or pressure in the fissure. One of the verification holes returned a core that contained a well grouted fracture with no signs of voids remaining. The other holes returned core with intact nontronite present, but no grout. Coupled with the observation that the total grout quantity injected was approximately one-third of the void volume assuming the fracture infill was totally removed, it appears that the void beneath the dam was a network of interconnecting discrete flow paths through the fissure infill that extended back to the reservoir. Based on grouting and verification data, a view of the extent of grout in the fissure is shown in Figure 7.

The underdrain capacity was tested in January 2002 and confirmed that grouting had not affected drain capacity.

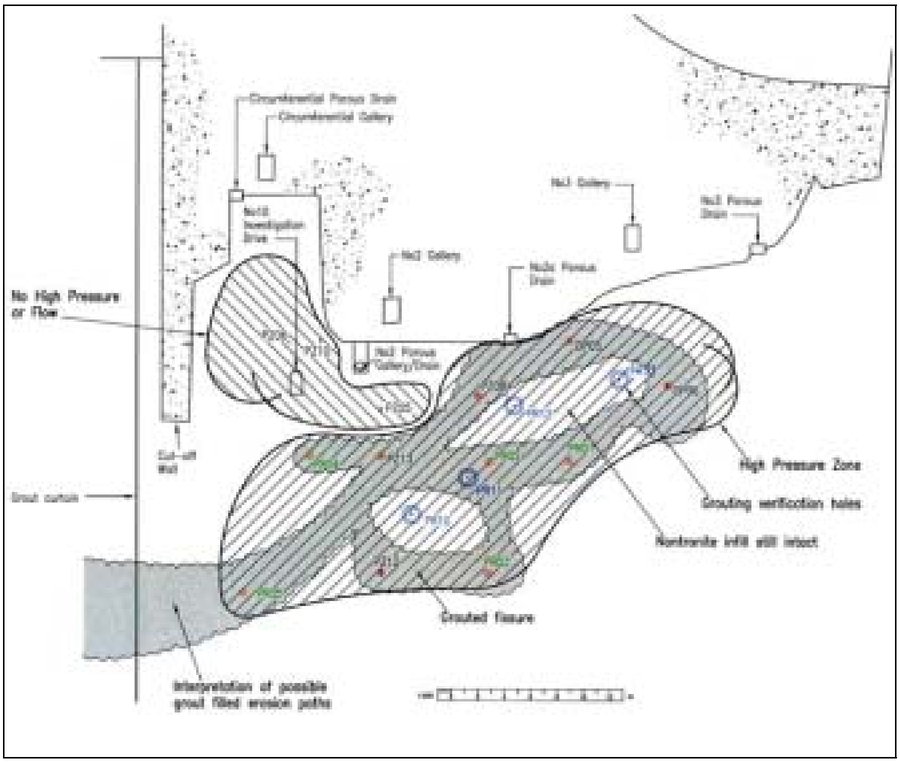


Figure 7. Elevation along main fracture, with conceptual interpretation of the grout distribution after treatment.

The diversion tunnel was dewatered in February 2002 to compare performance of the dam foundation to pre-grouting. Dye tests in the tunnel and flow and pressure monitoring at the dam did not show a repeat of the behaviour that had been observed pre-grouting.

Further work is under way evaluating the long-term seepage performance of the dam foundation coupled with a modern stability assessment of the dam to establish long-term performance. Any potential remedial measures necessary to achieve long-term performance will then be assessed.

Conclusions

The high pressures and seepage flows in a geological fracture under the foundations of Arapuni Dam were successfully grouted in December 2001.

Careful planning and the application of precautionary measures during grouting, such as pressure relief and sealing off the underdrain, allowed a comprehensive and well executed grouting operation to be run, without compromising dam safety. A large void allowing high-pressure, high-flow leakage from the reservoir to be present under the dam had been successfully filled using modern grouting practices, while the reservoir remained fully operational.

The post-grouting diversion tunnel test indicated that the former major seepage path from the tunnel to the dam foundation had been closed by the grouting operation.

An integral part of the success of the grouting was the flushing system, in combination with wood-chips to seal the connection between the high-pressure leak and the underdrain system. This prevented filling the underdrain with grout and preserved its foundation drainage function. Preservation of the dam drainage system was a significant achievement, as remediation of a damaged porous drain system under the dam would have been a very expensive operation.

Acknowledgements

Many people from a variety of organisations worked together very effectively to implement the preparations within eight weeks and successfully complete a unique grouting operation in difficult circumstances. The authors would like to acknowledge the contributions of all these people. The authors would also like to acknowledge the support from Mighty River Power Ltd and their advisors throughout this project. Special acknowledgement from the project team to the late Dr Sergio Giudici for his valued contribution to the success of this phase of the project.

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Tailings dams in New Zealand

Trevor Matuschka[1]

This paper reviews the design and construction of tailings dams and compares their features with those of water storage dams. In New Zealand, legislative requirements are similar to those for water storage dams, and consenting agencies take a conservative approach with respect to potential environmental impacts. Current design, construction and operation practice of the tailings dams associated with the Martha Mine at Waihi and the Macreas Flat Mine in Otago are described.The dams associated with these mines have been designed using the downstream construction method, and constructed using overburden materials from the pits from where the ore is extracted. This is the safest form of construction for tailings dams. One dam at Macraes Flat has recently commenced using the upstream construction method to raise an existing embankment constructed by the downstream approach. It is concluded that the performance of the dams to date has been very satisfactory and this can be attributed to the conservative design approach adopted.

Keywords: *tailings dams, gold mines, design, construction method, legislative requirements, Waihi, Macraes Flat.*

Introduction

The last 15 years has seen the re-emergence of gold mining as a significant industry in New Zealand. Three large hard rock gold mining operations have been developed (Martha and Golden Cross Mines at Waihi and the Macraes Flat Mine in Otago). Another is currently planned for Reefton. These mines produce large quantities of tailings that are required to be stored in impoundments formed by dams.

Gold mining was a significant industry in New Zealand in the late 1800s and early 1900s. At this time it involved both hard rock underground and alluvial mining. With hard rock mining the gold ore is bound to the host rock, and crushing and chemical treatment is required to extract the gold. With alluvial mining the gold is free and so the process to remove the gold is much simpler. Methods of hard rock mining have changed with the development of large machinery that permits economical mining of gold by excavating large open-pits. Current pits can be over 200 m deep, greater than 1 km wide and several kilometres long. Mining this way, together with efficient processing plants, enables low grade ore (down to 0.7 grams of gold per tonne of ore) to be recovered economically. Historically, underground mining only mined high grade ore, and significant quantities of low to medium grade ore were left untouched. Many current open-pit mines involve excavation of previous underground workings.

The modern open-pit mining process produces large quantities of overburden material (mostly rock) as well as large quantities of ore. The ratio of ore to waste (overburden) rock is typically of the order of 1:10. The ore is ground (milled) to particle sizes generally less than about 150 microns. Tailings are the residue left after the gold is extracted. The grain size distribution of the tailings depends on the characteristics of the original ore (rock type, degree of weathering and alteration). Typically tailings materials range in grain size from fine sand to clay-size, although the proportion of sand, silt and clay can vary significantly. Tailings are pumped as a slurry to be stored in impoundments that are typically formed by embankment structures. Such embankments are referred to as tailings dams. At the point of deposition into the impoundment the tailings segregate. The coarser fractions of the tailings tend to settle out nearer to the point of discharge and so tailings properties vary across an impoundment. Tailings associated with gold mines in New Zealand typically have permeabilities in the range of 10^{-6} to 10^{-9} m/s following deposition.

Characteristics of tailings dams

Tailings dams are essentially a 20th century invention. Prior to this the residues from mineral extraction were generally discharged either on to the surface close to the mine concerned, or into the nearest watercourse for

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disposal downstream. For example, at Waihi most tailings from the underground mines in the area were discharged directly into the Ohinemuri River, which was designated a sludge channel by an Act of Parliament. At times the river was apparently choked by tailings and this led to periodic flooding of the township of Paeroa. Ultimately the tailings washed into the Firth of Thames.

Worldwide, several thousand tailings dams are estimated to exist. Such dams store tailings from not only gold mining, but also all other types of mineral processing including copper, nickel, lead, zinc, bauxite, fertiliser minerals, uranium and coal. They range considerably in size from small to very large. One of the largest in the world is the Magna tailings impoundment in Salt Lake City, Utah. The impoundment occupies 2300 hectares with a perimeter of about 25 km that is formed by a 75 m high dam. There are a number of tailings dams around the world that are over 100 m high.

Large tailings storage dams are normally built in stages. In the first stage an initial (starter) embankment dam is constructed before the mining operation starts. This first-stage dam may store water, either intentionally or because there is no way to release water. In these cases the dam is normally designed as a conventional water storage dam. During many operations the dam is subsequently extended in stages or continuously during mining operations. In general terms there are three methods of constructing tailings dams; upstream, downstream and centreline (Figure 1).

With upstream construction, successive lifts (stage embankments) are achieved by constructing embankments on top of previously placed tailings. With downstream construction the embankment is lifted by placement of fill downstream of the starter embankment. With centreline construction the embankment is

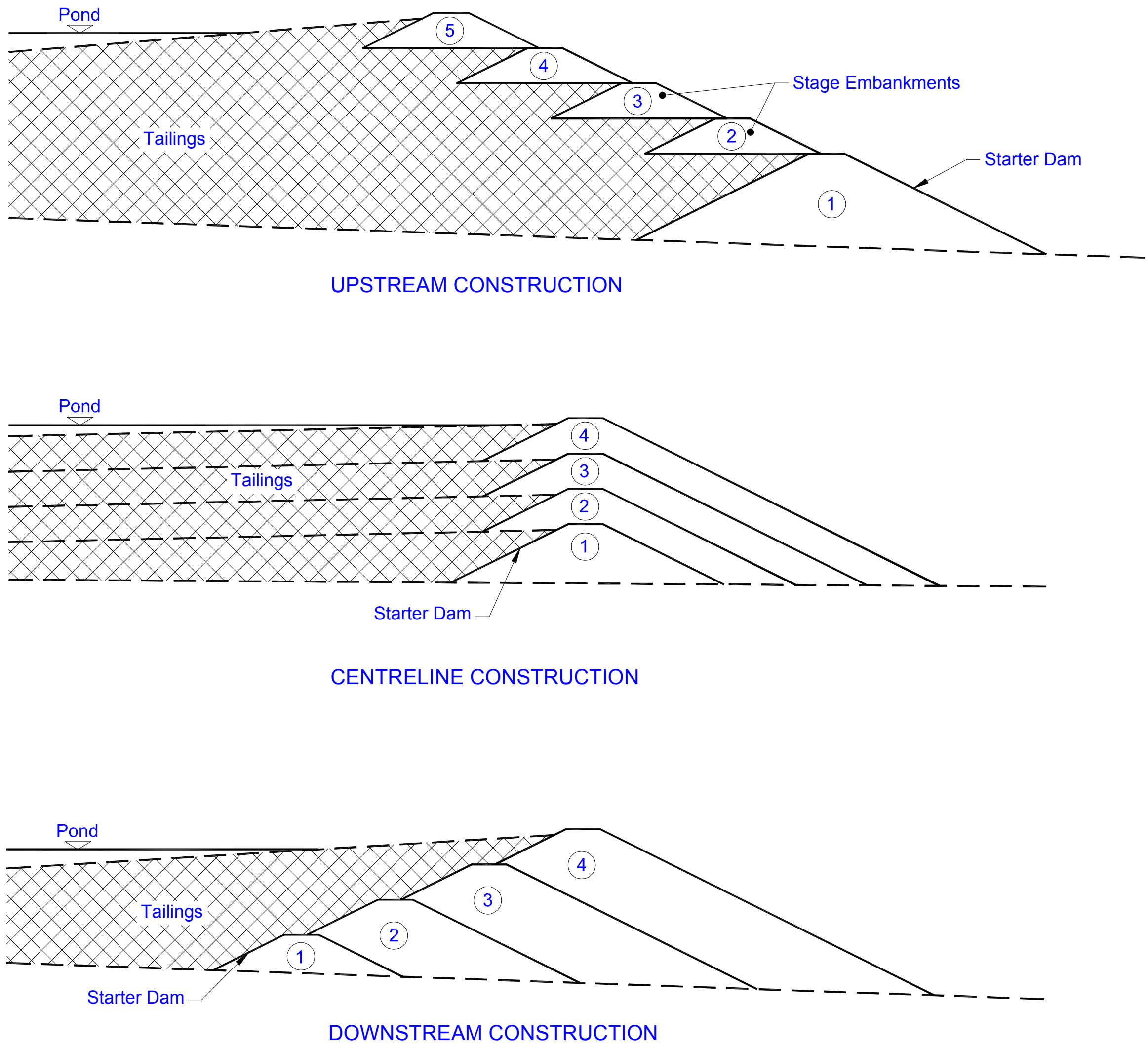

Figure1. Common types of tailings dam.

raised by placement of fill both upstream on previously placed tailings and downstream.

It is possible to have combinations of different construction methods. For example, a dam can commence as downstream construction and later be raised by upstream construction. The embankments can be constructed from waste rock, if sufficient quantities are available, or from tailings materials themselves. Many overseas tailings storage dams have been constructed from tailings materials. This practice is more economical than using waste rock. The tailings are cycloned to separate the sand sizes from the slimes. The cycloned sands are used in the dam construction and slimes are deposited in the tailings pond. In this case seepage through the dam wall occurs, and is often collected in a reclaim pond located downstream of the dam.

The most conservative form of tailings dam construction is downstream construction, using waste rock materials. This is similar to conventional earth/rockfill dam construction. A less conservative form of construction is upstream construction, because the integrity of the impoundment is dependent on the strength of the tailings. If the level of saturation of the tailings rises too high, instability can occur.

Tailings dams can take different forms, depending on the site (e.g. cross-valley, perimeter dyke, hillside, or incised). In New Zealand, because of limited availability of large flat areas, goldmine tailings dams are either cross-valley (e.g. Golden Cross and Macraes) or in the case of Martha, a dyke which abuts against a hillside. Cross-valley dams are proposed at the Reefton Gold Project site.

There have been some notable incidents involving the failures of tailings dams that have led to fatalities and significant environmental damage. Most failures have occurred as a result of either percolation through the dam wall, internal erosion, overtopping or flooding, and may be triggered by natural phenomena such as earthquake or persistent heavy rain. Failures have been attributed to a combination of inadequate design and investigation, and poor construction and operational procedures. Fortunately, large-scale incidents are infrequent. A study published in 1996 (1) identified fewer than 10 major failures or similar incidents between 1980 and 1996, which is small compared to the number of tailings dams worldwide.

The design and construction of tailings dams has evolved considerably over the last 25 years. For example, the International Commission on Large Dams (ICOLD) has published a number of guidelines (2-7) covering their design, construction and operation, and these are of particular assistance to designers and operators of tailings dams.

Comparison with water storage dams

Tailings storage dam design differs from conventional dam design in several ways.

Rehabilitation. Unlike conventional water storage dams, tailings storage dams cannot be breached at the end of their useful service and the valley allowed to return to its original condition. Thay often store hazardous fluids and solids, which cannot be discharged at the end of the operating life of the mine. Therefore the rehabilitation aspects of tailings dam operations require careful study.

Consistency of tailings. The bulk of material stored behind the dam is saturated, relatively impervious slurried tailings at various stages of consolidation. The consistency of the tailings may range between the solid state and the semi-fluid state. Under seismic loading, saturated tailings may liquefy, becoming a semi-fluid of high unit weight and with low residual strength. This is of particular importance for upstream construction dams.

Monitoring during construction. Most of the dam construction is carried out by the mining operators, with the dam being raised as required to stay ahead of the rising tailings pond. Because tailings storage dams usually are constructed slowly over a period of many years, the designer is able to select a design and then check its performance, making modifications as required throughout the long construction period. This can allow more flexibility than is available for design of conventional water retention dams. Historically, construction by mining operators presented risk from less attention to quality control than would be the case for conventional dam construction. However, this is not the case nowadays, especially for larger dams.

Upstream support. Tailings storage dams normally are subjected to only a nominal amount of drawdown of the overlying free water, as the tailings deposits are continually rising as the dam is raised. Consequently tailings storage dams develop a significant amount of upstream support from the tailings in the pond and this feature can be utilised in their design.

Materials available. In tailings storage dams there is not the same freedom of materials selection that there is with water storage dams because, for economic reasons, most of the construction material must come from the mine-pit. However, this concern is often offset by the fact that large quantities of waste rock are available so that conservative embankment dam shoulder slopes can be adopted.

Year-round construction. Mining operations normally continue throughout the year in all but the most severe weather conditions, including the disposal of waste rock and the construction of tailings storage dams. In comparison, water storage dams are normally only constructed during months of favourable weather. Consequently tailings storage dam design must make some allowance for this in the design strengths adopted and in the standards specified. Provision must be made for placement of unsuitable materials in appropriate zones. Construction may be programmed so that most structural fill is placed during favourable summer months with largely non-structural fill in winter months. In some cases material suitable for structural fill may be stockpiled in winter for later use in summer.

Consent issues in New Zealand

The legislative requirements for tailings dams are similar for conventional water storage dams. The two acts which take safety and construction into account are the Resource Management Act 1991 and the Building Act 1991. In an emergency situation, the Civil Defence Emergency Management Act 2002 may also apply.

The issues of greatest concern to consenting agencies are dam safety and potential environmental effects. Normally the resource and building consent conditions require tailings dams to be designed, constructed, operated and maintained in accordance with the general principles of the New Zealand Society of Large Dams (NZSOLD) Dam Safety Guidelines (8). Regular formal design and performance reviews are normally required to be undertaken by independent engineers.

Consenting agencies take a conservative approach with respect to potential environmental impacts. Extensive monitoring is required as a condition of the consents, with regular review and reporting. Typically this involves sampling and testing of groundwater from monitoring wells and discharges from seepage collection drains as well as receiving waters. Bonds are required to be put in place to cover rehabilitation in the event of premature closure of the mine and for mitigation of potential adverse environmental impacts.

New Zealand examples

In the following sections, case histories of tailings dams at the Martha and Macraes Flat Mines are presented. The tailings dam at the Golden Cross Mine, which has now been decommissioned, is documented in Weston & Jacobs (9). The tailings dams at all three sites have been designed using the downstream construction method, and constructed using overburden materials from the pits where the ore is extracted. This is inherently safer than using tailings, as is the case for many overseas tailings dams.

Martha Mine, Waihi

Martha Mine is located in Waihi, 115 km southeast of Auckland and is owned and operated by Newmont, an international gold mining company. The mine previously operated from 1882 to 1952 using underground methods. Mining recommenced in 1987 using an open-pit, re-working the area that had been previously mined by underground methods. Currently the annual ore production is approximately 1.3 Mt with gold production around 110 000 ounces.

There are two tailings dams at the Martha Mine, known as Storage 1A and 2. The dams are located immediately adjacent to two water courses (Ohinemuri River and Ruahorehore Stream, respectively) on land that was previously dairy farms. The climate is temperate and the annual average rainfall at the site is approximately 2000 mm. The site is located in a region of low historic seismicity, but the possibility exists for moderately strong shaking generated by earthquakes associated with movements on active faults.

Geological materials at the dam sites typically consist of a mantle of volcanic ash (approximately 3 m deep) overlying rhyolite. The rhyolite is jointed, but weathering has resulted in low rock mass permeability at shallow depths. Dacite and ignimbrite are also present in some parts of the site. An important feature is upward flowing groundwater, as evidenced by springs in gullies. This upward rising flow is especially important because it provides natural security to prevent any seepage from the tailings entering the deep groundwater system. Tailings from the Martha Mine contain a relatively high proportion of silt and claysized particles because of the highly weathered nature of some of the ore, and so are of low permeability (less than 10^{-9} m/s following consolidation).

Construction of Storage 2 commenced in 1987 and it reached its final crest height in 2001. Storage 1A commenced construction in 1999 and was commissioned in 2001. A typical cross-section of the Storage 1A embankment is shown on Figure 2. The zoning for Storage 2 is similar. The maximum height of both dams is approximately 60 m and the crest length is also similar, approximately 1800 m. The storage capacity of each impoundment is approximately 10 Mm^3.

Both tailings dams are constructed from mine waste rock that is transported by conveyor and offloaded into dump trucks, and selectively placed in the embankments. Waste rock consists predominantly of variably weathered andesite, but also includes significant quantities of ignimbrite (varying from hard rock to com-

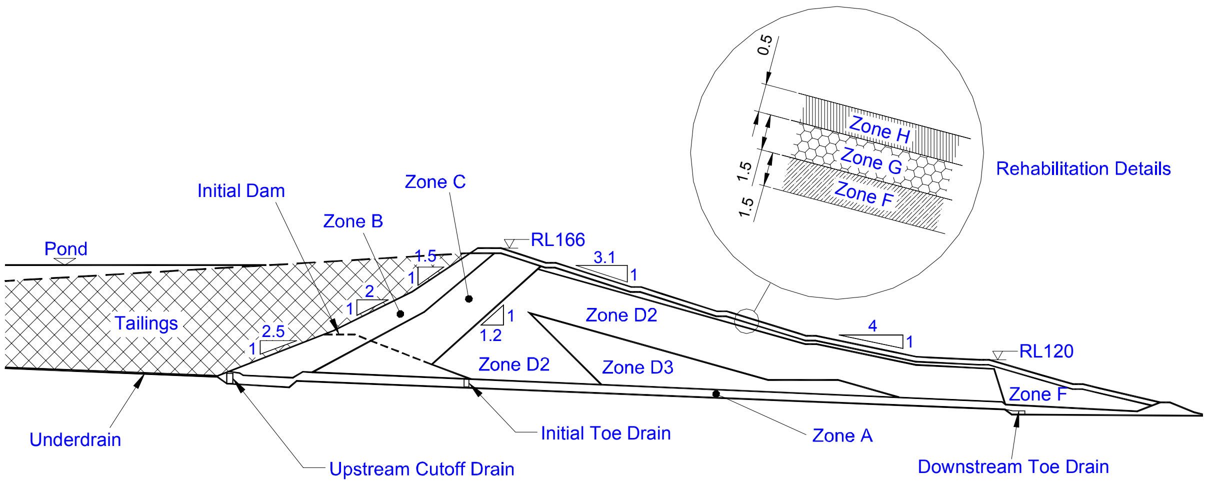

Figure 2. Martha Mine, typical section Storage 1A embankment.

pletely weathered soft soil) as well as some quantities of volcanic ash and lacustrine deposits. A special feature of the andesite waste rock is that some of it is potentially acid-forming (PAF) due to the presence of sulphides. This is considered in the design of the dams. Large quantities of non-acid-forming (NAF) soil and completely weathered rock have been stockpiled from the early stages of overburden removal of the mine pit for later use in completing construction of the tailings dams to the required standards.

The dams are designed essentially as water-retaining embankments, except that there are no special internal filter zones, and extensive subsurface drainage is included to control tailings seepage and leachate from PAF waste rock. Their geometry has been designed to accommodate all waste rock and tailings from the project. Downstream shoulders are considerably flatter than would normally be the case for water-retaining embankments because of the need to dispose of large quantities of waste rock.

Construction of the dams requires the selective use of and zoning of waste rock materials. Target crest levels are reviewed on an ongoing basis to ensure the crest is advanced ahead of tailings storage requirements. Embankment zoning provides for:

- restriction of tailings seepage;
- safe long-term stability;
- control of generation of acid drainage in the short and long term;
- interception and collection of tailings seepage and waste rock leachate for treatment;
- rehabilitation of the downstream shoulder to pasture and native plantings.

Due to the presence of PAF rock, special design and construction measures are required. PAF material is placed upon a low permeability underblanket and drains collect any leachate that is generated. Oxygen and water ingress is limited by rolling and compaction, and the placement of intermediate sealing layers. During construction, limestone is applied to exposed PAF rock to control acid generation. In the long term, generation of acid leachate is prevented by isolating PAF rock from atmospheric oxygen by the construction of a special layer on the outside of the dam (Zone G).

The embankment zones are shown on Figure 2:

Zone A. Forms the base underblanket. It is 1.5 m thick and is constructed from low-permeability materials with no acid drainage generating potential. It is required for internal leachate and foundation seepage control. It has a maximum permeability of 10^{-8} m/s.

Zone B. Forms the upstream structural shoulder of the dam and controls seepage from the tailings. It is normally constructed from a blend of weathered and unweathered waste rock materials including PAF material. The maximum allowable permeability is 10^{-8} m/s.

Zone C. Forms the remainder of the upstream structural portion of the dams. PAF material is acceptable.

Zone D. Forms the bulk of the dam. Includes PAF material. Weaker material is confined to Zone D3.

Zone E. Sub-compartments within Zone D comprising the softest and wettest waste rock.

Zone F. Forms a transition zone between waste rock in Zone D and Zone G. It is structural fill (1.5m thick) and also contributes to reducing the risk of earthquake-induced shallow-seated deformation.

Zone G. Outer seal layer (1.5 m thick) that acts both as an oxygen diffusion barrier and controls infiltration of water into the dams. Constructed from material with no acid drainage generating potential. Construction specification includes limits on permeability (not greater than 10^{-8} m/s) and degree of saturation (average not less than 90%).

Zone H. Forms final rehabilitation cover to downstream shoulder (0.5 m thick). This zone is overlain by a topsoil layer.

The dams and impoundments include a large network of subsurface drains to intercept tailings seepage, leachate from waste rock and groundwater. These drains include underdrains beneath the tailings, an upstream cutoff drain along the upstream toe of the dam, an initial toe drain and downstream toe drain, and gully subsoil drains. In addition, leachate collection drains are present within the dams. Seepage from these drains is collected at various sumps and pumped back to the process plant or water treatment plant.

Surface runoff from upslope of the tailings pond is diverted away from the area. The embankment crest level is designed to provide a safe height above the tailings plus stored water level. Storage capacity is provided for a 1200 mm rainstorm (probable maximum precipitation), plus 1 m of freeboard. This is to ensure the prevention of overtopping.

Construction of the dams is undertaken by contractors using conventional earthmoving equipment, with dump trucks up to 150 tonne payload. Supervision of the works is by Newmont staff, with input from the designer as necessary. Control testing to confirm specified earthfill standards are achieved is undertaken by an independent testing agency, with two staff on site over the summer months. Careful planning of construction activities is required to achieve the design objectives. Fill zones with low permeability requirements are generally constructed over the summer months. However, mining continues throughout the year and large volumes of fill are required to be disposed of in sometimes quite difficult conditions. A high level of supervision has been adopted, with three full-time Newmont staff dedicated to managing, planning and supervising construction and monitoring the performance of the dams.

Monitoring of the dams includes a network of piezometers (pneumatic, vibrating wire, and standpipe) within the dam and foundations, benchmarks on the outside shoulder and measurement of flows from all subsurface drains. This information is measured at regular intervals and evaluated. The designer prepares an annual inspection report. An independent peer review panel (appointed as a condition of the resource consents) consisting of experts in the fields of geotechnical engineering, geochemistry, hydrogeology and rehabilitation, also carry out inspections on a regular basis, review data and reports and provide independent reports to the Waikato Regional Council.

The performance of both dams to date has been very satisfactory. Deformations of the dams have been small and pore pressures in the downstream shoulders of the dams are low. Monitoring has indicated limited seepage through the dam and that the subsurface drains have functioned to intercept seepage from the impoundment.

Macraes Gold Project, Macraes Flat

The Macraes Gold Project is located approximately 60 km northwest of Dunedin. It was mined intermittently by underground methods from the 1890s to the 1940s. In 1990, mining recommenced with a single open-pit. Since then the owners (GRD Macraes) have opened an additional six pits. Currently the annual ore production is approximately 5 Mt with gold production around 175 000 ounces. Stripping ratios are high and a total of 43 Mt of waste and ore is forecasted to be moved in 2003. These large quantities of material are moved by a fleet of large excavators (3 × 180 t and 1 × 320 t) and dump trucks (8 × 250 t and 4 × 320 t) that operate 24 hours a day.

There are three tailings dams known as the Mixed, Concentrate, and Southern Pit Option 10 Tails dams. The Mixed and Concentrate dams are located across an incised valley. The Southern Pit Option 10 dam is constructed across a previously mined pit. A fourth dam has been consented and construction is to commence shortly.

The annual average rainfall at the site is only 630 mm. During winter it snows occasionally and the ground freezes which affects dam construction. The site is located in a region of low historic seismicity, although the possibility exists that strong shaking could be generated by earthquakes associated with movements on active faults. The average fault recurrence interval in the region is estimated to be in the range of 5000 to 12 000 years. The Alpine Fault, with a recurrence interval of approximately 250 years is located 200 km northwest of the site.

The site is located in the Otago Schist Belt. Schist rock within the mineralised shear zone is predominantly pelite and semipelite, with blocks of psammite. Beyond the mineralised shear zone the schist is predominantly psammite. The weathering profile varies, but is relatively shallow (5–20 m). A thin (0.1–3 m) veneer of loess and colluvium mantles the site area.

Tailings from ore processing are a sandy silt and consequently are considerably more permeable than the tailings at the Martha Mine. Following deposition into the impoundments from the dam crest the tailings segregate. Typically tailings immediately upstream of the dams have a permeability of close to 10^{-6} m/s.

The height, crest length and storage capacities of these three dams are summarised in Table 1.

Table 1. Details of the three tailings dams at Macraes Flat.

	Dam height(m)	Crest length(m)	Storage volume(Mm^3)	Construction commenced
Mixed	145	2300	31.5	1990
Concentrate	75	700	1.6	1990
Option 10	53	430	4.5	2001

The Mixed Tails dam has continued to be raised throughout the life of the project and has provided most of the tailings storage requirements. In 2001/2002, another tailings dam (Southern Pit Option 10) was constructed and this provided tailings storage for a period of approximately 15 months.

All three dams are of downstream construction. A typical cross-section of the Mixed Tails Dam is shown on Figure 3. The zoning for other dams is similar. Originally the Mixed Tails Dam was designed with the crest at RL490, but was redesigned with the crest at RL515. This was achieved by steepening of the upstream and downstream shoulders. Currently it is being raised above RL515 by upstream construction. These modifications have been necessary to accommodate increased tailings production. The decision to adopt upstream construction was undertaken only after careful evaluation of the properties of the tailings and monitoring of the pore pressures in the tailings over three years. The results indicated the tailings immediately upstream of the dam are in a drained and medium dense state, so the risk of liquefaction leading to failure is acceptably small. The proposals for upstream construction were prepared by the designer (Engineering Geology Ltd) and reviewed by international experts experienced in this type of construction as part of GRD's internal review process and also as part of the consenting process. An important part of the upstream construction is managing the deposition of the tailings so as to achieve maximum segregation and density of the tailings in the area beneath and immediately upstream of the stage embankments. To achieve this the tailings are discharged via closely centred spigots, the area of deposition is rotated so as to allow resting and drying, the ponded water volume is kept to a practical minimum, and the pond is kept away from the area of upstream construction. To maintain the tailings in a drained condition a network of drains is constructed in the tailings beneath the section of upstream construction.

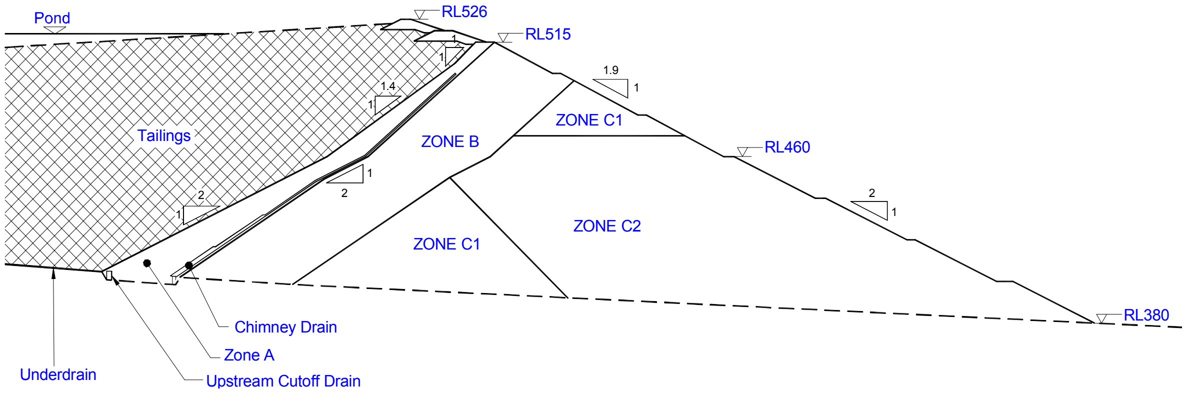

Figure 3. Macraes Gold Project, typical section, Mixed Tails embankment.

The tailings dams are designed essentially as water-retaining embankments. They are constructed from waste rock obtained from stripping of the pits. The geometry of the embankments is based on providing long-term stable structures, with the downstream shoulders capable of being rehabilitated to a standard equivalent to the surrounding country (i.e. grass and tussock). The zoning is relatively simple, with a low-permeability zone (Zone A) located on the upstream side to restrict seepage from the tailings. Zone B is structural rockfill placed in lifts of 0.6 m. Zones C1 and C2 are bulk rockfill zones placed in 2.5 m and 7.5 m lifts, respectively. A chimney drain is incorporated, but only in the lower elevations of the dams and on sections of the dam that are considered more prone to cracking from differential settlement (e.g. where the dam spans the deeply incised valley floor). Where the chimney drain is not present, special attention is given to placing Zone B rockfill with a high proportion of fines immediately downstream of Zone A so as to provide a transition zone.

Special care is required to select material that is suitable for Zone A. At the initial planning stages for the project, consideration was given to obtaining clay materials from off site. However, compaction trials indicated that the more weathered schist from the open-pit could break down sufficiently under heavy compaction to provide suitable low-permeability fill. The Specification for Zone A includes limits on dry density, water content and grading. The grading envelope ensures there are sufficient fines to achieve the permeability, but also limits excess quantities that could otherwise affect the strength and compressibility of the fill. Zone A fill can be placed in loose lifts of up to 350 mm provided compaction plant includes a self propelled protruding foot steel wheeled roller of not less than 30 tonne weight, primarily for breaking down the rock, and a large self propelled rubber tyred roller of at least 90 tonne (typically a Cat 773 watercart). To consistently achieve the specified standards it does, however, require special effort in identifying suitable material in the pit and in placing, mixing, conditioning and compacting. Zone A fill comprises material ranging from silt size through to gravel (typically 20–25% silt, 25% sand, and 55% gravel following compaction). Due to the high gravel content and low cohesion of the silt, the material is susceptible to segregation. *In situ* permeability tests indicate permeabilities are generally in the range of 2–5 × 10^{-8} m/s.

Subsurface drains include underdrains beneath the tailings and an upstream cutoff drain along the upstream side of the dam to intercept seepage from the tailings (Figure 3). These drains also assist consolidation of the tailings and enable higher densities to be achieved. There are various outlets from these drains constructed beneath the dams. Outlet pipes are laid in trenches and surrounded with concrete that includes an expanding agent. Seepage from the chimney drains and in the gully beneath the dams is also piped to collection sumps. The total seepage form the various subsurface drains associated with the Mixed Tails Dam has varied between 12–18 l/s.

Construction of the dams has been undertaken by various contractors under the supervision of GRD staff. GRD also undertake control testing to confirm embankment fill standards are achieved. More recently, specialist earthmoving contractors have been used just to construct Zone A and the subsurface drainage. The mining fleet provides the fill for Zone A in stockpiles and places Zones B and C. Foundation preparation beneath Zone A involves removal of all loose and weathered rock down to solid, tight rock (typically 1–3 m below original ground surface). There are specific requirements for shaping of the foundations to prevent stress concentrations in the overlying fill. All foundation areas are cleaned off with compressed air to enable observation of defects and to assess the need for any special treatment such as slush or pressure grouting or the installation of subsoil drainage. In Option 10, low-pressure grouting was used extensively to infill old blast holes and to lower the permeability of areas loosened by previous blasting.

The dams are instrumented with networks of piezometers and benchmarks. Flowmeters are installed on most subsurface drain outlets. A large number of piezometers are installed in the tailings beneath the section of upstream construction associated with the Mixed Tails Dam. Formal monitoring and surveillance procedures are followed as detailed in Operations, Maintenance and Operations Manuals prepared by the designer. Surveillance inspections are conducted at frequent intervals. Monitoring is undertaken at regular intervals, and monthly summaries including earthfill control tests are provided for review by the designer.

The performance of the dams to date has been satisfactory. Piezometers indicate that saturation and seepage through Zone A does not occur at higher elevations. This is attributed to the permeable nature of the tailings immediately upstream of the dams and the presence of subsurface drains to intercept tailings seepage. Settlements of the downstream shoulder of the Mixed Tails Dam have been in the range of 0.5–1% of the depth of fill. It is considered likely that some of the measured settlement will be due to creep of the downstream shoulder rather than vertical compression of the rockfill. The rate of settlement has decreased with time. Measurement of seepage flows indicate that once tailings deposition ceases, seepage from the impoundment decreases. While tailings were being deposited in Option 10, the Mixed Tails Dam was resting and seepage flows decreased by approximately 40%. At the same time pore pressures in the tailings dropped.

Conclusion

In the last 15 years a number of large tailings dams have been constructed to provide containment for tailings associated with hard rock open-pit gold mines in New Zealand. The dams associated with the Martha and Macraes Flat gold mines have been designed using the downstream construction method, and constructed using overburden materials from the pits from where the ore is extracted. This is the safest form of construction for tailings dams. One dam at Macraes Flat has recently commenced using the upstream construction method to raise an existing embankment constructed by the downstream approach. The performance of the dams to date has been very satisfactory and this can be attributed to the conservative design approach adopted.

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