DESIGN AND CONSTRUCTION OF SEEPAGE CUT-OFF WALLS UNDER A CONCRETE DAM IN NEW ZEALAND WITH A FULL RESERVOIR

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Abstract

Arapuni Dam is a 64m high curved concrete gravity structure across the Waikato River in New Zealand. It was completed in 1927 to form a reservoir for the 186MW hydroelectric power station. A series of foundation leakage events have occurred since water was first impounded. These were related to piping within, and erosion of, the weak clay infilling the defects within the volcanic ignimbrite rock foundation. The owner of the dam required the formation of a high quality and verifiable cut-off solution to be completed with the reservoir still in service. An international Alliance was formed to identify cut-off options, develop them and implement the selected methodology. The paper includes the following:

- Commentary on different solutions evaluated.
- Development of the adopted solution which required the installation of 400 mm diameter overlapping piles to depths of up to 90 m.
- Manufacture and development of specialized plant and equipment.
- Monitoring of the dam and foundation.
- Construction of the cut-off wall and quality control.

The outcome of this successfully implemented solution was the formation of a robust and verifiable cut-off wall. With few precedents for this type of work and none constructed in weak rock and to 90 m depth, the Arapuni Dam seepage cutoff project significantly extends international small diameter overlapping/secant pile technology and experience.

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Introduction

Arapuni Dam is a 64m high curved concrete gravity structure of crest length 94 m, on the Waikato River in the central North Island of New Zealand. It was completed in 1927 to form a reservoir for the 186MW hydroelectric power station. A series of foundation leakage events have occurred since water was first impounded. These were related to piping within, and erosion of, the weak clay infilling the defects within the volcanic ignimbrite rock foundation. Seepage changes have often involved sudden and significant increases, and cannot usually be related to external events, such as earthquakes.

The most recent seepage incident required grouting to fill an open void within a foundation defect in December 2001 to successfully control the deteriorating condition. Details of the grouting of the void in the fracture allowing high pressure seepage are described in Amos et al (2003b).

Seepage investigations prior to the emergency grouting established the location of the developing leak and the nature of the joint infill that was subject to piping, thereby enhancing the success of the targeted grouting operation. The concept used at Arapuni of evaluating seepage conditions in a targeted and safe manner before committing to remedial works is described in Bruce and Gillon (2003). Discussion of the overall process of monitoring, investigation and remediation for the high pressure seepage is also described in Gillon and Bruce (2002) and more detailed description of the investigation techniques employed are described in Amos et al. (2003a).

With the deteriorating condition arrested, the owner of the dam, Mighty River Power Ltd., required the formation of a high quality and verifiable cut-off solution to be completed with the reservoir still in service. A comprehensive investigation took place to determine the extent of foundation features requiring treatment to prevent further incidents from developing. A targeted and cost effective fix involving drilling and concreting overlapping vertical piles from the dam crest through the dam and underlying rock formation to a total depth of 90m was selected to form four separate permanent cutoff walls at selected locations beneath the dam. An international Alliance between the dam owner (assisted by their designer) and a contracting consortium was formed to identify cut-off options, develop them and implement the selected methodology. Construction of the cutoff walls commenced in September 2005 and was completed in mid 2007. Operation of the reservoir has not been affected and electricity generation has continued during the project works.

The Dam

The dam forms the reservoir for a 186 MW hydro-electric 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. The dam is shown on Figures 1 and 2.

Handman (1929) discusses the dam's construction. Original features of the dam include concrete cutoff walls and a network of porous (no-fines) concrete drains at the dam/foundation interface (the "underdrain"). The original cutoff walls extend beneath the dam to a depth of 65m below the dam crest and extend 20m and 33m 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 600mm high x 600mm wide "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 original cutoff wall. Radial porous drains discharge seepage water to the downstream toe, where seepage is measured at v-notch weirs.

In June 1930 the reservoir was completely dewatered for a number of repairs including construction of a grout curtain along the upstream heel of the dam and along the front of both abutment cutoff walls (Furkett, 1934). The grout curtain was a single row cement curtain with mostly vertical grout holes at 3m centres. It was constructed just upstream of the dam and cutoff walls, as shown on Figure 4, but is not physically connected to the dam. Figure 3 shows the extent of the grout curtain and original dam cutoff walls.

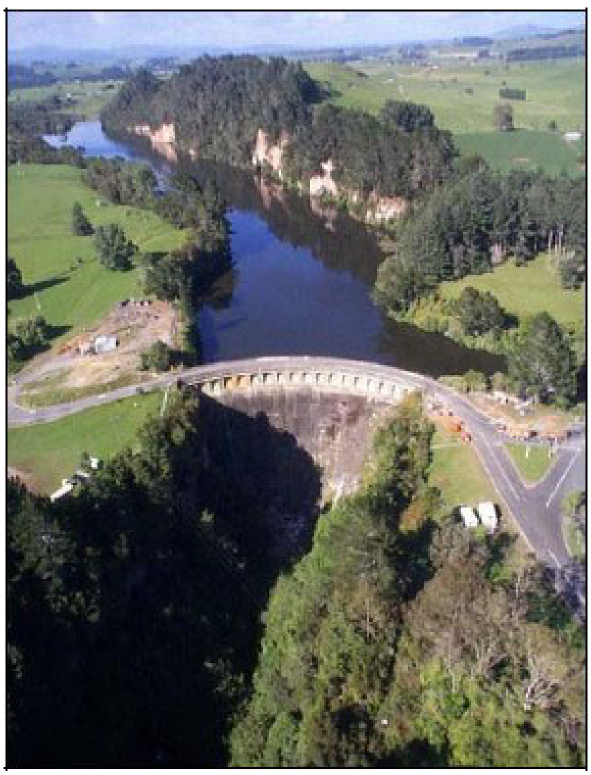

Figure 1. Arapuni Dam, New Zealand, looking West

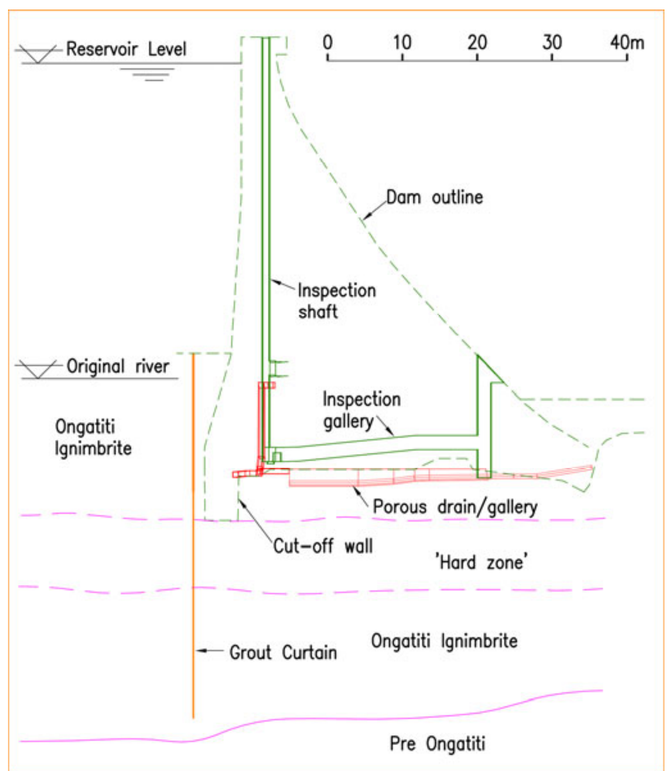

Figure 2. Cross Section of Arapuni Dam (Note the spatial separation of the grout curtain from the dam)

The Dam Foundation

The dam site is in an area of multiple ignimbrite flows from volcanic eruptions over the last 2 million years. The main dam footprint is founded on a 40-50m thick sheet of Ongatiti Ignimbrite (Figure 3), 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 original dam cutoff wall the Ongatiti is considerably stronger (up to 28MPa) and identified as the "hard zone" (Figure 3). Major sub-vertical defects in the form of cracks or fractures trending North-South are present in the Ongatiti. These fractures extend for the full depth of Ongatiti and vary in aperture from closed up to 80mm. The fractures relate to cooling (venting and contraction) of the ignimbrite after emplacement and are not tectonic in origin. Clay infill is generally present where the fracture opened around the time of emplacement. The fracture infill is nontronite, an iron-rich smectite clay with a very high moisture content and very low shear strength. This very weak clay is potentially erodible under pressure. Where infill was not present in fractures, seepage pressures correlating to reservoir level were present in some areas of open joints under the dam.

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

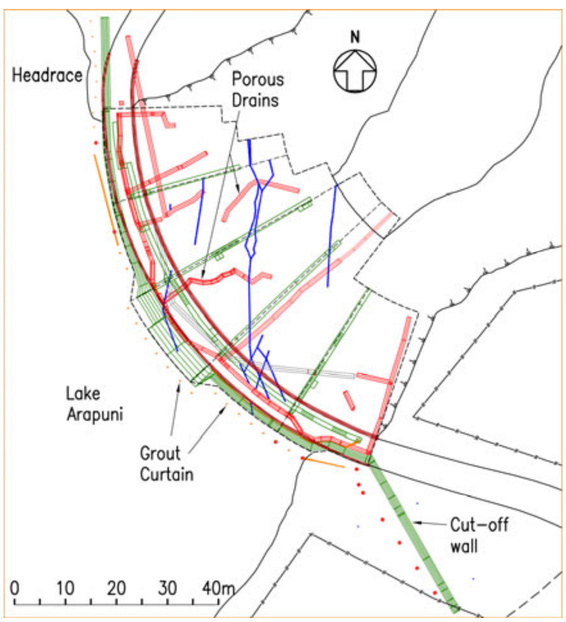

Figure 3. Plan view of Arapuni Dam
(The positions of the foundation fractures noted during construction are shown. The diversion tunnel is south of the right abutment cut off wall)

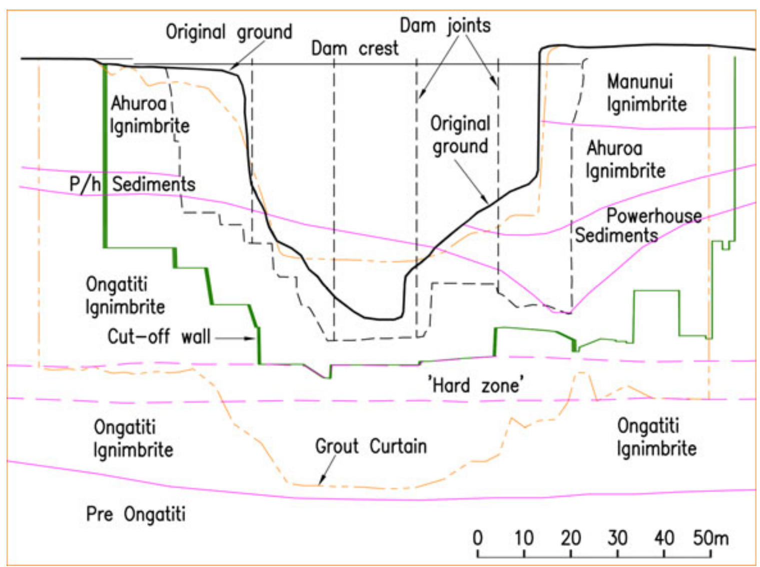

Figure 4. Elevation of Arapuni Dam, looking downstream

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 the Powerhouse Sediments (Figure 3), with a thickness of 4 to 8m.

The Seepage Problem

The seepage history of the dam (described in earlier papers such as Amos et al, 2003) includes several leakage connections identified between the lake and the dam foundation underdrain at various dates since first lake filling. The seepage paths appear to be quite long and complex and several remedial techniques were tried over the years, including the grout curtain in 1930 and bitumen grouting from 1935 to 1942. Seepage flow diminished from the 1950's (with no remedial works undertaken) until the new incident developed in the late 1990's. It is now evident that the various grouting works only filled voids where the vertical drillholes connected to open voids in vertical joints, leaving other leakage paths open. The most likely cause of seepage reduction is considered to be migration of fracture infill material gradually sealing seepage exit points.

Investigation of the seepage problem in 2001 indicated that an open zone was present under the dam and nontronite clay infill in the same fracture was eroding. If erosion were to migrate along the line of the fracture downstream of the void, then it was considered possible that an erosion pipe could connect to the downstream toe of the dam. There was genuine concern that high pressure could potentially blow-out remaining fracture infill at the dam toe, and the resulting jet of water then erode Powerhouse Sediments on the left abutment, destabilising the abutment rock face above. This same concern remained where other fractures with nontronite clay infill remained in the foundation without a permanent upstream cutoff.

Foundation Investigations to Determine Scope of Cut-Off Works

The 2001 seepage investigation (Amos et al., 2003a) primarily targeted the developing leak to determine its nature and extent. The investigation also looked wider than the immediate vicinity of the fracture, leading to the development of a groundwater model that describes the overall seepage behaviour in the dam foundation, including the seepage mechanism for the 2001 incident. An extensive programme of investigation core drilling and detailed foundation mapping was completed between 2002 and 2005 to determine the extent and nature of the fissure systems. Three major sub-vertical cracks or fractures were mapped during dam construction crossing diagonally across the dam footprint in an East-West orientation and a fourth set of fractures was identified in 2003.

A total of 86 cored investigation holes were drilled in the dam foundation after 2001, following the 36 holes that were drilled for the 2001 leak investigation. These holes were all angle holes drilled from the downstream face of the dam or from inside the dam galleries as appropriate.

Holes were generally angled perpendicularly across the fracture and logged by the geology consultant.

The investigations clearly indicated the zones where vertical joints were present, and hence the width of treatment panel could be determined. The important differences between the Ongatiti ignimbrite sheet in the dam foundation and the younger ignimbrite sheets in the Arapuni dam abutments are:

- the lack of orthogonal joints commonly seen in ignimbrites in this area
- the lack of joints in the areas between the four obvious fracture zones

Principles for Remedial Works

The assessment process following fracture grouting in 2001 identified two key issues relating to the fissure systems:

- The presence of highly erodible joint infill in the dam foundation that is vulnerable to piping erosion, and
- the presence of near-lake pressure in areas under the dam due to open fractures hydraulically connected to the reservoir.

Mighty River Power committed to upgrading the dam foundation seepage control measures so that the risk of further piping incidents would become extremely low and high pressures under the dam would be controlled. Furthermore, the objective was set to complete the remediation with no interruption to power station operations (i.e. maintain the reservoir at normal operating levels) to avoid the environmental (downstream effects of mobilising lake bed sediment) and business (electricity generation) impacts of lake dewatering. Therefore Dam Safety was an important consideration in selection of the final remediation technique.

The investigation findings allowed the remedial works to specifically target each of the four sets of identified vertical fractures and treat the open or infilled joint by removing infill and replacing the joint material with grout or concrete in order to create stable permanent barriers. The cutoff walls were located as far upstream as possible to restore the normally accepted uplift profile under the dam (Figures 5 and 6).

Contractor Procurement Using Alliance Framework

The project was implemented in the following stages:

- Stage 1 was the selection of preferred contractor
- Stage 2 was the Design Stage and required that the preferred-contractor work collaboratively with the design team to further develop the three nominated remedial options, determining; risks, opportunities and cost estimates of each to assist Mighty River Power with the selection of the preferred option. For this stage the contractor was

employed in a consultancy contract to work with the design consultant to develop the specification and design drawings. The works were priced and negotiated to agree and fix the Target Outturn Cost (TOC).

- Stage 3 was the construction of the selected option. For this stage the contractor signed an Alliance Agreement which set out the alliance principles, project objectives and incentives, cost and non-cost, for the owner and commercial participants.

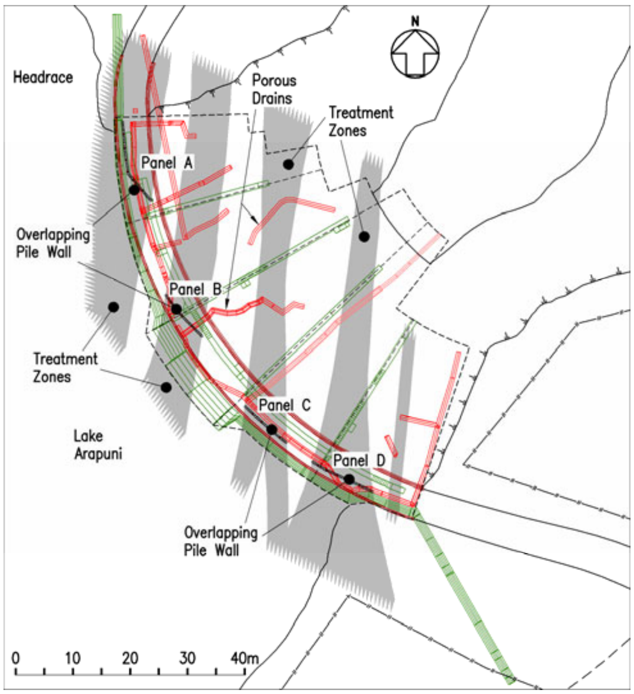

Figure 5. Plan of long-term seepage control remedial works,with cutoff walls, treatment zones and underdrain

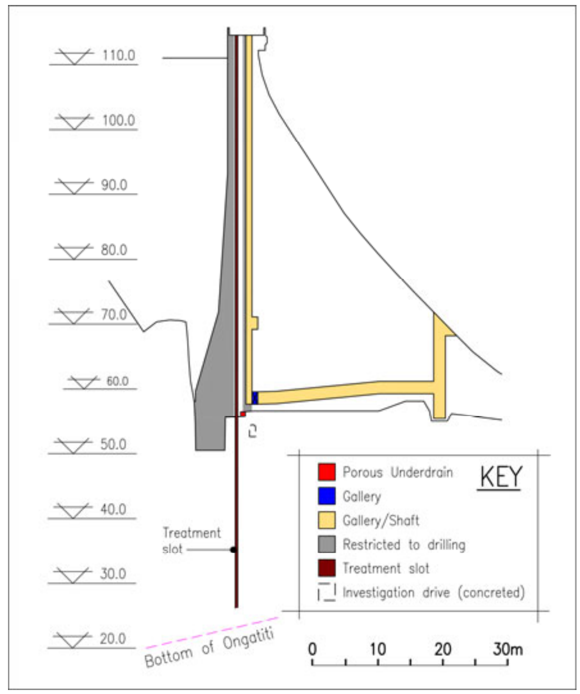

Figure 6. Typical cross section of dam at a contraction joint showing cutoff location and relationship with shafts, gallery and underdrain.

The commercial framework for the Alliance Agreement includes the following elements, described by Carter and Bruce (2005):

- a cost reimbursable component for direct costs
- a negotiated and agreed margin for overheads and profit;
- an agreed target outturn cost (TOC) developed during the ‘Stage 2 Design Phase’ together with gain share mechanisms for sharing cost savings or overruns between the commercial participants and the client;
- an incentive payment related to agreed project key performance indicators (KPI’s) for; quality performance, environmental and stakeholder management.

The principal reasons for choosing an Alliance for project delivery were:

- the clear need for contractor involvement in the selection and development of the preferred construction method and subsequent modifications as the works progressed, and
- the equitable sharing of dam safety and electricity generation risks in execution of the work, with a full reservoir, and
- to minimise the risk of contractual dispute.

The TOC was based on all parties' direct costs and onsite overheads (including the client). The TOC was independently assessed by Independent estimators and auditors appointed by the owner before final agreement was reached. The TOC included a contingency sum determined by the combined parties using a risk assessment process.

MRP selected the preferred contractor for the project through a call to pre-registered specialist foundation contractors. Given the unique nature of the project, the extension of foundation engineering practice beyond previous experience and the risks of construction with a full reservoir, it was considered vital to the success of the project that the team selected had the right mix of skills and could work collaboratively with the other project participants to develop and implement this project. Therefore the preferred contractor was selected on an attribute basis, with only a small commercial component in the scoring.

A consortium of two commercial participants Trevi SpA of Italy and Brian Perry Ltd of New Zealand were selected by the client Mighty River Power Ltd.

The client separately engaged the design consultant Damwatch Services Ltd to provide dam safety services to the Alliance and to provide owner's engineer services on site. The contract with the consultant did not include financial incentives, thereby ensuring independent safety advice was being provided at all times, in other words ensuring "best for dam" culture in the dam safety team. Contractural relationships are shown in Figure 7.

The human resources were drawn from the participant parties, with the Alliance Project Manager responsible for day-to-day project decisions, and the Alliance Leadership Team providing governance for the project on a unanimous decision-making basis. Project management relationships are shown in Figure 8.

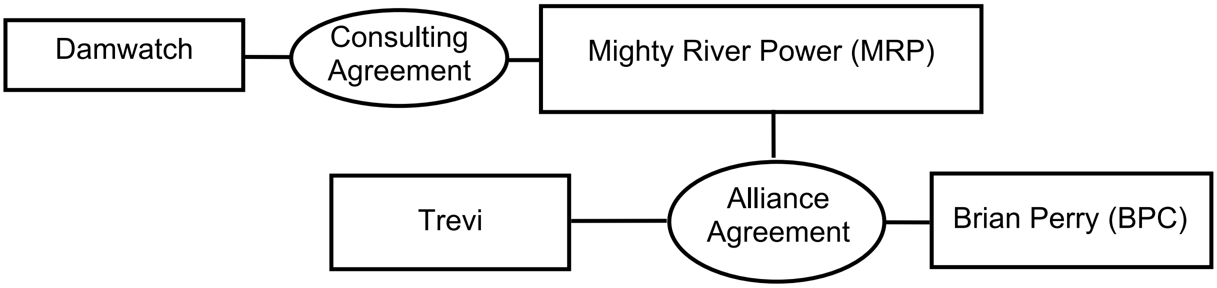

Figure 7. Contractual Relationships

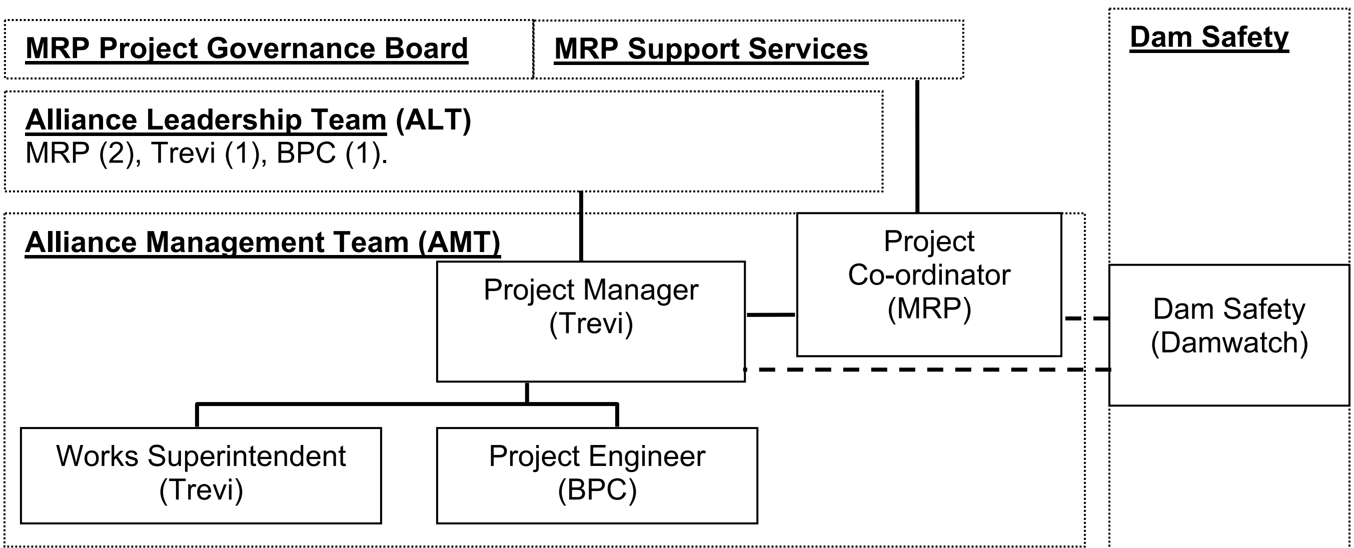

Figure 8. Project Management

Selection of Remedial Technique

Prior to engaging the contractor, several methods were considered by the owner for installing the cutoff barrier. Trials were performed of some technologies, such as the use of high pressure water and air jets to cut rock. Three remedial options were identified for further investigation:

1) Small Diameter Overlapping Piles: small diameter piles (≤450mm) drilled vertically from the crest of the dam to the bottom of the Ongatiti ignimbrite layer. Overlap of the piles is critical to the success of this option. Some tolerance can be allowed in drilling but the objective 'to minimise the likelihood of windows in the curtain' is stringent. The drilled holes are then concreted to form an effective replacement type cut off barrier roughly perpendicular to each of the identified fissures.

2) “Waterknifing”: targets the infilled joint, removes the clay and replaces it with a seam of grout. The method requires a fan of vertically raked holes to be drilled from the face of the dam and angled almost perpendicularly across the target treatment zone at approximately 1m centres. Infill clay would be removed by high pressure washing between the holes, using jet grouting equipment and grouted. The method would be very intensive and it was not possible to provide assurance that completeness of the “cut-off” could be predicted and verified. Furthermore the construction operation would be carried out from below reservoir level, increasing the dam and personnel safety risks.

3) Combination of Down-hole Wire Saw and High Pressure Jet Grouting: a combined process would be used to create vertical panels of grout under the dam. The proposed process would include drilling vertical 300mm diameter drill holes at 2 to 3m centres from the dam deck; then using modified diamond wire cutting technology to specifically create a vertical slot through the dam and foundation, followed by high pressure water jet blasting and jet grouting to break up the residual rock fragments in the slot and complete the grouted cutoff.

A diaphragm wall method using rock cutter equipment was considered but rejected because:

- the narrowness of dam crest and requirements to preserve historic parapet walls precluded use of this type of plant without extensive temporary works and,
- there was less geometrical flexibility to avoid drains and other features in the dam body.

A comparison of risk registers prepared for all three options identified that overlapping piles had the lowest associated risk considering technical objectives, constructability, cost and the safety of the dam during construction. The Waterknifing option had the highest risk. Final selection of the overlapping pile method was the result of collaboration between the owner, designer and constructors in association with specialist independent review.

Overlapping Pile Cutoff

A notable recent diaphragm wall dam foundation project with full reservoir has been completed at Walter F George Dam in Alabama USA (Simpson et al., 2006), where overlapping piles and diaphragm walls were installed in karstic limestone 30m below reservoir level at the upstream face of the dam. Small diameter (150mm diameter) overlapping piles have been successfully used to form a cutoff within the dam body at Rio Descoberto Dam in Brazil (Corrêa et al., 2002), thereby upgrading defective concrete while a full reservoir was present, but only to 38m maximum depth in concrete and not in the weak rock material encountered at Arapuni Dam. Elsewhere in the U.S., overlapping large diameter piled walls were used as cutoffs in karst at Wolf Creek Dam, KY (1975-1979) and Beaver Dam, AR (1992-1994) (Bruce et al., 2006). With relatively few precedents for this type of work and none constructed in such weak rock and to 90m depth, the Arapuni Dam project significantly extends international overlapping/secant pile technology and experience.

The overlapping bored pile wall at Arapuni Dam consists of 400mm diameter holes drilled at 350mm centres (Figure 9) to form the required overlap. The holes were drilled from the dam crest (i.e. above reservoir level) to minimise construction and personnel safety risks. The overlap was controlled by the use of a 400mm diameter guide piece attached to the drill string but running in the adjacent completed hole. Four discrete lengths of the wall were installed, to specifically target the four fissure systems shown in Figure 5 as follows:

Panel A – 15.45 m
Panel B – 9.85 m
Panel C– 9.85 m
Panel D– 11.95 m

The plan number of piles was 134 with a total drilling depth of 11,600 m.

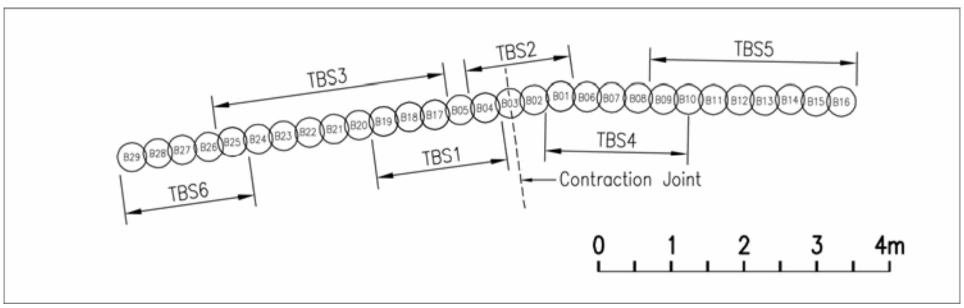

Figure 9. Plan of Treatment Panel B showing Slot sequence

The main reasons for selecting this construction method were:

- The “positive” cutoff concept offered by the overlapping bored piles was fundamentally the closest to a concept that would be used if the dam were to be built today;
- The chosen method was conceptually the simplest to construct and therefore there was high confidence in successfully accomplishing the treatment objectives with a quality assured outcome;
- The method meets all the technical requirements for construction with a full reservoir;
- By virtue of the equipment physically linking the hole being drilled to the previous hole, the resulting panel must provide assurance of a continuous cutoff;
- The selected option and methodology scored the lowest construction risk when compared to the other options considered, while not restricting construction alternatives if the methodology failed;
- Best cost/time profile: The selected option and methodology gave a construction cost estimate that had the lowest risks of construction cost overruns.

A rotary tricone drill bit with reverse circulation was the preferred drilling technology. While this is acknowledged to not be the fastest available drilling method, this method was considered to improve drilling accuracy, provide a suitably rough concrete finish (Figure 10) and reduce the risk of foundation damage that might occur with other drilling tools such as down-the-hole hammer.

Figure 10. Downhole photograph of overlapping drill holes

Total depth of the cutoff panels was set by the depth of the vertically jointed Ongatiti Ignimbrite. The panels terminate just above the interface with the underlying Pre-ongatiti Ignimbrite unit to avoid disturbance of the unwelded sediments between the ignimbrite units (Figure 11).

The cutoff wall was constructed in discrete segments or slots (Figure 9) to both:

- limit construction-induced tensile stresses on the unreinforced concrete dam face upstream of the cutoff wall; and
- limit the potential for weak foundation rock to collapse into the open cutoff slot before concreting.

The open holes were backfilled with concrete (or grout) using tremie concreting practices. Detailed finite element stress analysis of the upstream concrete face was carried out, particularly for the net outwards load during concrete backfilling. The rate of concrete rise was strictly controlled in the upper part of the dam to reduce tensile stresses in the upstream face due to lateral pressure from fresh concrete. The thermal stress state of the dam body at the time of the concrete pour was a key parameter for setting the rate of concrete rise. Vertical stressing rods were installed in the upstream face above lake level and tied back to the main dam body by steel straps to temporarily reinforce the upstream face of the dam above reservoir level against the net outwards forces in the slots from the reservoir of recirculation water required for drilling.

Drilling accuracy was important to avoid obstacles within the dam (such as drains and galleries) and to achieve the target cutoff area in the foundation rock. To reduce the risk of inaccurate holes, controlled directional drilling using a mud-motor was used to create initial highly accurate starter holes for each cutoff panel.

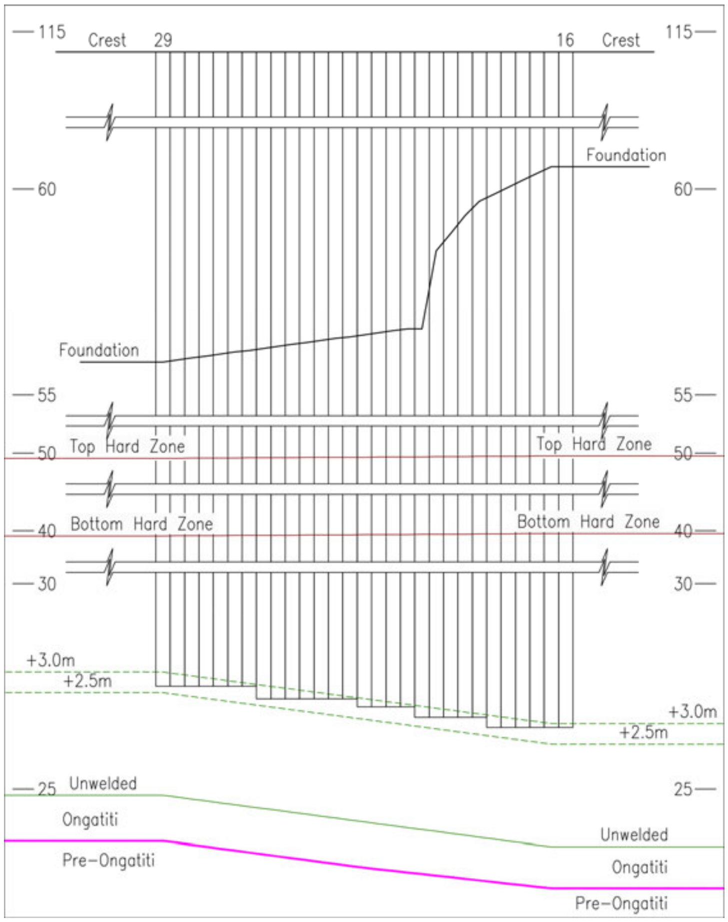

Figure 11. Typical Elevation of a Treatment Panel (Panel B)

Dam Safety During Construction

Because cutoff wall construction took place with a full reservoir present, there was an ever-present risk that the construction activities could have a detrimental effect on the fissures, potentially leading to erosion of fracture infill and the creation of a new leak under the dam. Detailed dam safety planning took place at the start of the project in conjunction with foundation coring and mapping. Sixty two electronic pressure and eighteen weir flow transducers were installed in the dam foundation at key locations. Piezometric transducers were installed in drill holes targeting fissures and other points of interest in the dam foundation. All drains were connected to dedicated v-notch weirs. Pressure relief holes were also drilled into fissures at the start of the project. These relief holes were normally shut, but were available to manage pressures in fissures during construction in the event that the risk of a leak developing became unacceptable. Discharge from the relief holes was measured at v-notch weirs.

A dedicated dam safety team was located on site throughout the construction period. A team member was required to be present during all construction shifts. The team was led by a very experienced dam engineer, with support from remote dam safety specialists as required. Twenty four hour monitoring was managed through transducers connected to multiplexers and a datalogger which sent raw transducer readings to a processing computer. The processing computer reduced the raw readings into engineering units, checked for trends outside preset alarm limits and dispatched alarm messages via email, pager, and SMS text messages to mobile phones. The readings were stored in a monitoring database for time dependent instrument data which was available to the site dam safety team in near real-time and also available to remote users via dedicated computer connections and an internet web site. The dam safety team also monitored turbidity and pH measuring transducers located in each weir box to identify fracture infill erosion or cement ingress into drains during slot backfilling activities.

Prior to construction a benchmark of pre-construction foundation behaviour was recorded. Piezometric behaviour in the dam foundation was quite dynamic when drilling works were underway. Behaviour was checked against precedent, and benchmark, conditions. Changing trends or dynamic conditions exceeding pre-construction levels were recorded and closely observed for indications of significant deterioration in foundation conditions.

The dam safety team was integrated with the construction team on site so that activities were coordinated and any change to the state of the foundation could be responded to rapidly. Regular communication occurred each day between these teams and contingency plans were in place to respond to a rapidly deteriorating condition in the dam foundation. The contingency plans ranged from changing drilling practices to emergency backfilling of slots and grouting of any open voids that were identified (similar to the 2001 fracture grouting) or (in the extreme but unlikely case) controlled lowering of the reservoir.

Verification of Completed Panels

Verification of the quality and successful completion of the works took place at several stages of cutoff panel construction. The requirement for a high level of quality assurance resulted in a minimum of two levels of verification for each of the key quality parameters.

1. Verticality, Continuity and Closure of the Treatment Zone
    - Readings from a bi-axial inclinometer taken at 2m intervals inside the drillrods as drillholes were being advanced to determine hole location with respect to the target zone for the cutoff (Figure 12).
    - Underwater camera surveys of slot walls to check rock conditions and verify fracture presence in rock face.
    - Sweeping each drilled slot with a steel frame to check that the slot met the minimum cutoff panel dimensions before backfilling.

2. Quality of Completed Cutoff Wall
    - Underwater camera surveys of the end of the adjacent completed slot concrete to verify concrete quality in adjacent completed work.
    - Flow meter surveys to check for concentrated seepage flows in fissures that could impair the quality of the new fresh concrete.
    - Carefully controlled tremie concrete operations and recording of any concreting problems that may require later testing.
    - Final verification by drilling with core recovery at locations of potential defects.

3. Foundation Response to the Completed Works
    - Post-concreting monitoring of downstream fissure pressures and drain flows and comparison with pre-construction benchmark behaviour.
    - Post-construction pressure response testing of the fissure downstream of the completed panel and comparison of results with similar pre-construction tests.

From these practices, any problem areas could be identified that required investigation drilling and core recovery to determine if further remedial works were needed. Regular assessment by specialist independent reviewers also took place throughout the construction phase.

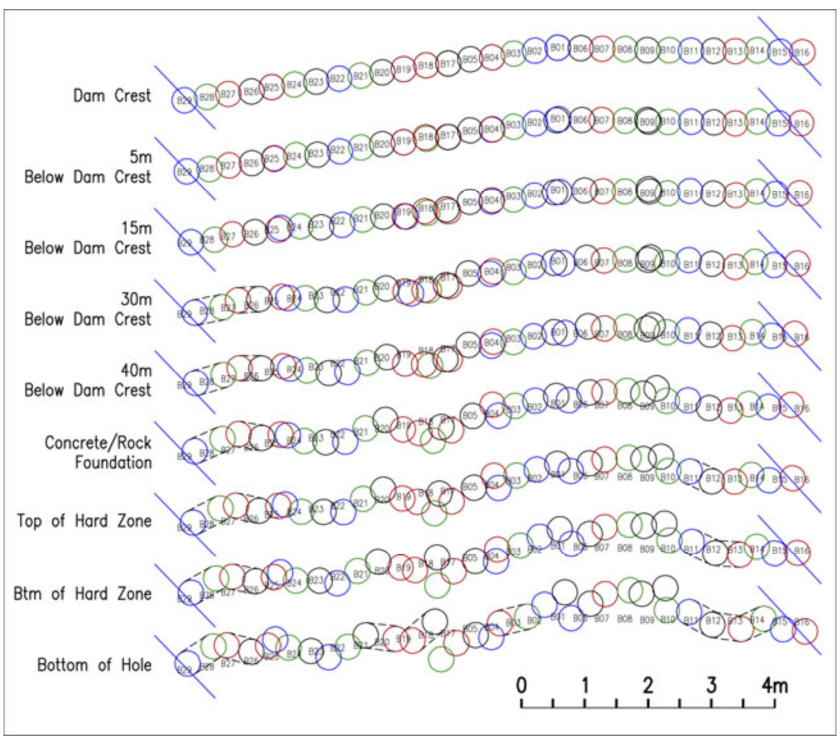

Figure 12. Elevation of Treatment Panel B, showing the relationship between inclinometer readings at depth for each drill hole

Conclusions

Arapuni Dam has had a history of foundation seepage incidents since first filling in 1927. Past seepage incidents have undoubtedly been related to erosion of clay infill in vertical joints in the ignimbrite rock foundation, allowing leakage paths to develop from the reservoir. The most recent leak was sealed in an emergency grouting operation in 2001.

The dam's owner, Mighty River Power Ltd, has undertaken a dam foundation enhancement project to construct concrete cutoff walls through the underlying ignimbrite sheet in order to prevent future leakage incidents from occurring. The project design and remedial works were reviewed by independent international specialists to ensure that the dam met internationally recognised dam safety standards.

The cutoff walls consist of overlapping 400mm diameter holes drilled through the dam and underlying ignimbrite sheet with a full reservoir. The construction technique required for 90m deep cutoff walls significantly extends international overlapping / secant pile technology. Construction was undertaken with close monitoring of the dam foundation to ensure that the construction activities did not generate another leak requiring emergency action and to ensure that the dam's safety was not compromised. The collaborative design process and use of the project Alliance procurement model delivered a mechanism for equitable risk share and reward for outstanding delivery of the solution and construction works.

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